DFI 41st Annual Conference on Deep Foundations

International Conference on Deep Foundations, Seepage Control and Remediation



2016 Proceedings

Seepage Control and Remediation

DFI 41st Annual Conference on Deep Foundations

International Conference on Deep Foundations, Seepage Control and Remediation

New York Marriott Marquis | October 12-15, 2016



CONFERENCE PROCEEDINGS

- SEEPAGE CONTROL AND REMEDIATION -

2016

New York, New York | USA

Deep Foundations Institute
326 Lafayette Avenue
Hawthorne, New Jersey 07506 USA
Tel: (973) 423-4030
Fax: (973) 423-4031
staff@dfi.org
www.dfi.org

Table of Contents

Case Histories on Risk Management and Long Term Performance

A Seepage Control Cutoff in an Extreme Saline Environment; *Iván Contreras, Jon Ausdemore, Rachel Leier, Aaron Grosser, Amr Ragy and Kevin Green …….. 1*

After Sandy: Bay Park Sewage Treatment Plant Flood Protection; *Gilbert Tallard, David Guy and John Stanbury …….. 13*

Remedial Drilling and Grouting at Abiquiu Dam, NM and Fluid Losses in the Embankment, a Case History; *Amy F. Ebnet, Suzanne R. Hess-Brittelle, Tyler J. Quick and Sharon L. Schulz …….. 23*

Diaphragm Wall Construction, Instrumentation, and Testing for a Hydroelectric Power Facility; *Jeff A. Segar, P.E., SE, Nicolas A. Willig-Friedrich and David Gatto …….. 33*

Current Research and Future Research Needs

Seismic Stability Analysis of Seepage Control Measures for Levees in Central Valley, California; *Zia Zafir, Ph.D., P.E., GE …….. 45*

Cutoffs for Dewatering Applications and Temporary Cofferdams

Analysis and Behavior of Double Sheet Pile Cofferdam; *Bashar S. Qubain, Jianchao Li and Kamil Nuzha …….. 55*

Cut-Offs in Temporary Coffer Dams as Seepage Barriers for Concrete Dam Construction – A Case Study of Teesta - VHE Project in Sikkim (India); *I. D. Dayal, Executive Director and Anil Kumar Jain, Chief Engineer …….. 65*

Design and Construction of the Cannelton Hydroelectric Plant Cofferdam and Cut-Off Wall; *James M. Tantalla, Roderic A. Ellman Jr. and Dwight Metcalf …….. 87*

Drawdown of Ground Water Control System in Deep Excavation Surrounded by Cutoff Walls: Numerical Investigation; *Dalia R. Mohamed, Marawan M. Shahien and Hisham H. Hafez …….. 97*

Ground Freezing for Groundwater Cutoff at the Seattle Seawall Replacement Project; *Daniel Mageau, P.E., Larry Applegate, Brenton Cook, P.E. and Aaron McCain, P.E. …….. 107*

Grouting of an Artesian Zone in Advance of Powerhouse Excavation for the Red Rock Hydroelectric Project; *Rachael V. Bisnett, P.E., Thomas G. Andrews, P.E., Dewey W. Geary, P.E. and R. Michael Bivens, P.E. …….. 117*

Jet Grouting for Groundwater Cutoff at Tie-In between Existing Tunnel and New Construction; *Chu E. Ho, Aaron Evans and Tom Richards …….. 127*

Metro Rome Line "C" - Artificial Ground Freezing Application for the Underpass of the Existing Metro Station San Giovanni - Line "A"; *Alessandro Bertero, Ferruccio Cribari, Roberto Sorge, Eliano Romani, Giulia M. B. Viggiani and Francesca Casini.…… 137*

Willow Island Hydroelectric Project: Taming the Ohio River to Create Green Energy; *James M. Tantalla Ph.D., P.E. and Roderic A. Ellman Jr., P.E. …… 149*

Data Management for Construction Optimization and Final Product Acceptance

Interactive 3-Dimensional Modeling of Anchors at Bluestone Dam; *Jamey Rosen and Georgette Hlepas …… 159*

Seepage Barrier Material Design and Placement/Installation

An Evaluation of Soil-Bentonite Backfill Consolidation Properties; *Jeffrey C. Evans, Hejintao Huang and Daniel G. Ruffing …… 169*

Bentonite Self-Hardening Slurry Mix Variations and Field Implementation Methods; *Christopher R. Harris …… 179*

Case Study: Feather River West Levee Program Cutoff Walls; *Jared P. Williams and Kyle R. Lackner …… 191*

Cockatoo Island Seawall Seepage Barrier - Permeation Grouting, Jet Grouting and a Soil/Cement/Bentonite Mixed Slurry Capping Trench; *Russell J. Denny and Chris L. Powell …..... 203*

Construction of the Tailings Storage Facility Freshet Management Embankment Cut Off Wall at Mount Polley Mine; *Keith MacKay, David Siddle, Luke Moger and Terry Eldridge …… 215*

Innovative Grouting Techniques for Installation of a Seepage Barrier at Barrage Des Quinze Dam (Quebec, Canada); *Michael Byle, D.GE, F. ASCE, Peter Bowman and Danny Déry-Chamberland, Ing. …… 225*

Seepage Barrier at Bolivar Dam; *Mohamed Elgouhary, Daniel Koziol and Carol L. Tasillo …….... 235*

Seepage Barrier Mix Design Development, Installation, and Evaluation; *George W. Little and John Councilman …… 245*

Use of Enhanced Pre-Mixed Self-Hardening Grout for an Emergency Cut-Off Wall Intervention in West Africa; *Bruno Vingiani, Claudio Asioli and Maurizio Siepi ……..... 255*

Wadi Dayqah Dam Curtain; *Frank Haehnig, Dr. Harald Krenn and Harm-Christoph Claassen ….. 263*

Solving Underground Problems in Water Infrastructure Systems

Analysis of Water Leakage in Downstream Dam Toe of Komani Dam; *Përparim Alikaj, Geophysical expert, Arjan Beqiraj, Hydrochemistry expert, Arjan Jovani, Director and Altin Karriqi, Geophysical expert* *275*

Cannonsville Dam Emergency Borehole Response; *Paul C. Schmall, P.E., John Vickers, P.E. and William H. Hover, P.E.* *287*

Gowanus Pumping Station: Jet Grout Bottom Seal and Deep Support Columns; *Paul J. Lewis, P.E., Vice President, Paul Schmall, Ph.D., P.E. Vice President, Chief Engineer, John M. Stanbury, Grouting Field Operations Manager and Kevin Clarke, Portfolio Manager* *297*

Improved Drainage by Drilled Filter Drains, Dam Safety Upgrade, Case Study; *Åke B. Nilsson, M. Sc (Civ. Eng.), Principal Engineer and Carl-Oscar Nilsson, M. Sc (Civ. Eng.), Dam Engineering Specialist* *307*

Rehabilitation of the Kölnbrein Dam/Austria; *Frank Haehnig and Kurt Kogler* *317*

Subsurface Characterization Practices & Selection & Design of Cutoff Technologies

A Review of Guidelines for Dewatering Design within the Abu Dhabi Municipality; *Juan J. Gutierrez, Raghav Ramanathan, M. Melih Demirkan, Jared Deible, Michael Rosenmeier and Mazen Elias Adib* *331*

Drilling and Grouting for the Rehabilitation of Contaminated Areas: An Italian Case History; *Raffaella Granata and Daniele Vanni* *345*

Laboratory Modeling of Internal Erosion in Dam Foundations; *John D. Rice, Kenneth E. Henn, Dave B. Paul and Brian H. Greene* *355*

Verification Methods and Acceptance Criteria for Seepage Barriers

Alternative Analytical Methods for Borehole Permeability Tests in Seepage Barrier Walls; *R. Lee Wooten, P.E., David R. Shields, P.E. and Darren D. Clark, P.E.* *367*

Assessment of the Durability of Cementitious Barrier Materials; *Stephan A. Jefferis, Professor and Adrian H. Bath* *385*

Diaphragm Walls for Seepage Control and Remediation: Three Case Histories; *Raffaella Granata, Mario Mauro and Fabio Santillán* *395*

Unified Specifications for Soil-Mixed Seepage Cutoff Walls; *Erik James, P.E., GE, PG, Jose R. Gomez, P.E. and Wayne S. Smith, Ph.D., P.E., GE* *407*

PREFACE

On behalf of Deep Foundations Institute (DFI) and the 2016 Conference Organizing Committee, we extend our thanks and gratitude to all attendees, exhibitors, underwriters, invited speakers, and presenters who helped make the DFI International Conference on Deep Foundations, Seepage Control and Remediation (41st Annual Conference) a success. The Conference was held in New York City from October 12-15, 2016.

As we prepared to host this conference in the The Big Apple for the first time since 2008, it was remarkable to witness just how much it has changed and reshaped itself since September 11, 2001. Moreover, we were reminded how integral DFI members were, and still are, in the history of the construction of some of the world's most famous landmarks. During the 1960s, DFI members served on design and construction teams for the diaphragm wall systems, underground garages, and tunnels under and adjacent to the original World Trade Center towers. During the 2000s, other members provided design and construction expertise to support the existing substructures and transit lines during the recovery, then expansion and renewal process. DFI members have a rich, laudable history in Manhattan, the outer boroughs, and metropolitan region where they have designed and built foundations for skyscrapers, incredible underground facilities, new tunnels to transit system capacity, and retrofit systems for increasing the longevity of existing structures. More recently, key DFI members became involved with urban sustainability efforts in the city, which are critical in the aftermath of Hurricane Sandy's destruction of the region in 2012.

The DFI 41st Annual Conference featured two concurrent technical tracks focused on (1) deep foundations and earth retention, and (2) seepage control and remediation. Together these sessions provided an international forum for a wide range of geo-professionals to gather together and present, discuss, and debate all aspects and current challenges pertaining to the advancement of the deep foundation, earth retention, and seepage control and remediation industries. This conference stressed the importance of the work performed, the quality thereof, and the key role DFI members have throughout the process. We kept to the conference goals of analyzing the present to better prepare for the future and its uncertainties, while recognizing and striving to develop areas for improvement and innovation.

More than 90 technical presentations were delivered and discussed by industry experts from more than 12 countries including the U.S.A., Canada, U.K., Australia, Germany, Italy, Austria, Sweden, Belarus, Albania, Egypt, India, and China. The deep foundations track focused on planning and mitigation, design and construction, project delivery, communication and training, incorporation of emerging technology, data management, risk and liability, and advancements for deep foundation and earth retention systems. The seepage control and remediation track focused on cutting-edge information on technologies for ground improvement and seepage remediation, dam and levee design and construction guidance for controlling seepage, best practices in dam and levee safety risk assessment and management, and the latest research on internal erosion and piping mechanisms. The seepage control and remediation track was developed to concentrate attention on the importance of water, as we consider it the most precious and most feared natural resource; there is no life without water and, simultaneously, lives are taken away by water's power if it is not contained. Worldwide, human tragedies and the economic damages produced by scarcity of unstoppable flooding water are incalculable. This publication contains the peer-reviewed technical papers pertaining to the session topics described above and as selected by the Conference Organizing Committee for 2016.

We were honored to host four keynote presenters this year. Dr. Satoru Ueda, leader specialist at the World Bank, was this year's Conference Keynote Lecturer. He shared the crucial role of the World Bank's involvement, strategy, policy, and lessons learned while supporting worldwide projects and programs. George J. Pierson, president and CEO of Kleinfelder, was this year's Deep Foundation Keynote Lecturer. He spoke about his career and his expertise in risk management, mitigation, and resolution. James C. Dalton, P.E., director of civil works for the U.S. Army Corps of Engineers, was this year's Seepage Control and Remediation Keynote Lecturer. He delivered a presentation on the incredible challenges to remediate the Mosul Dam (Iraq), which is considered the most unstable and seepage related problematic dam in the world. Notably, Dr. Mae Jemison, former NASA Astronaut, delivered this year's Hal Hunt Lecture entitled "Exploring the Frontiers of Science and Human Potential." She addressed the

achievements and obstacles she faced throughout her life as a NASA astronaut, Peace Corps volunteer, and principal of the 100 Year Starship Project.

This year's conference included networking and social opportunities for attendees including DFI's Annual Awards Reception and Banquet, Women in Deep Foundations Reception, a business meeting luncheon, welcome reception and more. Members of DFI's 21 technical committees and working groups also gathered to discuss, debate, and offer solutions to the various challenges affecting the deep foundations industry. In addition, more than 125 exhibits were featured and showcased by member companies with innovative products, highly technical methods, their latest materials, and professional services.

Undoubtedly, without the hard work and dedication of all those involved, this Annual Conference would not have been possible. We, especially, would like to express our sincerest gratitude to the conference organizing committee, session chairs and moderators, authors, reviewers, underwriters, exhibitors, and volunteers. As we have come to expect and appreciate, t DFI has done an amazing job organizing, preparing, guiding, supporting, corralling, and gently prodding to ensure the Conference was executed flawlessly. We would be remiss if we did not acknowledge specifically the efforts of DFI Executive Director Theresa Engler, Events Manager Lauren Nance, and Technical Activities Manager Mary Ellen Bruce Large, P.E., D.GE-, and all staff at DFI for making this event possible.

In closing, we express our appreciation for the opportunity to be a part of the DFI 41st Annual Conference on Deep Foundations, Seepage Control and Remediation. We look forward to next year's event in New Orleans, Louisiana!

Conference Co-Chairs, Deep Foundations

Dr. Antonio Marinucci, M.B.A., P.E.
Independent Consultant; Research Professor at NYU

Silas C. Nichols, P.E.
Federal Highway Administration

Conference Co-Chairs, Seepage Control and Remediation

David Paul, P.E.
U.S. Army Corps of Engineers

Gianfranco Di Cicco
GDConsulting, LLC

A SEEPAGE CONTROL CUTOFF IN AN EXTREME SALINE ENVIRONMENT

Iván Contreras, Barr, Minneapolis, MN, United States, 952.832.2882, IContreras@barr.com
Jon Ausdemore, Barr, Minneapolis, MN, United States, 952.832.2611, JAusdemore@barr.com
Rachel Leier, Barr, Minneapolis, MN, United States, 952.842.3558, RLeier@barr.com
Aaron Grosser, Barr, Minneapolis, MN, United States, 952.832.2609, AGrosser@barr.com
Amr Ragy, Treviicos, Boston, MA, United States, 617.788.9837, ARagy@treviicos.com
Kevin Green, Ciner, Green River, WY, United States, 307.872.7309, kgreen@ciner.us.com

ABSTRACT

Construction of a new tailings dam was required to increase tailings storage capacity at a trona mine near Green River, Wyoming. The new dam was founded on overburden soils underlain by both fractured and competent bedrock. The stored tailings and water had high salt content, resulting in salty seepage water. To reduce the impact of the salty water on groundwater and the Green River, the design of the new tailings dam included a 257,260 SF (23,900 m^2) seepage cutoff extending into competent rock.

An extensive subsurface characterization was performed prior to completing the design and suitable seepage cutoff methods were evaluated. The approach selected was a full replacement panel using hydromill and clamshell for construction and self-hardening slurry (SHS) for the wall material. Laboratory testing was performed and ongoing monitoring has continued to evaluate the hydraulic conductivity and seepage cutoff performance in the briny environment.

The paper includes project background, the collaborative approach and outcomes of the QA/QC program, and the monitoring results for the seepage cutoff wall.

Keywords: trona mine, saline environment, seepage cutoff wall, self-hardening slurry (SHS), hydromill, test-section deconstruction, QA/QC program, monitoring instrumentation

PROJECT BACKGROUND

Ciner Wyoming, LLC (formerly OCI Wyoming, LLC) owns and operates the Big Island Mine facility near Green River, Wyoming. This underground trona mine has produced soda ash since 1962. The Greater Green River Basin covers much of southwestern Wyoming and extends into northeastern Utah and northwestern Colorado. The Ciner facility is located in the Green River Basin, which is part of the Wyoming Basin physiographic province. The Green River runs to the west and the south of the Ciner facility.

Beginning in the early 1990s, Ciner primarily disposed tailings underground in mined-out panels. However, in recent years, underground storage has become limited and some disposal has transitioned to the ground surface. Long-term planning revealed a need to expand Ciner's surface tailings disposal capabilities. Pond 2, a previously permitted disposal area, was identified as the most viable location for a storage facility. The Pond 2 site is directly north of the Ciner plant facilities, northwest of the active Upper Delta tailings storage facility, and northeast of the Green River (Fig 1).

Surface tailings deposition to Pond 2 began in 1970; however, excessive seepage was noticed downstream in the rock outcrops. After modifications to reduce seepage, tailings disposal resumed in 1973 but stopped in 1974 due to rising water levels, as indicated by nearby piezometers. Pond 2 has not since been used as an active tailings disposal location because the potential for high-salinity process water to seep into groundwater and flow to the Green River is a major regulatory concern.

© 2016 Deep Foundations Institute



Figure 1. Pond 2 site location

The surficial geology of the Pond 2 site consists of three distinct units—unconsolidated sediments (quaternary alluvium and colluvium), fractured bedrock, and competent bedrock (tertiary Bridger Formation). As the Green River meandered in its channel, it deposited alluvial materials consisting of silt, sand, and gravel in the meander belt. In the vicinity of Pond 2, there is a zone of rounded gravel and cobbles, up to approximately 5 feet (1.5 m) thick. Fractured bedrock underlies the unconsolidated deposits in most locations and typically ranges in thickness from 10–30 feet (3–9 m). This zone has been known as a preferential path for groundwater and seepage from the tailings basin. The relatively high permeability of this fractured zone is related to the presence of joints, fissures, and fractures in the underlying siltstone, mudstone, and sandstone, associated with stress relief and weathering. The degree of fracturing decreases with depth; the competent Bridger Formation does not transmit significant amounts of water. The groundwater has historically flowed southwest through the site towards the Green River.

Management of seepage is important for controlling the environmental impact of Ciner's tailings basin on the Green River. In addition, seepage through the fractures has the potential to create high pore pressures in the foundation of the dam embankment and create seepage pathways at the embankment interface. This could make Pond 2 containment dams vulnerable to piping and failure. A seepage cutoff wall was the first key feature of the Pond 2 Development Project (installed prior to the construction of the containment dam) due to the history of excessive seepage from the Pond 2 tailings disposal facility.

© 2016 Deep Foundations Institute

SUBSURFACE INVESTIGATION AND MONITORING INSTRUMENTATION

Two geotechnical investigations were performed to assess subsurface conditions at the site and install instrumentation to monitor the seepage cutoff wall and future containment dam. The investigations included soil borings and rock coring, monitoring while drilling (MWD), test pits, and packer testing for permeability of the stratigraphy. The objective was to carefully identify the fractured rock zones. This would facilitate the design of a seepage cutoff wall extending through the fractured rock into competent rock, where it would be embedded.

A geophysical survey was also conducted along the centerline of the proposed cutoff alignment using Multichannel Analysis of Shear Wave (MASW) technology. The results provided a profile of shear wave velocity along the cutoff alignment that correlated with the degree of rock fracturing and quality of rock mass.

In 2014, six fully screened (FS) piezometers and six pairs of nested vibrating wire (VW) piezometers were placed approximately 100 feet (30 m) downstream of the seepage cutoff wall (upper VW in overburden, lower VW in rock). Six additional pairs of nested VW piezometers were placed approximately 100 feet (30 m) upstream of the seepage cutoff wall and parallel to the downstream VW piezometers. This allowed monitoring of groundwater and pore pressures near the seepage cutoff wall and the future containment dam (Fig 2).

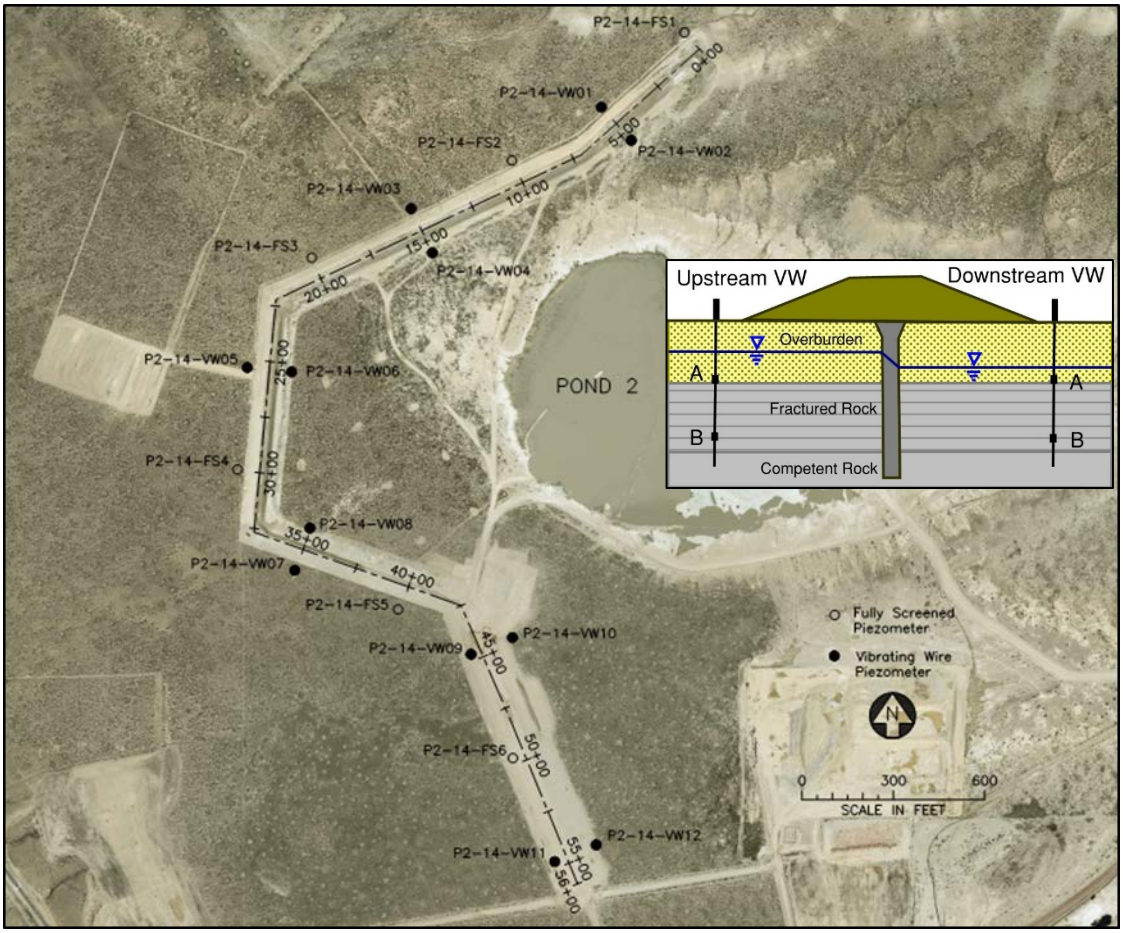


Figure 2. Pond 2 monitoring instrumentation

© 2016 Deep Foundations Institute

DESIGN OF SEEPAGE CUTOFF WALL

Successful design of the Pond 2 seepage cutoff wall required careful consideration of the depth of the seepage cutoff wall; the strength, permeability, and durability of the wall material; and the wall thickness.

Depth of the seepage cutoff wall. Information from the field investigations (rock coring data, MWD, packer tests, shear wave velocities) and engineering judgement were used to interpret the rock mass properties and define the top of the competent rock. The target rock mass permeability was lower than 5 Lugeon, with a rock quality designation (RQD) greater than 85% and a shear wave velocity of at least 1,750 ft/s (530 m/s). The unconfined compressive strength averaged around 1,750 psi (12 MPa) and was used as reference value for the competent rock at the Pond 2 site. The top of the competent rock along the alignment was interpreted from comprehensive evaluation of field investigation results. The bottom of the cutoff was designed to be at least 5 feet (1.5 m) into the competent rock layer; the adequacy of this penetration was verified by seepage modeling. Figure 3 shows an example of the information compiled to determine the bottom depth of the wall.

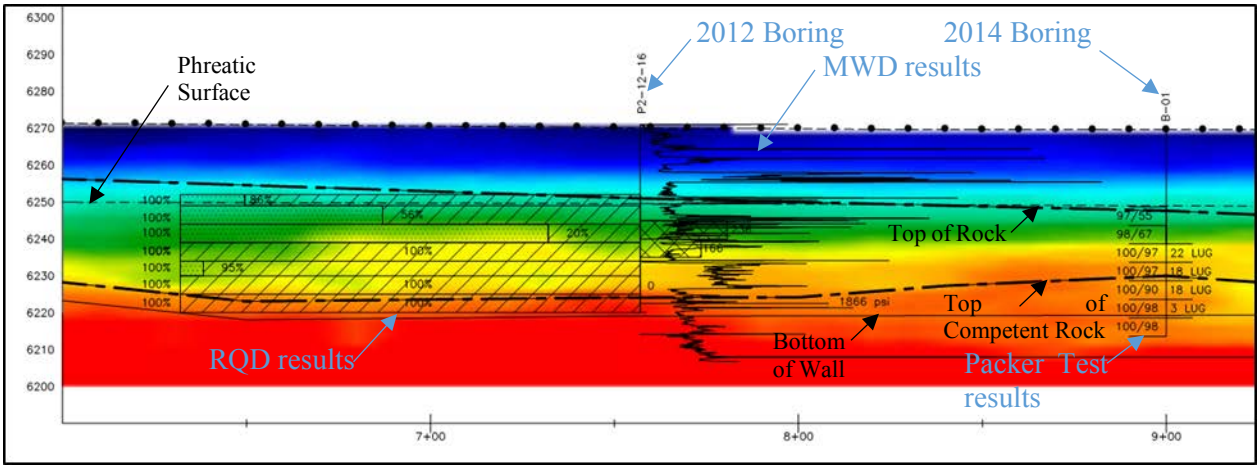


Figure 3. Seepage cutoff wall bottom depth

Strength and permeability of seepage cutoff wall. The wall had to be strong enough to support the containment dam and tailings depositions. It also needed to prevent seepage of salty water from the Pond 2 tailings disposal site. The wall and containment dam system were modeled using seepage, stability, and deflection modeling programs. It was determined that the seepage cutoff wall should have a minimum unconfined compressive strength of 60 psi and a maximum permeability of 1.0 x 10^{-6} cm/s.

Durability of the seepage cutoff wall. Salt in the groundwater would affect the durability of the seepage cutoff wall. Based on measured total dissolved solids (TDS), the salinity of the pond water at the Ciner tailings basin is 5.7 times greater than sea water and the sulfate concentration is 4.5 times higher. To avoid deterioration of wall material in this saline environment, the team used potable water and sulfate-resistant cement for the slurry.

Thickness of seepage cutoff wall. The seepage cutoff wall needed to be thick enough to generate adequate head drop while withstanding seepage forces. The thickness of the seepage cutoff wall was modeled extensively and designed to be 30 inches (76 cm).

Due to a lack of published information on the performance of seepage cutoffs in this type of extreme saline environment, the design team relied on GeoStudio seepage and stability modeling and FLAC deflection

© 2016 Deep Foundations Institute

modeling, as well as a panel of experts who reviewed the design, construction technology/methodology, QA/QC program, and monitoring results.

WORKING WITH CONTRACTOR TO FINALIZE DESIGN

A unique aspect of this project was the contractor's involvement in prior to the bidding process and during final phases of the seepage cutoff wall design. After the 2012 investigations, Ciner and Barr requested preliminary pricing and schedules from five geotechnical contractors to extend the wall into the competent rock layer. This allowed for the owner to plan ahead with funding and gave the engineer a better basis for writing appropriate performance-based specifications that would provide the bidders with some freedom to propose methodology, as long as the goals of the project were achieved. Treviicos, a contractor that proposed using a hydromill and self-hardening slurry (SHS) for backfill material, was selected. One key benefit of this approach was that it would remove the salt-impacted native materials and replace them with an engineered slurry better suited for the harsh saline conditions. Ciner and Barr worked closely with Treviicos to complete final design of the seepage cutoff wall. This entailed refinement of the final layout, development of the wall material design, a QA/QC program, and finalizing acceptance criteria.

SHS SEEPAGE CUTOFF WALL CONSTRUCTION METHOD

The Pond 2 seepage cutoff wall is approximately 5,500 feet (1,676 m) long. Stationing progressed southwest from the north abutment, then wrapped around the west side of the Pond 2 site, stopping along the south access road (Fig 2). The 30-inch-wide (76-cm) homogeneous seepage cutoff wall was installed using the SHS method. This involved excavation of the overburden and rock materials under the head of the SHS, which sets up after installation to serve as the final wall. The replacement process includes cycling a portion of the excavated material in the form of sands and fines back into the SHS. The fines content is variable, dependent on the materials being excavated; on this project the average SHS density increased from 1.13 (fresh SHS at batch plant) to approximately 1.35 (in-trench SHS). A primary/secondary panel excavation sequence was used with hydromill and clamshell equipment. The clamshell began pre-excavating each "bite" of a panel until the hydromill could be submerged safely in SHS to complete the excavation. The engineered SHS was prepared in an automated batch plant and pumped to the active wall panels. Using this construction method, the wall's homogeneity, continuity, verticality, strength, and permeability could be closely monitored and inspected for compliance with requirements and specifications.

A desander removed the excavated native material from the circulating SHS with a large sieve, and the coarse materials and cuttings could be seen, aiding observation of the stratigraphy during each panel bite.

The SHS mix stabilized the excavation during wall installation and was left in the trench as the final wall material. It was designed to be minimally impacted by the pond water while meeting requirements related to strength and permeability. Other considerations were the ability of the mix to consistently maintain a viscosity, bleed percentage, and cure time that allowed for constructability. Due to its high salt content and potential negative impact on SHS durability, pond water was not used. Instead, the mix included a binder of cement (Type II-V with high sulfate resistance) and slag, hydrated bentonite, water filtered from the Green River, and a dispersing agent additive.

The contractor's program for developing the mix entailed laboratory design and bench-top tests followed by in-situ adjustments during test-section construction. Through lab testing, the contractor developed several mix configurations that the designer accepted, including a richer mix used at the start of the project. The contractor was allowed to use a second, lighter mix after results from the full-scale test section with the richer mix were verified.

© 2016 Deep Foundations Institute

QA/QC PROGRAM AND INSTRUMENTATION MONITORING RESULTS

The contractor's QC team and the owner's QA team collaborated closely throughout the construction process; therefore, this paper refers to one QA/QC team. This team observed construction of the offset and full-scale test sections; verified geometry controls of the depth, alignment, and width; and verified the homogeneity and integrity of the seepage cutoff wall during construction. The team performed frequent testing on the water source, hydrated bentonite, fresh SHS at the batch plant, and the SHS material sampled from the slurry trench prior to casting cylinders for permeability and strength laboratory testing.

The instrumentation monitoring program involved daily manual readings of the fully screened piezometers along the wall alignment and weekly data downloads from the VW piezometer data loggers. The monitoring program included evaluation of the installed instrumentation response to groundwater pressure and seepage during wall construction.

Test sections

An offset test section and full-scale test section within the main wall alignment were first installed to confirm that construction procedures and the SHS mix design complied with requirements. Figure 4 shows the location of the test sections.

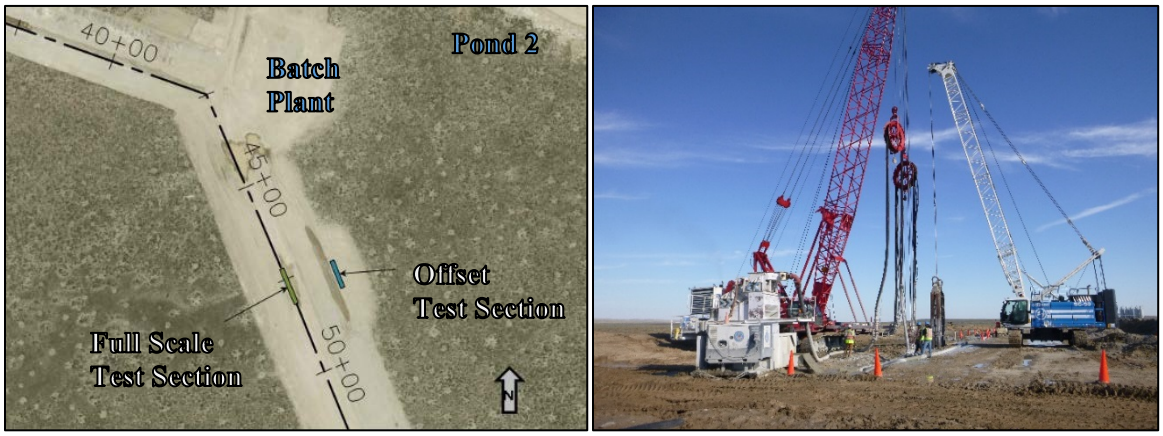


Figure 4. Pond 2 seepage cutoff wall test-section locations (left); photo of offset test-section construction (right)

Due to the low strength requirements for the SHS wall material, the team and expert review panel were concerned that coring would damage the samples (e.g., cause micro-fractures) and that this, in turn, could lead to unrepresentative permeability test results or wall damage. Instead, the project team elected to conduct field testing on the offset test section after it had cured at least 28 days. This necessitated taking fewer cores (five), along with wet grab sampling, laboratory testing of the cured cylinders and cored samples, excavation to expose the wall/soil interface, and complete deconstruction to observe continuity at the joints and homogeneity of the hardened SHS material.

The offset test section consisted of two primary bites and a centered secondary bite extending the full depth of the wall. Installing the bites in this sequence provided a test section that was representative of the primary/secondary panel.

Figure 5 shows a summary of the unconfined compressive strength (UCS) test results versus the wall depth for both the SHS cylinders collected during installation and the core samples taken after the wall had hardened in place. The cylinder sample results were used as the acceptance criterion for the UCS results.

© 2016 Deep Foundations Institute

The circle symbols in Figure 5 represent UCS results for SHS wet grab samples cured at least 28 days. The solid square symbols are UCS results for the cored samples removed from the hardened wall after they had cured at least 28 days. The individual acceptance criterion is indicated by the dashed line at 60 psi (414 kPa). The average strength results in the upper portion of the wall were lower than those for the rest of the wall profile, and the wall strength varied consistently below the top 5–10 feet (1.5–3 m). The difference in the upper portion of the wall is likely attributable to a combination of settling, water removal, wall top-off procedures in the early stages of curing (where SHS without aggregate is added to the top of the trench), and climate factors that only affect the shallow portion of the wall.

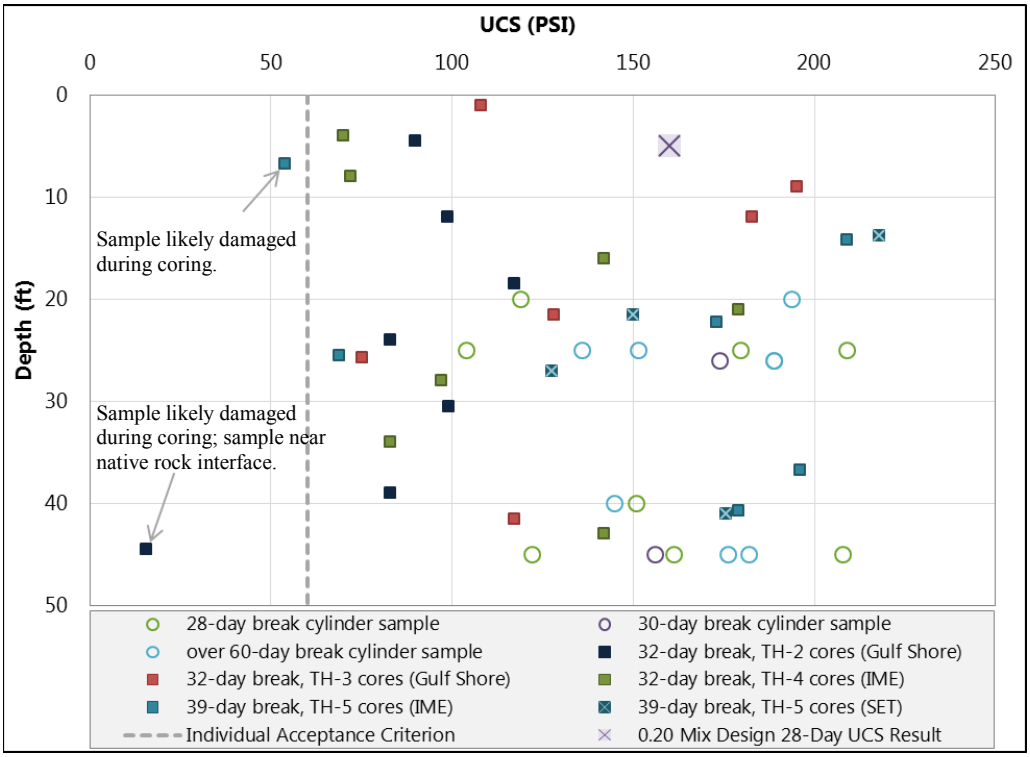


Figure 5. Offset test-section UCS vs. depth results

Figure 6 shows the permeability results of the offset test section. The dashed line represents the maximum specified permeability of 1.0E-06 cm/s. The circles show permeability results for SHS wet grab samples cast into cylinders and cured at least 28 days. The squares represent permeability results for the cored samples removed from the hardened wall after it had cured at least 28 days.

© 2016 Deep Foundations Institute

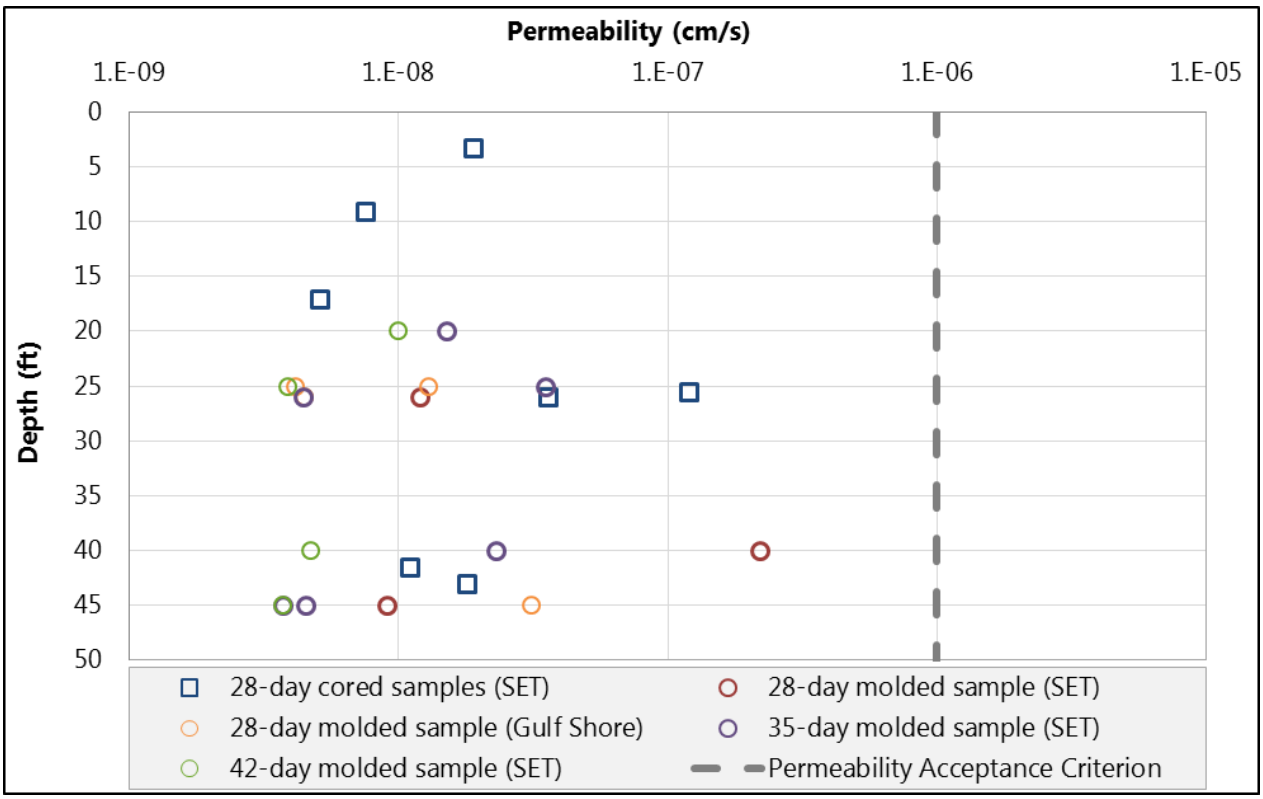


Figure 6. Offset test-section permeability vs. depth results

The offset test section was deconstructed in August 2014 and observed by members of the QA/QC team, including the project engineer and Dr. George Filz (a member of the expert review panel). Deconstruction provided evidence of the consistencies in depth, width, and integrity of the hardened wall. Figure 7 shows the test section being exposed and inspected by the project team and the SHS wall material being demolished.

This testing program was used as an acceptance criterion for the Pond 2 seepage cutoff wall. The successful results and observations during offset test-section construction allowed the project to move forward and eliminated the need for destructive testing on the main wall.

Field verifications and geometry controls

The geometry of the seepage cutoff wall was monitored carefully during construction. The main geometry control verification methods in the QA/QC program measured depth, width, verticality, and panel overlap. Wall depth and verticality was measured using the depth-meter and inclinometer instrumentation on the hydromill. During test-section construction, the contractor calibrated instrumentation and set up its Drilling Mate System (DMS) to monitor live parameters during installation. DMS depth records and verticality were submitted for each bite of the wall, allowing the QA engineers to monitor the DMS system during installation and verify depth with known depth markings on the hydromill slurry return hoses.

© 2016 Deep Foundations Institute



Figure 7. Offset test-section deconstruction

Guide walls provided the initial width for the clamshell or hydromill to excavate the wall panels. The hydromill frame-width dimension was used as the consistent minimum wall-width measurement. Excavation was performed under SHS, which kept the excavation open and prevented cave-ins. The QA inspectors observed that when the hydromill exited the trench and re-entered to resume excavation, minimal material returned through the desander until the previous depth had been reached. This indicated that the slurry was maintaining an open trench. Deconstruction of the offset test-section panel provided additional confidence that the wall width remained intact throughout excavation and curing.

The QA team carefully reviewed panel joint locations in the field and evaluated the full depth of the joint using DMS records to verify adequate overlap between panels. Deconstruction of the offset test-section panel provided additional confidence in the wall joints.

SHS testing

QA/QC testing was performed on the SHS before and after the mix was pumped to the wall excavation. Prior to pumping, the SHS was tested for temperature, density, viscosity, and bleed to verify that expected rheological properties were achieved. After the SHS was sent to the slurry trench, wet grab samples were collected as the slurry cycled through the desander equipment. The depth from which the samples were collected was randomized by the QA/QC team; however, sampling was more frequent at depths within the fractured rock zones. SHS wet grab samples were cast into cylinders for unconfined strength and laboratory permeability testing.

UCS

The UCS results were used as acceptance criteria for the wall. There were 135 UCS strength test results for samples at least 28 days old; all of these met the requirement for a minimum of 60 psi (410 kPa) for any single test and a minimum of 75 psi (515 kPa) for the moving 10-point average. The minimum UCS result was 61.5 psi (424.0 kPa), the maximum was 239.2 psi (1649.2 kPa), and the average strength was 164.6 psi (1134.9 kPa).

Permeability

The permeability test results were used as acceptance criteria for the wall. There were 132 permeability test results for samples at least 28 days old and all met the requirement for a maximum permeability of 1.0×10^{-6} cm/s. The minimum permeability result was 9.83×10^{-10} cm/s, the maximum was 3.24×10^{-7} cm/s, and the average permeability was 1.66×10^{-8} cm/s.

© 2016 Deep Foundations Institute

Extended SHS permeability test

An extended permeability test was performed to evaluate the behavior of the SHS when subjected to salty pond water over time. Figure 8 shows the long-term permeability test results for one sample exposed to high salinity pond water from the Ciner active water storage basin. A total of 1 pore volume was passed through the specimen during the testing period (slightly less than one year). Figure 8 shows that the permeability of the specimen continued to decline over time despite contact with the salty pond water. When the test was stopped, the final permeability result was 9.3 x 10^{-10} cm/s. The specimen remained intact with no signs of erosion, indicating minimal-to-no impact by the pond water.

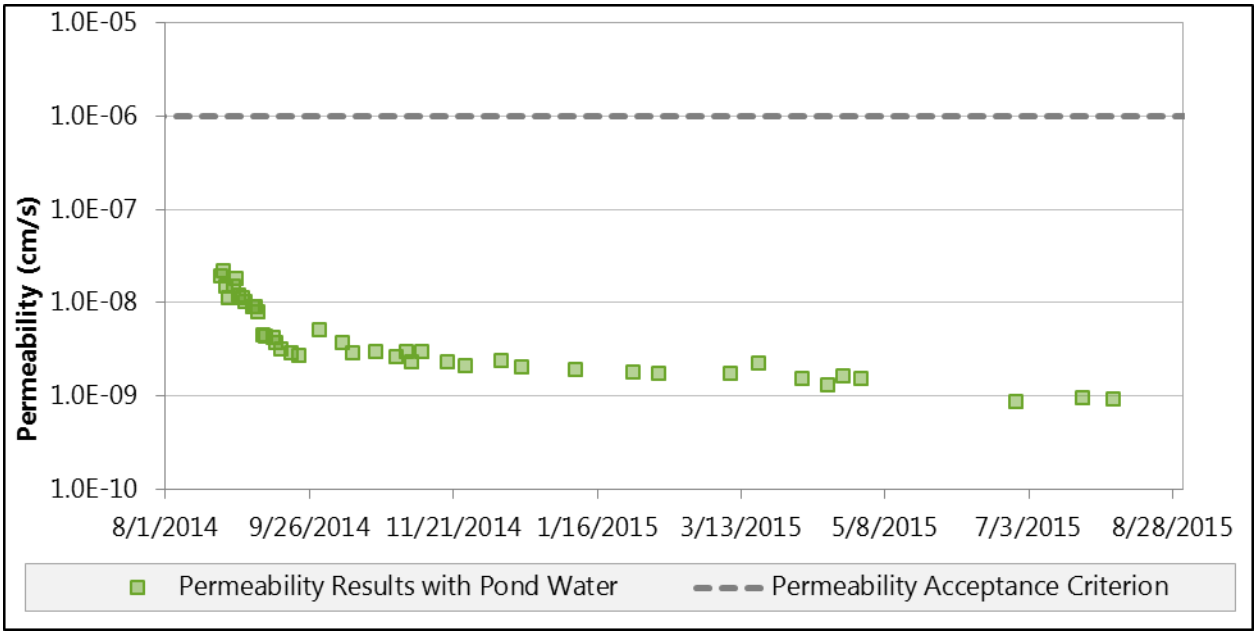


Figure 8. Extended SHS permeability results with pond water

Pore-water pressure response

Performance of the seepage cutoff wall was evaluated by the response of the piezometers installed as part of the instrumentation monitoring program. Figure 9 shows the total head of the piezometers installed at Station 15+00 within the area of most aggressive groundwater flows through the Pond 2 site towards the Green River. Figure 9 includes the total head of the piezometers upstream and downstream of the cutoff wall with sensors in both the overburden and bedrock.

Before wall construction, the head differential across the wall alignment at Station 15+00 was approximately 0.8 feet (0.2 m) in the overburden and 2.7 feet (0.8 m) in the rock. As the wall was built through Station 15+00, the piezometers reacted. After the wall was constructed the new head differentials stabilized at approximately 11.5 feet (3.5 m) in the overburden and 13.6 feet (4.1 m) in the rock. This increase in head differentials between the upstream and downstream VW piezometers is indicative of a successful seepage cutoff wall. The head differentials across the wall are expected to increase further when Pond 2 starts operation and tailings disposal increases the upstream head.

© 2016 Deep Foundations Institute

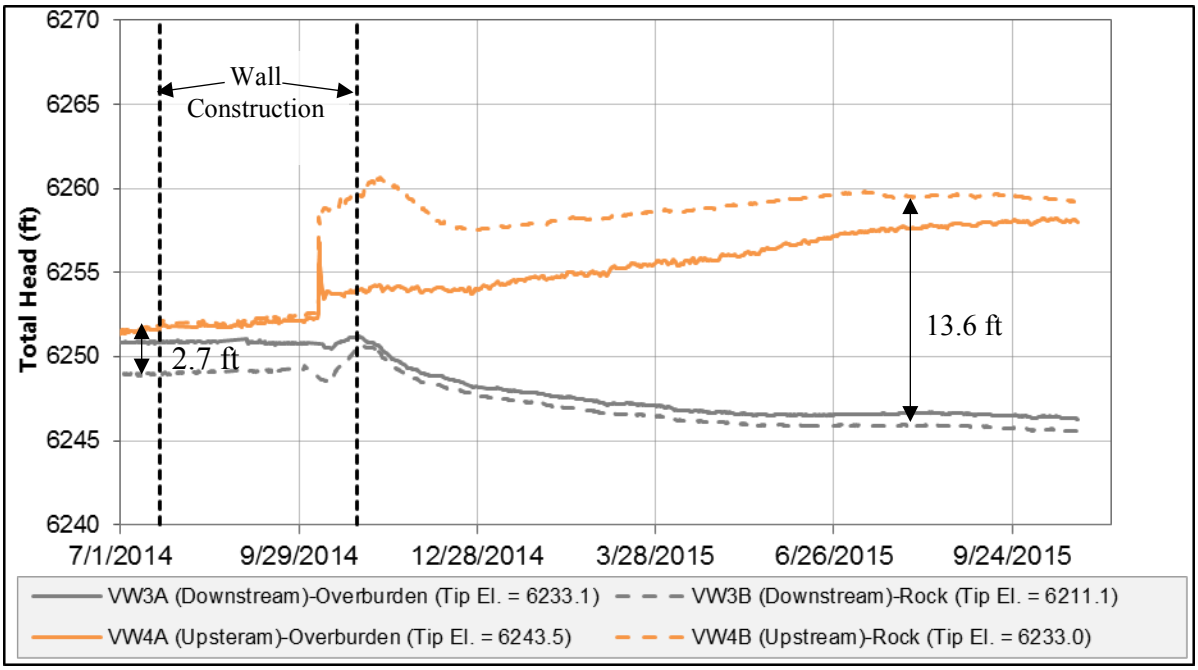


Figure 9. Station 15+00 vibrating wire piezometer reaction to seepage cutoff wall

Similar to the results shown in Figure 9, the head differential across the wall alignment has adapted significantly along the full length of the wall. The north abutment has gone through a unique groundwater flow pattern adjustment and is the exception to the significant increases seen along the rest of the wall length as shown in Table 1.

Table 1. Pond 2 vibrating wire piezometer summary

Vibrating Wire Piezometers (Station of Wall)	Rock Head Differential (feet)	
	Before Wall (May 2014)	After Wall (Oct 2015)
VWs 01 and 02 (Station 5+00)	6.3	3.7
VWs 03 and 04 (Station 15+00)	2.5	13.7
VWs 05 and 06 (Station 25+00)	5.7	16.2
VWs 07 and 08 (Station 35+00)	1.8	5.9
VWs 09 and 10 (Station 45+00)	0.3	5.2
VWs 11 and 12 (Station 55+00)	1.2	8.9

Groundwater elevation contours

Figure 10 shows the groundwater elevations from Pond 2 instrumentation before and after the seepage cutoff wall construction.

Before the wall was installed, the local groundwater flow was directed southwest through the Pond 2 area towards the Green River. Groundwater flow is now blocked by the wall and backing up into the pond. Evidence of the wall can be seen from the stacking of groundwater contours along the location where the wall was constructed. Because the seepage cutoff wall impedes the natural groundwater flow patterns, the pond level has risen approximately 8 feet (2.4 m) even prior to tailings deposition.

© 2016 Deep Foundations Institute

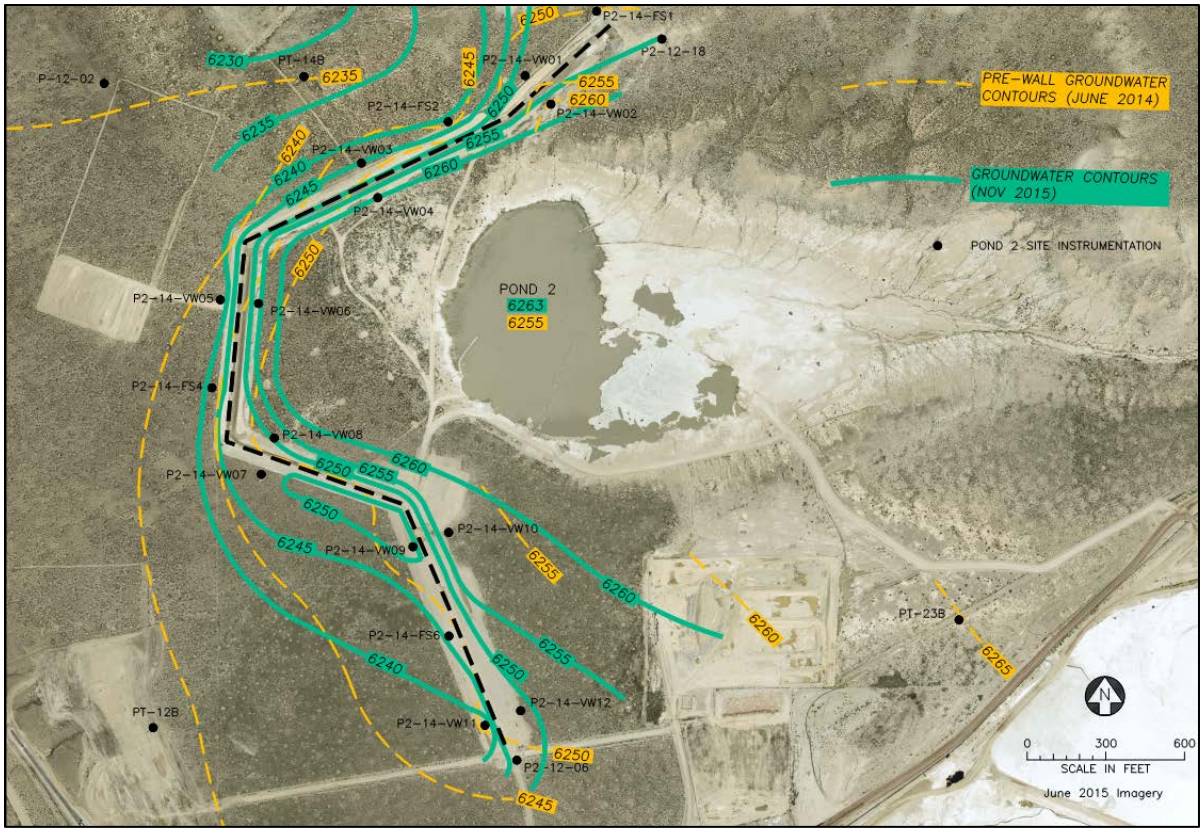


Figure 10. Pond 2 groundwater contours before and after seepage cutoff wall installation

SUMMARY

Ciner Wyoming, in cooperation with Treviicos, Barr Engineering Co., and the project's expert panel, successfully completed seepage cutoff wall construction as part of the Pond 2 development project for tailings storage expansion. Key project takeaways are summarized as follows:

- A collaborative approach among owner, engineer, contractor, and the expert review panel was taken to complete the final design and develop the QA/QC program.
- Pre-production testing included an offset test section which helped eliminate destructive testing of the main wall and provide confidence in the construction methodology and QA/QC program.
- Construction was completed using thorough and consistent QA/QC practices. The QA/QC program results have provided confidence in the current and future performance of the new Pond 2 seepage cutoff wall.
- Instrumentation monitoring is verifying the presence of a seepage cutoff wall in this extreme saline environment.

REFERENCE

Barr Engineering Co., August 2015. Construction Documentation Report: Pond 2 North Seepage Cutoff Wall Construction (Phase 1)

© 2016 Deep Foundations Institute

AFTER SANDY: BAY PARK SEWAGE TREATMENT PLANT FLOOD PROTECTION

Gilbert Tallard, Liquid Earth Support, Inc., McLean, VA, USA, (703) 734-0379, gtallard@liquidearthsupport.com
David Guy, Moretrench, Rockaway, NJ, USA, (973) 627-2100, dguy@moretrench.com
John Stanbury, Moretrench, Rockaway, NJ, USA, (973) 627-2100, jstanbury@moretrench.com

ABSTRACT: In 2009, storm surge from Hurricane Sandy inundated the Bay Park Sewage Treatment Plant located at El. +4 to +12 (+1.2 to +3.6 m) in East Rockaway, Long Island, NY. With all systems down, raw sewage water mixed in the floodwaters. A recurrence of such an event being unacceptable, and in view of rising future sea levels, a flood protection project was developed by Nassau County with funding coming in large part from the federally funded Sandy Rescue Fund. Designed by Arcadis/Hazen & Sawyer Engineers J.V., the protection took the form of a perimeter barrier capable of withstanding future storm surge. Due to limitations imposed by property lines and existing facilities, a levee design with three different types of cross section was developed to adapt to each location, adding construction complexity. The Long Island subsurface is characterized by beach sand extending to great depth. The clay required for the core and levee cover therefore had to be brought in from out of state. A thin silty clay layer at El. -23 to -25 (-7.0 to -7.6 m) served as an aquitard. The under-seepage was reduced by a hanging cut-off wall designed as a vibrated beam thin wall and filled with a salt water resistant, low permeability, self-hardening slurry.

Keywords: barrier, berm, T-wall, I-wall, cut-off wall, clay core, displacement pile, augered cast-in-place pile, jet grout panel, vibrated beam, self-hardening slurry, flood control

INTRODUCTION

On October 29, 2009, Hurricane Sandy hit New York City, with a devastating effect on the region. The low lying Bay Park Sewage Treatment Plant (STP) (Fig. 1) serving Nassau County on Long Island was submerged (Fig. 2). All basements and underground mechanical facilities were flooded (Fig. 3) and millions gallons of effluent were released into the waters of the nearby Hewlett Bay which connects via other small waterways to the Atlantic Ocean.



Fig. 1. Aerial view of Bay Park Sewage Treatment Plant.

© 2016 Deep Foundations Institute



Fig. 2. Bay Park STP main gate, the morning after the hurricane (courtesy Hazen and Sawyer).



Fig. 3. All systems below grade were submerged and inoperable (courtesy Hazen and Sawyer).

© 2016 Deep Foundations Institute

FEMA (Federal Emergency Management Agency) funding was allocated for repair and restoration of areas affected by the Hurricane Sandy storm surge (FEMA 4085-RD-NY Final Environmental Assessment, 2014). Given the widespread devastation, demand for funding was high. Nassau County therefore moved quickly to secure adequate allocation for restoration of the Bay Park STP, including construction of perimeter flood protection against future events, and also for restoration of surrounding public recreation areas, which was a high priority with local residents. The design of the flood wall defenses was given to a joint venture of Arcadis/ Hazen & Sawyer Engineers. With the plant required to remain in operation during the work, the concept options were limited and a perimeter barrier with three types of cross section was selected as the most practical and cost effective solution.

DESIGN CONSIDERATIONS

The goal was to provide a flood control system that would be 100% effective on short notice while not impeding plant operations under normal conditions. The system had to allow normal ingress and egress of truck traffic, requiring the installation of flood gates at the facility's north and south entrances. The ground elevation varied from +4 to +12 (1.2 to 3.7 m). The height of the barrier was calculated based on a protection flood level of 17 ft (5.2 m) NAVD (North American Vertical Datum). This included 1ft (0.3 m) sea level rise in the next 50 years and 2 ft (0.6 m) of free board.

In this part of the State, storm and sanitary sewers are separated, meaning that storm water is conveyed to the outfall through a pumping station within the Bay Park STP without treatment. This underground portion of the pumping station was flooded during the Sandy event and had to be designed with new parameters and built anew.

The flood control barrier was primarily designed as a cost-effective earthen levee (Fig. 4). Since Long Island soils are predominantly sandy, these soils could be utilized for the body of the levee, with a protective clay blanket added that could be vegetated with grass on both slopes for visual aesthetics.

Under-seepage was addressed by inclusion of a clay core at the center of the levee, extended by a vibrated beam slurry cut-off wall installed to sufficient depth to create a flow net under flood conditions where velocities are well below the internal erosion threshold. Where sufficient open space for the earthen levee was not available, the overall design incorporated a pile-supported T-wall connected to the cut-off wall by a relatively short HDPE membrane embedded on the cut-off wall and in the T-wall concrete footing to guarantee continuity between the two. The designed piles were concrete displacement piles that enhance their bearing capacity by compacting the surrounding soil (Fig. 5).

Where sufficient open space was available but would cause the berm to slope too steeply, the engineer elected to use an I-wall section (Fig. 6), which was designed as a smaller earthen berm with a steel sheet pile/concrete cap installed atop the berm to achieve the design elevation. For the I-wall sections, the earthen berm was constructed to El. +14. Steel sheet piles were then driven into the completed berm leaving 3 ft (1 m) of the sheet pile exposed to achieve the design elevation of +17 ft (+5.2 m). The steel sheet piles were encased in concrete, with the outward facing side having a decorative design for aesthetic purposes. Figure 7 shows the various locations of the different sections.

© 2016 Deep Foundations Institute

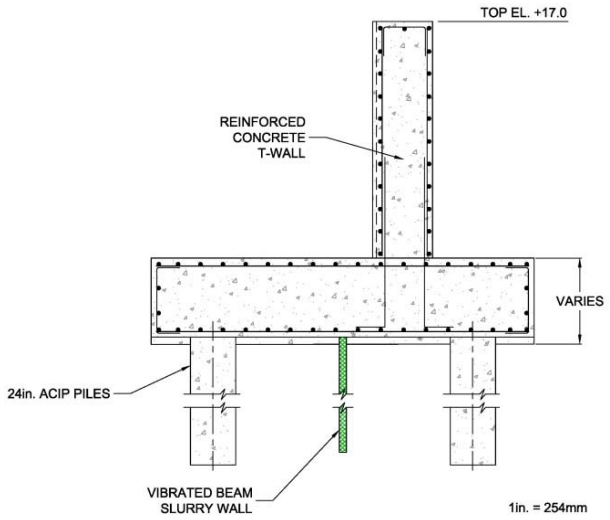


Fig. 4. T-wall section

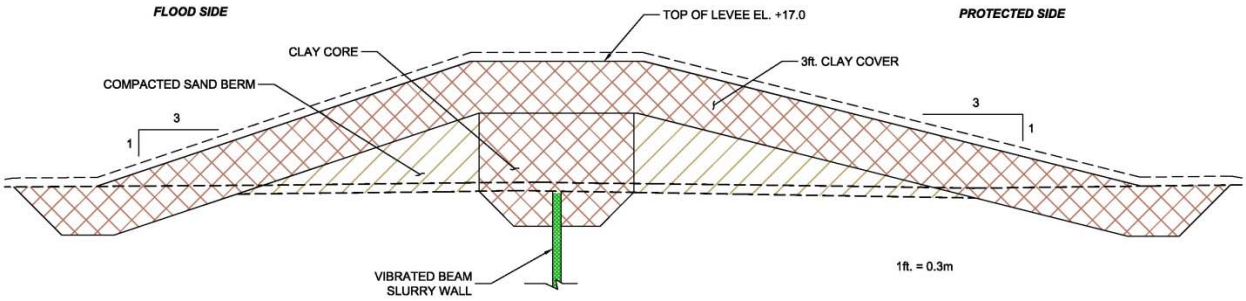


Fig. 5. Typical levee section

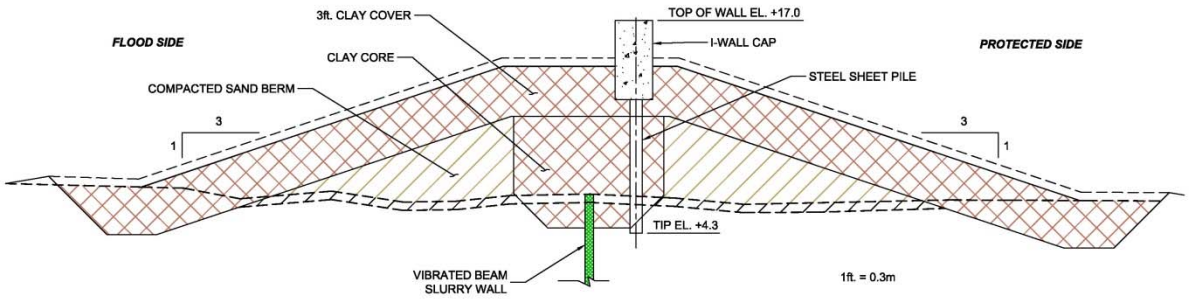


Fig. 6 Typical I-wall section.

© 2016 Deep Foundations Institute



Fig. 7. Location of perimeter flood barrier construction techniques

Given the property lines limitations, no enlargement of the plant footprint can be considered for future plant enhancement. The Bay Park STP therefore has a limited growth potential but is serving a fairly stable community, mostly residential and small industrial and commercial businesses. Future STP more restrictive regulations may force the plant to build down, always problematic with high ground water tables.

Self-hardening Slurry

Self-hardening slurry was selected for the vibrated beam cut-off wall. The term "self-hardening" is increasingly replacing the more common "cement-bentonite" term as representing a broader array of possible clays and cements used in a variety of cut-off wall construction techniques. In the case of a thin wall, as created here by the vibrated beam method, a lower than usual hydraulic conductivity coefficient by one or two orders of magnitude (10^{-8} cm/sec.) is achieved, thus providing the equivalent of a thicker conventional trench wall.

Aside from permeability, strength represents resistance to high gradients and internal erosion determined by pin hole testing. In that respect, a higher strength than provided by a conventional cement-bentonite is often desirable. On this project, the ultimate unconfined compressive strength was on the order of 200 psi (1.38 MPa).

On this particular project, the potential exposure to marine salt water called for ingredients compatible with sea water. Both attapulgite clay and ground blast furnace slag cement, the basis of the self-hardening slurry formulation adopted for this thin cut-off wall, met these requirements.

The long setting time of slag cement is favorable for the formation of continuous cut-off walls without vertical cold joints. Actual joints did occur on this project at changes of direction and re-connections between phasing of the work; however, in contrast to concrete, high water content self-hardening slurries have no shrinkage during curing and chemical bonding at the contacts prevents the formation of an actual cold joint.

© 2016 Deep Foundations Institute

CONSTRUCTABILITY CONSIDERATIONS

Available space was the main determinant in selecting the most appropriate construction methodology. The berm being the most cost-effective, this method was applied whenever possible.

The piles for the T-wall foundation were initially designed as drilled displacement piles. However, if sequencing called for installation of the cut-off wall before that of the piles, displacement pile construction in proximity of a vibrated beam thin wall might cause damage to the latter. The piles were therefore re-designed as augered cast-in-place (ACIP) piles with a larger diameter of 24 in. (0.6 m) to achieve the same skin friction bearing capacity. Constructing the vibrated beam cut-off wall between the rows of auger cast piles required a higher level of care along the T-wall alignment.

Self-hardening slurry cut-off wall construction is not recommended during very cold weather. As a result of a hard 2014-15 winter, and delays in securing alignments related to utilities work, the installation of the cut-off wall was interrupted for a period of 5 months.

Due to phasing issues and change orders, both the ACIP pile work and cut-off wall work were accomplished between the Fall of 2014 and Fall of 2015. Mixing of the self-hardening slurry required a large demand for constant water flow. Since water came from the plant fire hydrants, pressure varied drastically and could not service the slurry mixing plant adequately. The geotechnical contractor therefore elected to install a water line from a municipal-fed hydrant over 2,000 ft (610 m) away from both mixing locations. This allowed the crew to mix slurry using bulk ingredients in a semi-automatic plant in a timely fashion as well as allowing an increase in cut-off wall production.

FOUNDATIONS & SUBSURFACE WORK

A specialty geotechnical contractor was awarded a subcontract to perform the augered cast-in-place (ACIP) pile foundation work as well as the vibrated beam cut-off wall and also areas where jet grouted connection were required.

Piling

In areas where an earthen berm could not be constructed, a T-Wall section was constructed consisting of a double row of 24-in. (0.6 m) ACIP piles as the foundation elements which supported the above ground reinforced concrete structure. The ACIP piles were installed using a typical geotechnical pile rig. Each pile was reinforced with either #8 or #11 rebar cages. In total, 699 ACIP piles were installed to varying tip elevations and constructed using over 3,000 yd^3 (2294 m^3)of pile grout. The geotechnical contractor successfully conducted a compression load test for both a 90-ton (81,645 kg) and 70-ton 63,500 kg) design load.

Cut-off Wall

A vibrated beam thin cut-off wall installed around the entire site perimeter was selected by the project design team to mitigate the possibility of under-seepage in the event of future flooding situations. The advantages of vibrated beam construction included the simplicity of the process, the very limited impact on the site; the small footprint of the mixing plant; the ability to pass through the forest of auger cast piles under the T-wall; the engineered self-hardening slurry compatible for use in the brackish conditions ensuring long term performance; and ultimately the very competitive pricing in a NYC environment.

© 2016 Deep Foundations Institute

The vibrated beam technique does require significant quality control. A 17-in. (430-mm) fin was installed on the far face of the beam to act as a guide to ensure that each penetration was continuously connected with the section previously created (Fig. 8). Verticality and stationing were obviously the paramount controls.



Fig. 8. Vibrated beam fin and shoe

The self-hardening slurry is a one-step mixture of attapulgite clay and slag cement, exclusively. The water/cement ratio was slightly above 8. The concept of hydration is foreign to this technique since the viscosity is achieved mechanically by shearing the attapulgite clay with the slag cement. The slurry is stable at a Marsh Funnel viscosity of 36 seconds. The hardened slurry is a non-shrink grout and shrinkage cracking does not occur in the vadose zone. Superficial cracking of the unprotected gelled slurry, can occur, much as in the cases of cement-bentonite and dried lake beds ("tetragonal boden").

The geotechnical contractor elected to use an ABI pile rig to vibrate a large, built-up W33x152 beam section into the ground to create the cut-off wall. The shoe was 5 in. (125 mm) thick, defining the width of the wall by displacing and compacting the ground laterally. During driving and extraction of the beam section, a self-hardening slurry was pumped through the shoe to fill the void created, thus creating the cut-off wall. A shallow wider trench on the surface acts as a slurry reservoir that guarantees the constant saturation of the successively created slots and thus constructing the continuous thin cut-off wall.

The wall was installed to a tip elevation of -25 (-7.6 m), with ground surface/levee height being anywhere from elevation +6 to +15 (+1.8 to +4.6 m). In the areas where the T-Wall section was to be installed, the cut-off wall was driven in between the row of ACIP piles. A short HDPE barrier membrane was embedded into the top of the wall to make a connection to the reinforced concrete section, functioning as a water stop. Over 6,700 lf (2,042 m) of cut-off wall was installed for the project for a total vertical area over 218,500 ft^2 (20,300 m^2).

Jet Grouted Connections

Given the presence of large underground utilities, a few areas had to be addressed where the cut-off wall could not be constructed. Once the entire cut-off wall was installed, jet grouting was used to close the gaps beneath each underground utility. The geotechnical contractor installed a series of jet grout panels that intersected the previously installed cut-off wall to achieve the closure. The same self-hardening slurry as for the vibrated beam cut-off wall was used for the jet grout panels.

© 2016 Deep Foundations Institute

CHANGES FROM THE ORIGINAL DESIGN

In addition to the previously mentioned substitution of the displacement piles with the augered cast-in-place piles, material availability issues drove a change to the vibrated beam cut-off installation procedure.

The original design proposed was to construct the vibrated beam slurry wall from existing ground surface and build the earthen berm on top of the slurry wall, with the clay core acting as the wall element within the berm. Due to the lack of availability of clay material on Long Island, clay would have to be shipped over great distances to supply the job, prompting design efforts to be focused on reducing the amount of clay used on the project. It was ultimately decided that the geotechnical contractor would be able to construct the slurry wall from atop the partially completed sand berm as a value engineering adjustment to the original design. Since the slurry wall was constructed at a higher elevation, the requirement for a such a large clay core within the berm was reduced. Working platform elevations varied from El. +9 to El. +14.5 (2.75 to 4.42 m).

CONCLUSIONS

In view of the significant impact of the Hurricane Sandy storm surge event on the Bay Park STP itself and to the waters of the nearby Hewlett Bay, adequate flood protection against a future occurrence was considered essential and funding was made available through FEMA. Given space limitations along parts of the plant boundaries, the base earthen levee design was supplemented by T-wall sections. Seepage control beneath the levee was addressed by a cut-off wall. A view of part of the completed flood protection is shown in Fig. 9. Changes to the original concept were instigated to overcome installation and material availability issues. All field testing of materials passed the specified project requirements. The project demonstrates good use of a little known slurry wall technique to provide the best results for the most cost effective price. While the berm height was designed based on the calculated NAVD protection flood level, and with a projected rise in seas levels over the next 50 years taken into consideration, the only real test in place will be another major storm surge, an event that is unfortunately all too likely.



Fig. 9. View of the completed levee portion (Courtesy Hazen And Sawyer).

© 2016 Deep Foundations Institute

ACKNOWLEDGMENTS

The authors extend their thanks and appreciation to Hazen and Sawyer for the use of their images in the preparation of this paper.

REFERENCES

FEMA 4085 –RD-NY Final Environmental Assessment, Bay Park STP Hazard mitigation: Flood wall and berm construction, East Rockaway, Nassau County, New York, 2014.

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

REMEDIAL DRILLING AND GROUTING AT ABIQUIU DAM, NM AND FLUID LOSSES IN THE EMBANKMENT, A CASE HISTORY

Amy F. Ebnet, US Army Corps of Engineers, Seattle District, Seattle, WA, USA (206) 764-7209, Amy.F.Ebnet@usace.army.mil
Suzanne R. Hess-Brittelle, US Army Corps of Engineers Risk Management Center, Denver, CO, USA, (303) 963-4549, Suzanne.R.Hess-Brittelle@usace.army.mil
Tyler J. Quick, US Army Corps of Engineers, Seattle District, Seattle, WA, USA, (206) 764-6168, Tyler.J.Quick@usace.army.mil
Sharon L. Schulz, US Army Corps of Engineers, Portland District, Portland, OR, USA. (503) 808-4890, Sharon.L.Schulz@usace.army.mil

ABSTRACT

Abiquiu Dam is a rolled earth fill dam with impermeable core located on the Rio Chama in New Mexico. Abiquiu has had a history of excessive seepage through the fractured sandstone abutments since water was first impounded in 1963. The foundation was treated with a single row grout curtain in the cutoff trench prior to placement of the embankment. During the 1960's and 1970's, the grout curtain was extended 1,000 ft. into both abutments and sections of the foundation beneath the embankment were re-grouted. Grouting of the foundation beneath the embankment required drilling through the embankment. Drilling fluid loss or loss of circulation within the impervious core was observed during multiple phases of supplementary grouting and during verification drilling following the 1970's grouting program. It is likely that the drilling methods employed for these grouting and drilling programs caused hydraulic fracturing of the impervious core. The fluid losses in the embankment may be related to seasonal shut-down layers in the embankment or benches in the abutment foundations. In addition, it is likely that the impervious core was fractured from grout pressures during installation of grout pipe through the embankment.

Keywords: Abiquiu, grout, drilling, embankment, dam, hydraulic fracture.

INTRODUCTION

Abiquiu Dam is located on the Rio Chama in New Mexico and consists of a rolled earth fill dam with impermeable core, controlled outlet works, and an uncontrolled rock-cut spillway. The dam was authorized for flood control and sediment retention and was designed as a dry dam, holding flood pools for relatively short durations. The foundation is comprised of Permian and Triassic age, interbedded mudstones and sandstones that were deposited in an active braided stream system. Sandstones are moderately fractured and have variable aperture and infilling. Generally speaking, joints are near vertical. Abiquiu has had a history of excessive seepage through the fractured sandstone abutments since water was first impounded in 1963. The foundation was treated with a single row grout curtain in the cutoff trench prior to placement of the embankment. In 1966-1968, the grout curtain was extended 500-ft into the left abutment and the foundation beneath a 560 ft. section of the embankment was re-grouted. In 1978-1980, the grout curtain was extended 1,000 ft. into the right abutment, an additional 500 ft into the left abutment and the foundation beneath the right 520-ft section of the embankment was re-grouted. The alignments of these grout programs are shown in Figure 1. Grouting of the foundation beneath the embankment required drilling through the embankment. Drilling fluid loss or loss of circulation within the impervious core was observed during both phases of supplementary grouting and during verification drilling following the 1978-80 grouting program. It is likely that the drilling methods employed for these grouting and drilling programs caused hydraulic fracturing of the impervious core. In addition, it is likely

© 2016 Deep Foundations Institute

that the impervious core was fractured from grout pressures during installation of grout pipe through the embankment. This paper provides a summary of the drilling and grouting methods used during these programs and the potential for damage to the embankment.

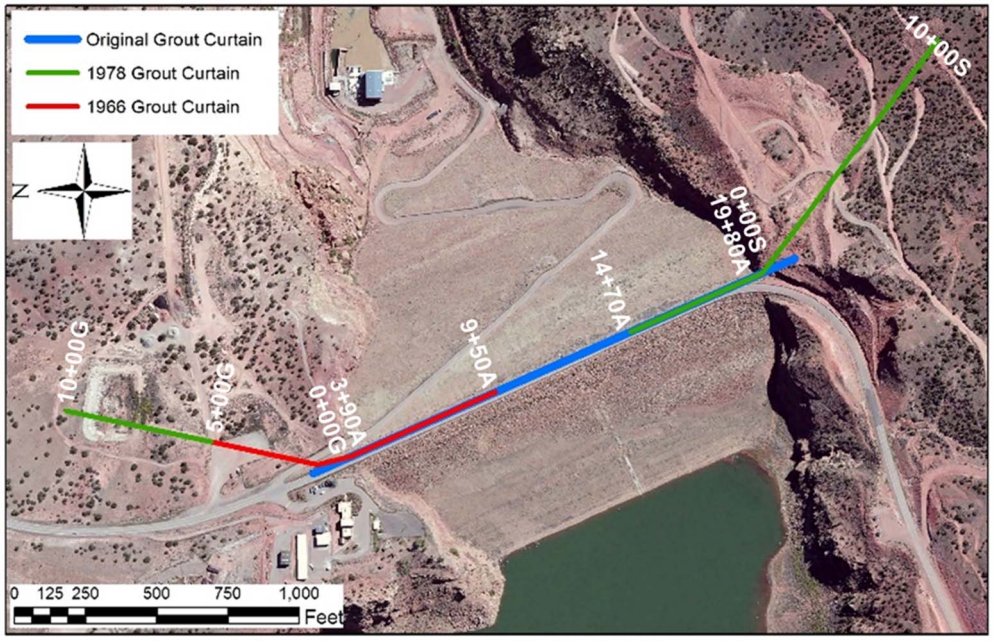


Figure 1: Alignments for Original Grout Curtain and 1966 and 1978 Grout Curtain Extensions.

ORIGINAL GROUT CURTAIN

A single line grout curtain was placed along the axis of the embankment from the base of the cutoff trench following completion of preliminary excavation to approximate grade (USACE Albuquerque District, 1963). Figure 1 shows the extent of the original grout curtain in plan view and Figure 2 shows the profile. Grout holes in the streambed were split-spaced to 10 to 20 ft. apart and abutment holes were 5 to 10 ft. apart. The largest grout takes were in the upper sandstone units in the abutments.

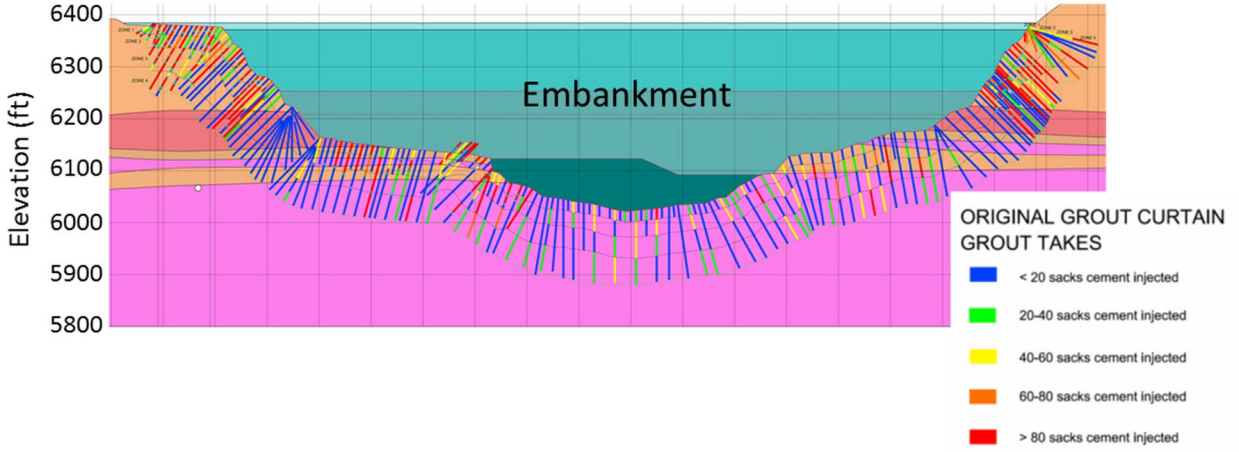


Figure 2: Profile and grout takes along original grout curtain.

© 2016 Deep Foundations Institute

1966 SUPPLEMENTAL DRILLING AND GROUTING

Minor pools (< elevation 6100 ft. NAVD 88) were impounded as early as April 1963 to improve sediment trapping. Intermittent storage for flood control began in the spring of 1965, causing excessive seepage. Seepage was observed through both left and right abutments, into the embankment drainage blanket and to exit faces in the canyon walls downstream of the dam. Subsequent pool storage to elevation 6,189.3 ft (NAVD 88) in the fall of 1965 caused a marked increase in seepage volumes through the abutments (USACE, Waterways Experiment Station, 1984). As a result of the observed seepage, supplemental drilling and grouting were conducted during 1966. The 1966 supplemental drilling and grouting program included a 1,000-foot grout curtain extension on each abutment. By several contract modifications, the work was reduced to a 500-foot grout curtain extension in the left abutment and a 520-foot section of foundation was re-grouted beneath the dam (USACE, Albuquerque District, 1967).

1966 Left Abutment Grout Curtain Extension

One hundred and five holes were drilled vertically and grouted along the curtain on the left abutment extending the curtain 500 feet into the abutment (Figure 1). Thirty-one primary grout holes were drilled on 20-foot centers under the original contract. Due to serious caving and squeezing off of the holes in the mudstone zones, it was not possible to place grout in these zones and a number of holes were lost due to stuck rods. Only three of the original 31 holes could be completed to the required depth (USACE, Albuquerque District, 1967). The method of drilling grout holes was modified to require the use of perforated two-inch casing through the mudstone to overcome the caving problem. Thirty-one secondary holes were drilled on 20-foot centers, split-spaced between the primary holes drilled under the original series. Large grout takes in the upper sandstone indicated the need for more extensive grouting, therefore 43 tertiary holes were drilled reducing the spacing between the grout holes to five feet along most of the curtain (USACE, Albuquerque District, 1967).

1966 Regrouting Beneath the Embankment Near Left Abutment

Three holes were drilled and grouted on the axis of the dam to evaluate the effectiveness of the original grout curtain (USACE, Albuquerque District, 1967). Due to large grout takes in these holes and the large grout takes at elevation 6,000 feet, the grout curtain was extended and deepened along the axis of the dam along a 520 ft. section from Sta. 4+00A to 9+50A (Figure 1). Twenty-seven primary holes were drilled and grouted on 20-foot centers from Sta. 4+30A to Sta. 9+50A and 11 secondary holes were drilled and grouted from Sta. 4+40A to Sta. 6+40A through the embankment and into the foundation.

Holes were drilled through the impervious fill using mud rotary methods to a minimum depth of 15 feet into the foundation rock. Holes were advanced using drilling mud (‘Aquagel’) and an inner pipe was set. The pipe was specified to be cleaned of drilling mud and cuttings prior to grouting. The pipe was then grouted in place. Grouting of the pipe was specified to be either placed using a packer at the bottom of the pipe to fill the annular space between the pipe and the impervious fill or through the pipe to fill both the annular space and the pipe. In either case, the pipe was to be cleaned of grout by washing or some other means before it had set sufficiently to require re-drilling (USACE Albuquerque District, 1966). The gage pressure used to grout 2-inch or 4-inch pipe into place through the fill was generally 20 psi (the gage pressure in grout hole 6+10A was 40 psi). Once the grout around the pipe had set, smaller diameter holes were drilled into the foundation and grouted using the stage and zone method.

© 2016 Deep Foundations Institute

Grout mixes varied from a water-cement ratio of 3:1 to 0.8:1. Grouting gage pressures were not allowed to exceed 0.9 to 1.25 psi per foot of cover and did not exceed a maximum gage pressure of 150 psi. A total of 50,659.5 cubic feet (378,959 gallons) of cement was pumped into 49,486 linear feet of grout holes for an average of 1.02 cubic feet per foot drilled.

RIGHT AND LEFT ABUTMENT GROUTING (1978-1980)

From late 1978 until late 1980, two additional supplemental grouting programs were completed. Under the first contract, 510 feet of the embankment curtain was re-grouted from Sta. 14+70A to Sta. 19+80A. The curtain was then continued up the right abutment 500 feet. Under the second contract, the 1966 increment was extended 500 feet on the left abutment, and 500 feet of curtain were extended on the right abutment. Figure 1 shows the location of the 1978-1980 grout curtain extension.

All drilling was performed using rotary drilling equipment (USACE, Waterways Experiment Station, 1984). Drilling fluid was specified to be clean water with the exception that drilling mud may be used to drill through embankment fill. The specification also required if circulation was lost in the embankment fill, drilling would be stopped and the drilling fluid viscosity adjusted until recirculation was regained (USACE, Albuquerque District, 1978). Three-inch diameter drill holes from Sta. 14+80A to 18+60A were drilled using water and 5-inch diameter drill holes from Sta. 18+60A to 19+80A were drilled using Baroid Drill Foam (USACE, Waterways Experiment Station, 1984). Holes were drilled to a minimum depth of 10 feet into the sandstone then grout pipe was installed and grouted into place. Gage pressures used for grouting in the casing were generally 20 psi. However, some holes in the embankment had reported gage pressures of up to 100 psi. Once the grout around the pipe had set, smaller diameter holes were drilled into the foundation and grouted using the stage and zone method.

The total linear footage drilled on these two contracts was 163,000 feet into which 318,000 cubic feet of grout (116,000 sacks of cement) were pumped for an average of approximately 2 cubic feet per foot of hole (~0.71 sacks of cement per foot).

In October through November 1980, four verification holes were drilled along the downstream side of the dam crest at Sta. 14+70A, Sta. 14+90A, Sta. 14+93A and Sta. 18+50A. The purpose of the exploration drilling was to determine whether there is grout from placement of the grout curtain in the embankment and/or embankment-foundation contact. No grout was found in the embankment, or the embankment-foundation contact. Grout was found in the sandstone foundation in the drill hole at Sta. 14+70A (USACE, Albuquerque District, 1981).

LOSS OF DRILLING FLUID & HIGH GROUT TAKES WHILE DRILLING THROUGH THE IMPERVIOUS CORE

During drilling operations through the impervious core for the 1966 supplemental grout program, some losses of drilling mud (‘Aquagel’) were reported. These fluid losses occurred as the drill holes were being advanced through the impervious core. Locations of fluid losses are shown in Figure 3 and Table 1 and summarized below.

© 2016 Deep Foundations Institute

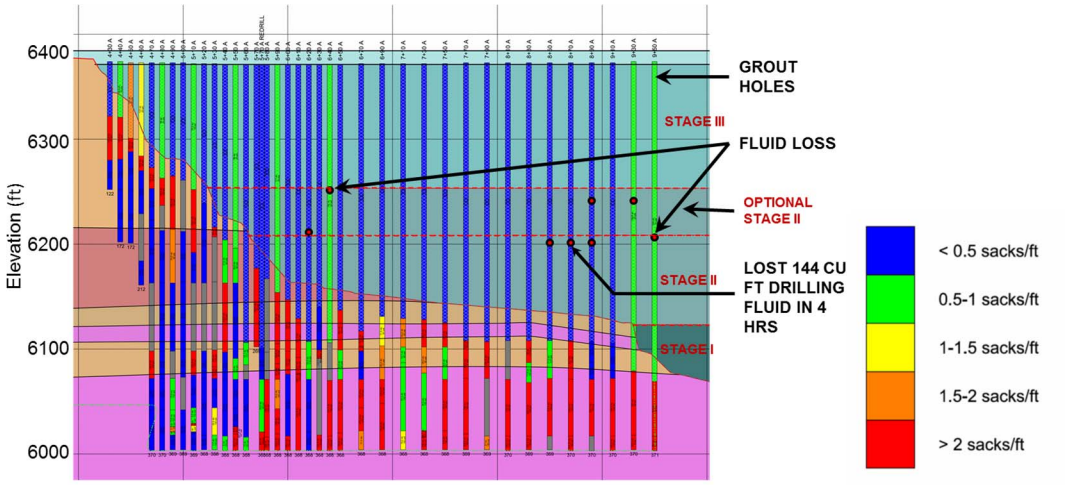


Figure 3. Grout hole locations and areas of drilling fluid loss in the embankment between Sta 4+00A and 9+50.

Table 1. Grout holes where drilling fluid was lost into the embankment (USACE, Albuquerque District, 1967)

Hole at Station	Depth (ft)	Elevation (ft)
6+20A	160	6211.3
6+40A	120	6251.3
8+50A	170	6201.3
8+70A[1]	170	6201.3
8+90A	130	6241.3
8+90A	170	6201.3
9+30A	130	6241.3
9+50A	165	6206.3

[1]~144 ft^3 of drilling fluid lost over ~4 hrs. in 8+70A

When loss of circulation occurred, further advancement was stopped and the drilling mud thickened to attempt to regain circulation (USACE, Albuquerque District, 1967). Circulation was apparently regained on all holes except those at Sta. 8+50A and 8+70A. The field records on the drilling were inadequate so the drillers' recollection was relied upon for details of the fluid losses. It was estimated that about 144 cubic feet (~1,077 gallons) of drilling mud was lost in about four hours in an attempt to regain circulation at a depth of 170 feet (elevation 6201 feet) in the hole at Sta. 8+70A. Failing to regain circulation, the hole was drilled to a depth of about 265 feet, about 25 feet below top of rock. There are no records available of how much drilling fluid was lost in the remaining holes.

During drilling through the embankment for the 1978 supplemental grout program, lost circulation was noted on driller's logs for the grout holes at Sta. 14+70A, Sta. 15+10A, Sta. 15+40A and Sta. 18+40A. Additionally, intercommunication was noted between holes 14+88.75A and 14+86.25A in the embankment. The locations of lost circulation are shown in Figure 4 and summarized in Table 2. No other information about the loss of circulation in these holes besides the driller's logs has been found. In addition, drilling fluids were lost in the embankment in the verification holes at Sta. 14+70A and 14+93A (Figure 4 and Table 2).

© 2016 Deep Foundations Institute

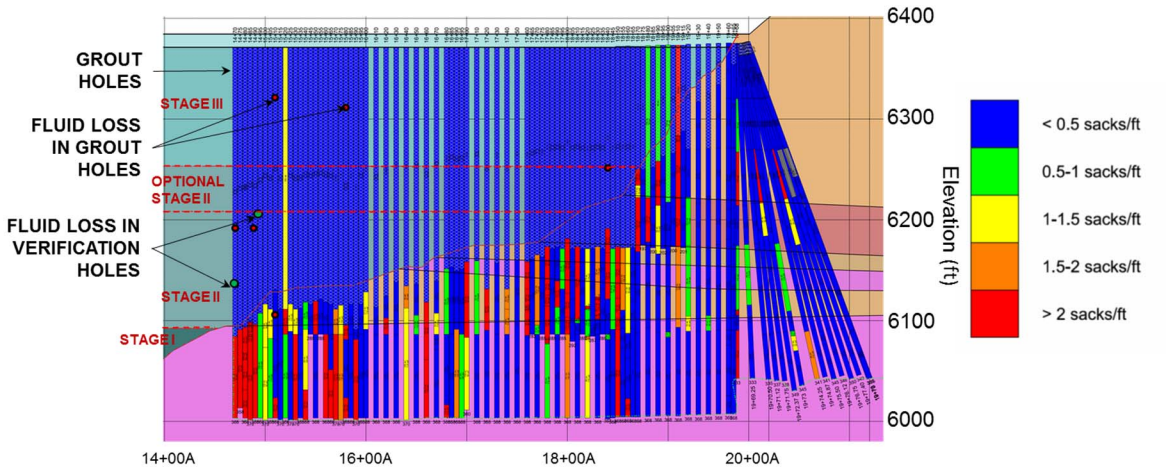


Figure 4. Locations of fluid loss in embankment during drilling for 1978-1980 grout curtain and post-grouting verification holes. Note: Primary through Tertiary grout holes are shown.

Table 2. Summary of noted lost circulation in the embankment during 1978-1980 grout hole drilling

Hole ID	Depth (ft.)	Note
14+70A	170	Lost circulation.
14+70A	160 to 200	Circulation Problems for six hours. Mixed mud and cotton seed hulls.
14+88.75A	170	Intercommunication with 14+86.25A.
15+10A	50?	Drilled 50 feet with foam. Lost circulation.
15+10A	267	Drilled hole to 267 feet. Cased hole to 265 feet. Used 2 sacks of cotton seed hulls, 2 quick seal and 4 super gel. *Note top of rock is approximately 242 feet.*
15+80A	60	Lost circulation. Shut down.
18+40A	120	Lost circulation at 120 feet but managed to drill anyway.
V14-70A	235	Lost 1,000 gallons of water.
V14+93A	159 to 180	Lost 1,000 gallons of water-revert.

Note: V14+70A and V14+93A are verification holes

Evaluation of Fluid Losses

It is probable that the drilling methods employed caused hydraulic fracturing of the impervious core. The core is in contact downstream with granular fill used for the chimney drain. The impervious core only needs to hydraulically fracture along the shortest path to the chimney drain for substantial fluid loss to occur. It is likely that hydraulic fractures did not go all the way through the core upstream to downstream, but rather exited into pervious fill in the chimney drain. The chimney drain, therefore, may have been contaminated by drilling fluid and grout. Static drilling fluid pressure may not have been sufficient to overcome the internal stresses of the core material. Surging drilling fluid pressures and caving boreholes may have been necessary to initiate fracturing. (Michael, Bailey, personal communication July 13, 2015).

Five of the fluid losses occurred near elevation 6200 ft. and three occurred near elevation 6250 ft. (Figure 5). The construction drawings show optional limits for Stage II fill at both of these elevations. These limits are shown on Figure 3 and Figure 5. Some of the fluid losses may be related to a construction defect in the embankment due to a seasonal shutdown. No records have been found for construction of the embankment, so the actual limits of the embankment stages are uncertain.

© 2016 Deep Foundations Institute

The fluid losses at Sta. 6+20A and Sta. 6+40A are located above an abrupt change in foundation slope (Figure 3). This slope is approximately 45 to 90 feet below the zone where losses were reported. From Sta. 8+50A to Sta. 9+50A, fluid losses were reported in a zone from elevation 6201.3 to 6206.3 feet and at elevation 6241.3 feet (Figure 3 and Table 1). The fluid losses in these holes are above a steep 25-ft high bench in the underlying foundation. The bench is approximately 75 to 115 feet below the zones where the losses were reported. The fluid losses between Sta. 14+70A and Sta. 15+10A are above a steep bench in the right abutment (Figure 4). These abrupt changes in foundation slope geometry may have associated low confining stress zones that may be more vulnerable to hydrofracture from the weight of the drilling mud.

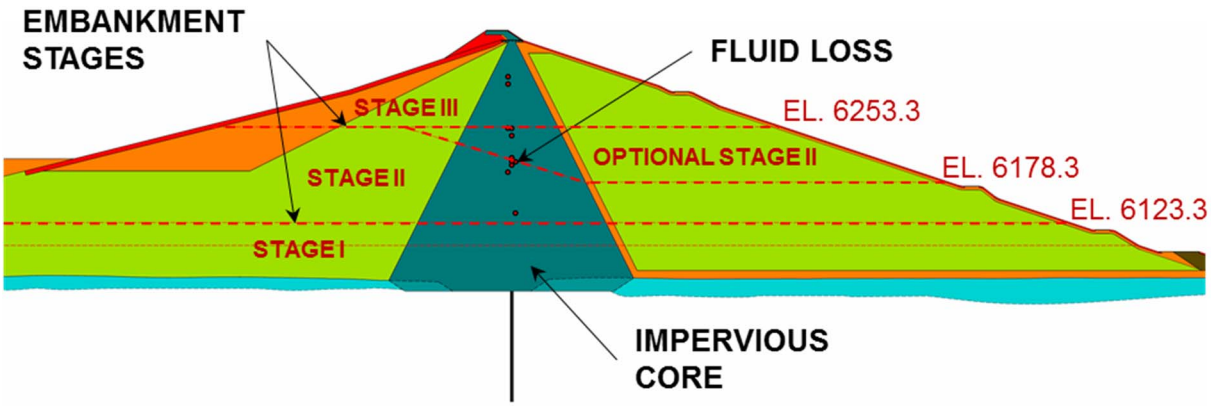


Figure 5: Cross section through embankment showing projected locations of fluid losses from the supplemental grouting programs in relation to construction staging.

The applied grouting pressures may have also hydrofractured the impervious core during installation of the grout pipe. A Sigma /w analysis was performed to estimate the internal stresses from the weight of soil to compare with estimated grout pressures to determine if there was potential for hydrofracture during grouting operations. Modeling results show that the minimum effective stresses within the embankment would likely be significantly less than the effective pressures exerted by the grout column in some areas (Figure 6), which suggests hydrofracture was possible during grouting operations. Modeling also confirmed the presence of low-stress zones along some of the steep abutment sections such as in areas near Station 9+30 and 14+70. Localized, low-stress zones would be more likely to hydrofracture due to smaller confining stresses. These low-stress zones correspond with several of the noted areas of drilling and fluid loss, consistent with expectations in areas of lower stress. Although estimated stress values from modeling are not precise due to modeling limitations, they do indicate trends in the internal stresses that suggest hydrofracture was possible during drilling and grouting operations.

In general, the highest grout takes through the embankment zone were in the grout holes near the areas of drilling fluid loss. The estimated volume of grout used to install the grout pipe was compared with the estimated volume of the grout hole for the holes that experienced fluid loss in Table 3. The volumes of grout injected were significantly larger and the volume of the grout holes. However, the grout holes were drilled at least 10 feet into bedrock before installing and grouting the pipe. Therefore a portion of the grout was likely injected into the fractured rock.

© 2016 Deep Foundations Institute

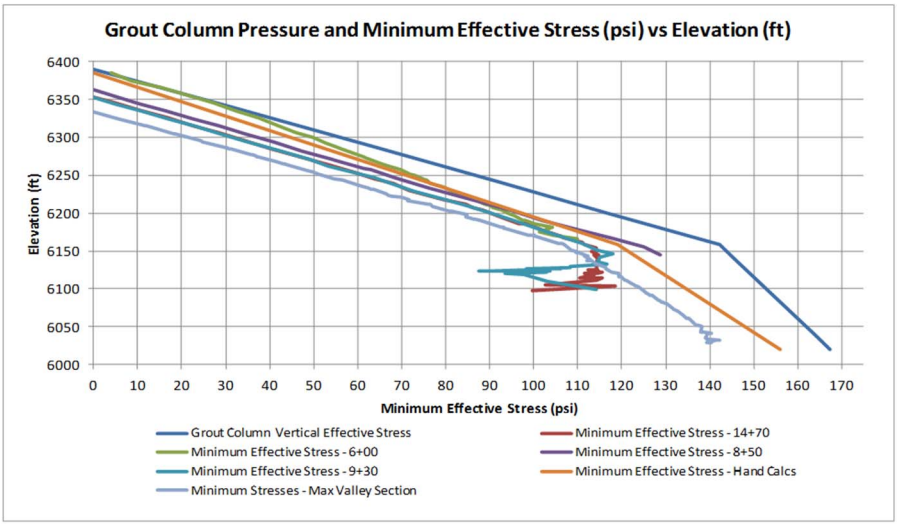


Figure 6: Grout column pressures and minimum effective stress (psi) by elevation for the longitudinal profile model.

Table 3. Comparison of volume of grout placed and volume of drill hole

Hole at Station	Estimated Volume of Drill Hole (ft^3)[1]	Sacks of Cement Placed	Estimated Volume of Grout Placed (ft^3)[2]
6+20A	13.25	24	84
6+40A	13.25	182	413
8+50A	13.25	65	211
8+70A[1]	13.25	78	243
8+90A	13.25	132	345
8+90A	13.25	132	345
9+30A	14.5	150.6	373
9+50A	15	250	500
14+70A	14.1	23	57
14+88.75A	13.5	23	57
15+10A	11.8	50	124
15+80A	13.5	65.5	219
18+40A	8.8	46	114

[1]Calculated for 3-inch ID drill hole, neglecting space taken by installed drill pipe.
[2]Volume of grout placed for 6+20A through 9+50A was estimated based on the methodology presented in the 1966 grouting report regarding how grout was batched: all grouting started with a 3:1 water-cement ratio by volume and was thickened every 50 sacks of cement until the water-cement ratio was 0.8:1.

GROUT CLOSURE ANALYSIS

Figure 7 shows all of the grout holes from the 1966 and 1978-1980 grout curtain extensions in the left abutment and potential windows in the grout curtain where seepage is more likely to get through.

The grout holes for the 1978-1980 grout curtain extension were angled 20 degrees from vertical and were oriented into the abutment to maximize the interception of the high angle joints. The grout holes in the 1966 grout curtain extension were vertical resulting in a large window in the curtain at Sta 5+00G (Figure 1) where the two sections of the grout curtain meet at the surface. The window is approximately 100 ft wide at the base of the grout curtain.

© 2016 Deep Foundations Institute

The final hole spacing for the grout holes in the 1966 grout curtain extension varied from 5 to 20 ft. Closure of the grout curtain was not achieved in the foundation beneath the embankment, where the grout holes were spaced 10 to 20 ft. apart. Grout takes were high (greater than 2 sacks per length of stage) in many of holes in this area. This area is highlighted in Figure 7. Also highlighted, are areas in the Poleo Sandstone and the Arroyo del Agua where there were numerous holes with large grout takes (in red) in the Tertiary grout holes that were grouted under Modification 4 of the grout program. Additional grout holes would have been needed in these areas to close the gaps in the grout curtain.

The following are additional issues with the 1966 grouting program that reduce the effectiveness of the grout program. Refusal was set at a maximum of 500 sacks of cement placed if the target rate of take (0.1 cubic feet per minute) was not achieved. Therefore grouting was likely halted in areas before actual refusal was achieved. The majority of the grout holes were vertical and likely missed some or many high angle joints in the rock. High grout pressures were used (greater than 1 psi per ft. of depth in some locations) that could have hydraulically fractured the foundation and increased the permeability.

Final hole spacing was much closer (2.5 to 5 ft) for the 1978-80 grout curtain extension and better closure was achieved in this area of the grout curtain (Sta 5+00G to 10+00G).

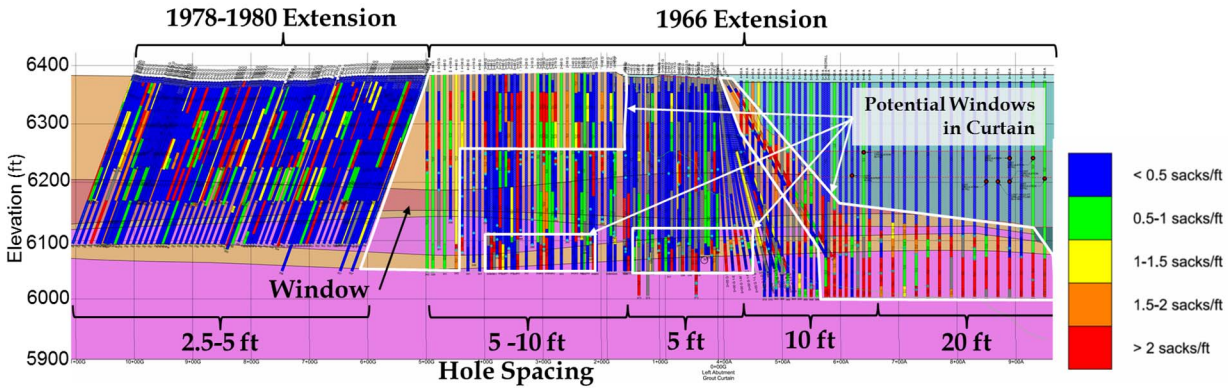


Figure 7: All grout holes from the 1966 and 1978-80 grout curtain extensions in the left abutment, showing a significant gap in the grout curtain and potential windows in the grout curtain.

Figure 8 shows all of the grout holes from the 1978-1980 grout curtain extensions in the right abutment. Final hole spacing ranged from 1.25 ft. to 10 ft. The closest hole spacing (1.25 ft.) was between Sta 14+70 and 14+99 (highlighted in white in Figure 8). The largest grout take in the foundation beneath the embankment was in the Arroyo del Agua Formation between Sta 14+70A and 15+07.5A. Grout holes in this area were split spaced down to Quaternary and Quinary holes, 2.5 and 1.25 ft apart, respectively. The total grout take in the Arroyo del Agua Formation in these holes was 6,991 sacks of cement or approximately 15,600 ft^3 of grout. The tertiary grout hole at Sta 14+85A received the largest volume of grout, where 2,449 sacks of cement or approximately 4,545 ft^3 of grout was injected into the formation. Closure was not achieved in this area of the grout curtain as there were still high grout takes in the Quinary holes. These high grout takes may be from the use of high pressures during grouting resulting in hydrofracture of the foundation. Gage pressures up to 120 psi were used in this area. Grout holes between Sta 14+70 and 19+65 were vertical and likely missed some or many high angle joints in the rock. Similarly to the 1978-80 grout curtain extension in the left abutment, better closure was achieved in the right abutment grout curtain than in the 1966 grouting program. There are a still a few areas where the closure criteria was not met most of which are in the Poleo Sandstone between Sta 0+00S and 5+00S.

© 2016 Deep Foundations Institute

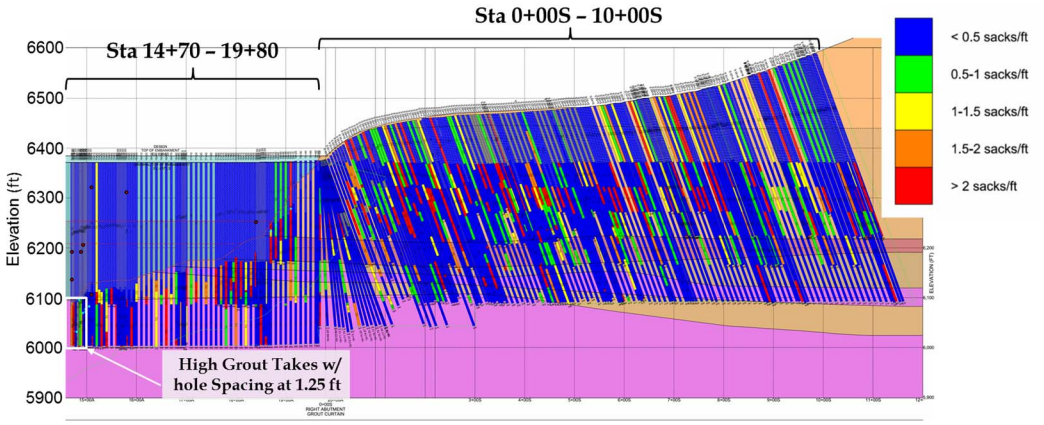


Figure 8: All grout holes from the 1978-80 grout curtain extensions in the right abutment.

SUMMARY AND CONCLUSIONS

It is likely that the drilling methods employed for these grouting and drilling programs caused hydraulic fracturing of the impervious core. The fluid losses in the embankment may be related to seasonal shut-down layers in the embankment or steep benches along the abutment foundations causing low stress zones. In addition, it is likely that the impervious core was fractured from grout pressures during installation of grout pipe through the embankment. Current USACE drilling regulations (ER 1110-1-1807 Drilling in Earth Embankment Dams and Levees, 2014) do not allow use of drilling fluids while drilling through the core of an embankment and require an approved drilling program plan prior to starting work. In addition, they require stage grouting when installing casing through a core to avoid hydrofracture.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the entire Abiquiu Dam Issue Evaluation Study Team.

REFERENCES

USACE Albuquerque District. (1963, February). Foundation Report Embankment and Spillway, Abiquiu Dam and Reservoir Project. Rio Chama, New Mexico.

USACE Albuquerque District. (1966, March). Specifications for Drilling and Grouting Abiquiu Dam CIVENG-66-43.

USACE, Albuquerque District. (1967, April). Supplemental Grouting Contract No CIVENG-29-005-66-43. Rio Chama, New Mexico.

USACE, Albuquerque District. (1978, May). Specifications for Supplemental Grouting and Drainage System, Abiquiu Dam, New Mexico.

USACE, Albuquerque District. (1981, February). Memorandum, Report on Abiquiu Dam Exploratory Grout Curtain Holes.

USACE, Headquarters. (2014, December). Engineer Regulation 1110-1-1807, Drilling in Earth EMbankment Dams and Levees.

USACE, Waterways Experiment Station. (1984, October). Foundation Grouting Practices at Corps of Engineers Dams.

© 2016 Deep Foundations Institute

DIAPHRAGM WALL CONSTRUCTION, INSTRUMENTATION, AND TESTING FOR A HYDROELECTRIC POWER FACILITY

Jeff A. Segar, PE, SE, Braun Intertec Corporation, Minneapolis, Minnesota, USA, 952-995-2340, jsegar@braunintertec.com
Nicolas A. Willig-Friedrich, Case-Bencor JV, Roselle, Illinois, USA, nawillig@casefoundation.com
David Gatto, Ames Construction, Minneapolis, Minnesota, USA, DavidGatto@amesco.com

ABSTRACT

With the increasing demand for electricity, and requirements to generate this power cleanly, more producers are turning towards hydroelectric power. A project in Iowa currently under construction will retrofit an existing Army Corps of Engineers Dam with a hydroelectric generating facility. The design of the intake structure requires the construction of a diaphragm retaining wall. The T-shaped diaphragm wall elements retain an unbalanced fill height up to 21.3 meters (70 feet). The design required a single production element be constructed, tested, and instrumented prior to beginning construction of the rest of the elements. The testing included extensive concrete mix design development, coring, and Cross Hole Sonic Logging. Installed in the other production elements are 10 inclinometers, 4 pressure cells, 4 piezometers, and multiple survey monuments. Some of these instruments require manual readings while most are monitored by an automatic data acquisition system and instantly transferred to a website providing real-time graphical display of the data. This paper will discuss the construction of the diaphragm wall from the contractor's perspective when working on a site only large enough for two service cranes and a Hydromill excavation unit, without any storage area on the construction pad, describe the testing performed and the quality control procedures employed, and investigate the information gained from the automated instruments providing real time measurement readings.

Keywords: Diaphragm Wall, Instrumentation, Slurry,

INTRODUCTION

In 1964 the US Army Corps of Engineers (USACE) constructed a dam by raising the surface grade approximately 34.7 meters (114 feet) above the natural ground line using a combination of fill types. This earthen dam stretched slightly longer than a mile intersecting the flow of the existing Des Moines River. The center of the dam is a concrete spillway consisting of 12 monolith sections. On each end, three of the sections extend 42.6 meters (140 feet) from the spillway to transition from the concrete dam sections to the earthen dam section. Adjacent to these three sections on the East end is where the contractor is actively working, for a private developer under a lease agreement with USACE, on this site to convert it to a Hydroelectric facility licensed for an output of 36.4 megawatts, enough power to meet the needs of about 18,000 homes. To create this, a powerhouse structure, housing two vertical Kaplan turbines is being added to the downstream side of the existing dam. From the power house, two penstock pipes will be cored through the existing concrete monolith sections and connect to the Intake structure on the upstream side of the dam. Protecting and reinforcing the dam adjacent to the intake structure is a new diaphragm wall. This wall is constructed using T-shaped diaphragm elements, acting as a cantilever retaining wall for the intake platform. Figure 1 shows a diagram of the upstream features of the construction.

© 2016 Deep Foundations Institute

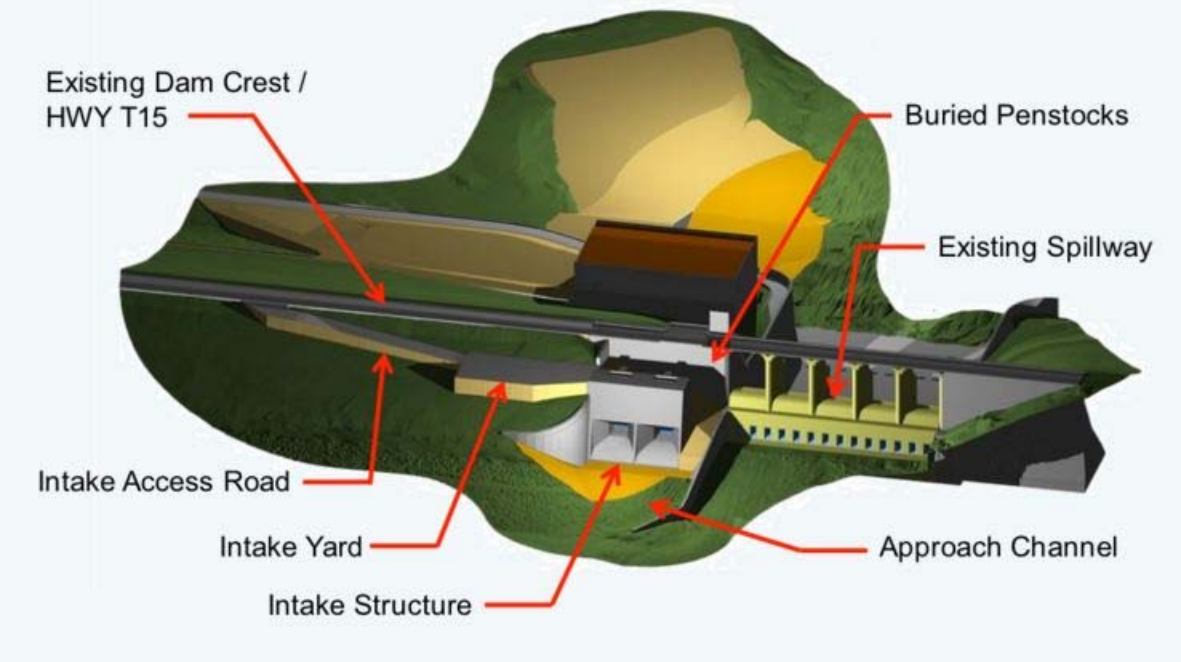


Fig. 1. Upstream features, diaphragm wall can be seen to the left of the intake structure. (Image copyright Missouri River Energy Services)

GEOTECHNICAL SITE CONDITIONS

Site Investigation

The borings taken for the original dam construction in the 1960's show approximately a three meter (ten foot) layer of clay, often noted just as "Impervious" material, underlain by roughly a 6 meter (20 foot) layer of sand and gravel to rock consisting of alternating layers of sandstone and limestone above a thick layer of dolomite. From the original construction drawings, the clay was to be stripped off and 27.4 meters (90 feet) of pervious fill was to be placed on top of a 9.1 meter (30 feet) thick impervious keyway, additionally the fill extending for 6 meters (20 feet) horizontally from the face of the monoliths was to be impervious for the full 36.5 meters (120 feet) of backfilling.

Test borings taken by the owner's geotechnical engineer and the contractor's geotechnical engineer, in the area of the diaphragm wall showed that the soils at the site consisted of granular soils overlying alternate layers of sandstone and limestone bedrock. The granular soils were categorized as medium to very dense. The sandstone had Rock Quality Designation (RQD) values ranging from 0 to 56 and the Limestone had RQD values from 42 to 71. One boring, positioned near the end of the diaphragm wall encountered clay fill ranging in consistency from rather soft to hard extending to the elevation of the native sand layers.

Laboratory and Insitu Testing

A number of various laboratory and insitu tests were conducted by the owner's geotechnical consultant across the site to identify the material strength parameters. Unconfined Compression Tests were conducted on various clay, limestone and sandstone samples. Water pressure tests were conducted at various levels in the rock stratum to determine the hydraulic conductivity and the Lugeon pattern. Triaxial tests were also performed on six samples.

© 2016 Deep Foundations Institute

DIAPHRAGM WALL PRODUCTION

Slurry mix

Based on the information from the geotechnical exploration, the trench supporting slurry for the diaphragm wall production would need to be able to support soil pressures exerted from medium to very dense sands and from rather soft to hard clays for depths up to 39.6 meters (130 feet) and with equal depth of water head. Additionally this slurry would need to support the surcharge pressure from the equipment sitting 5.2 meters (17 feet) away from the open excavation. To accommodate these challenges the diaphragm wall specialty contractor used a bentonite based slurry mix composed of 4% of bentonite by weight and miscellaneous admixtures to control density and viscosity during excavation. To provide a reasonable factor of safety against slurry loss or washout a reserve tank holding 63,000 liters (16,000 gallons) was placed in-line with the desanding process to be able to fill additional slurry rapidly as needed.

Production Process

The diaphragm wall production process began with a 1.2 meter (four foot) thick formed concrete guide wall to provide an outline and a "guide" for the work to come. Then the excavation process was started using a mechanical clamshell operated by a crane through the variable overburden soils to the rock surface for the primary elements. Once rock was encountered in the primary elements and for the entirety of the secondary elements the excavation was performed using a Bauer BC-40 trench cutter capable of cutting a panel of 1.5 meters (5 feet) by 3.2 meters (10.5 feet). The frame of the cutter head is equipped with inclinometers for measuring inclination in the horizontal X and Y axes and gyroscopes for measuring the rotation about the vertical Z axis. This information is displayed in real time for the operator to monitor and make the required adjustments to the cutter head allowing Case-Bencor JV to achieve the specified vertical tolerances of 1 horizontal to 200 vertical.

The tee shaped elements were created with three bites excavation for the stem of the tee (two (2) bites) and one bite making the top of the tee secondary elements overlap 203 mm (8 inches) into the primary elements. After completion of any given element, verticality of the excavated element was confirmed using an Ultrasonic Echometer (Koden). The Koden ultrasonic sensor is lowered down the shaft emitting waves that then reflect off of the walls and are received by the sensor. The time for the wave to return is measured and since the velocity of the wave is known, the distance from the wall to the sensor can be calculated within 2% accuracy. This operation was critical as the tee shaped reinforcement was set in full depth and little free space was available.

Because of the limited space on the upstream platform the slurry mixing and desanding plant had to be placed on the downstream side of the dam where space was available. A concrete culvert was installed below the roadway on top of the dam for the piping. The culvert's design incorporated a flood door to seal off the passage in the event of high water, protecting the dam from water rushing through the tunnel and eroding the downstream earthen core.

Rebar cage testing

The rebar cages were constructed at the laydown area site on the downstream side of the dam. They were tied together as one cage, picked horizontally in one piece, placed on a transport trailer and driven though the nearby town in the middle of the night to get to the upstream platform. The rebar cage for the test element was the first cage that would be placed, it was one of the largest cages at 3 meters (10 feet) wide by 39.6 meters (130 feet) long and weighing approximately 890kN (100 tons). Once the tipping process

© 2016 Deep Foundations Institute

was completed the cage was suspended from a custom designed lifting device that contained five junior beams and one spreader beam.

The preliminary design for the connection from the cage to the junior beams was to extend some of the main rebar beyond the end of the cage and clamp them to the junior beams with wire rope clamps. Due to the non-uniform distribution of rebar within the cage the center of gravity of the cage was approximately 600 mm (2 feet) eccentric from the geometric center. The cage would hang in the shaft from the junior beams until the tremie process was complete which required the junior beams to be clear of the tremie pipe locations and resulted in varying loads in the connections at each junior beam with the maximum expected connection load of 62 kN (14 kips).

To verify the strength of the connection, the preliminary design was tested in Braun Intertec's lab. Standard rebar has two distinct rebar deformations: one set of deformations that runs longitudinal to the central axis of the bar at 180° apart, and a series of transverse deformations connecting at a skew angle between the longitudinal deformations. To account for the different orientations that the bar could be inserted one test was run with the apex of the clamp on the longitudinal deformation and one test with the apex of the clamp on the transverse deformations. Figure 2 shows the load vs displacement curve for the controlling situation when the apex of the clamp was on the longitudinal deformation of the rebar. In this orientation the maximum test load achieved was approximately 102 kN (23 kips), when the apex of clamp was on the transverse deformations the maximum load was approximately 124 kN (28 kips). The testing showed that the clamps would provide a factor of safety ranging from 1.6 to 2.0, for this condition the required factor of safety should be 3.0. Both maximum capacities occurred at a deflection of about 6 mm (0.25 in), visual observation showed that most of this deflection was slipping of the rebar through the clamp and bending of the clamp assembly.

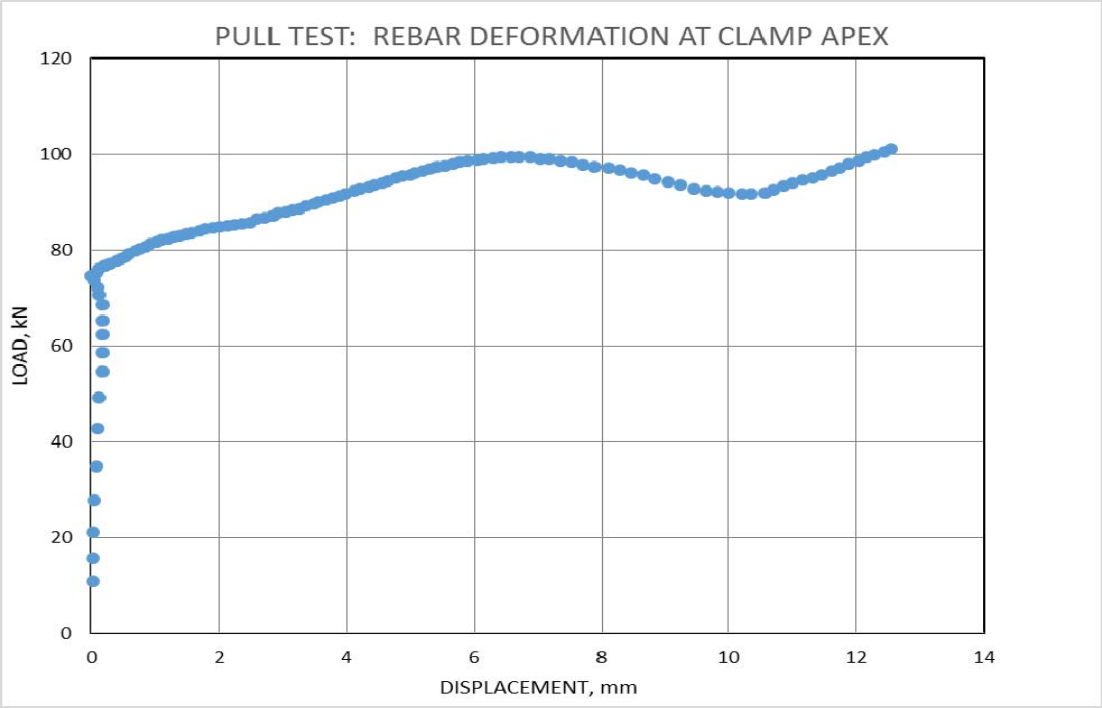


Fig. 2. Load vs Displacement for rebar test

To increase the factor of safety the connection was re-designed by adding a rebar coupler bearing on two stiffener plates. Between the coupler and the bearing plates a 6 mm (¼") plywood spacer was installed to

© 2016 Deep Foundations Institute

allow the wire rope clamps to engage and take all of the anticipated loading before crushing the plywood and transferring the load into the coupler, ensuring that the coupler remains a backup system. Rebar couplers of this type are designed to transfer 125% of the yield strength of the bar, for this application the bar yield strength provides a factor of safety of 2.67 over the design loads. Figure 3 shows the final connection design.

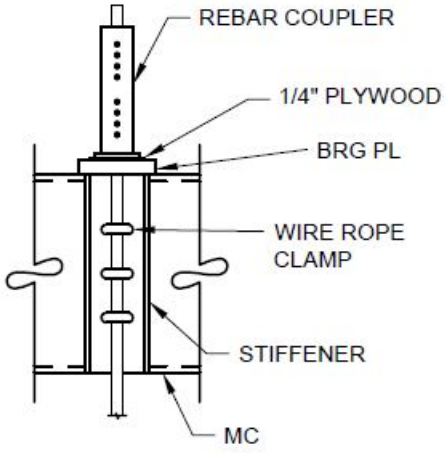


Fig. 3. Clamp design

Rebar cage picking

Picking the rebar cage was done with the primary crane, a Manitowoc 2250, and a Manitowoc 888 as the assist crane. The process began by backing the cage down on the platform in between the two cranes, resulting in approximately 4.5 m (15 feet) of clear space on each side of the cage. The assist crane began the lift with an eight point pick of the lower three-quarters of the cage as the primary crane lifted at the spreader beam. Because of the tight space and loads near crane capacity, the cranes had to position their booms at their maximum vertical position. The process required frequent stops to verify distances between the booms, at one position the booms would pass within a meter of each other. Once successfully tipped vertical the cage was completely suspended on the spreader beam and junior beam system. Figure 4 shows a photo of the tripping process and Fig. 5 shows the cage hanging from the beam system getting inserted into the excavation.

© 2016 Deep Foundations Institute



Fig. 4. Photo of Rebar Tipping



Fig. 5. Photo of Rebar Placement

© 2016 Deep Foundations Institute

CONCRETE MIX DESIGN

Three factors contributed to making the tremie concrete mix design for the diaphragm walls difficult. These were the specification, the contractor workability requirements and the local supplier's familiarity with different admixtures. The specification for the tremie concrete had acceptance values for slump, visual stability index, washout, bleedrate, time of set and air content that impacted the design of the mix. In addition the contractor had requirements for time of set and workability that were difficult to achieve while still meeting the acceptable range of values provided in the specification and using the local admixtures. To meet the requirement for less than 5% washout by mass, an anti-washout admixture or viscosity modifier is often used. Increasing the viscosity impeded the removal of the casings for the secant piles installed elsewhere on the project and produced concerns that the mix would not uniformly flow from the tremie pipe to fill the diaphragm.

The specification required the initial time of set to occur between 3 to 26 hours with the final time of set to happen within 48 hours. Based on the construction process and unknowns of moving the equipment and rebar cages around in the small space available, an initial set time near 24 hours was targeted with the concrete remaining workable for 8 hours.

Mortar batches as well as calorimetry were used to determine the impacts of admixtures and their dosages on the bleed rate and set time. These initial tests aided in the determination of which admixtures to use and provided a starting point for the dosages before moving to concrete batching and testing in the lab. The batches were refined using different ratios of cement to fly ash. Figure 6 shows the change in temperature with time based on 22% fly ash to cement and different rates of Recover hydration stabilizer. The initial rise in temperature is used to estimate the initial set of the concrete. Based on these tests a trial batch was made using 2.5 oz./cwt of stabilizer. This batch had an initial set of 15 hours and a final set of 17 hours, while providing workability for the eight hour window.

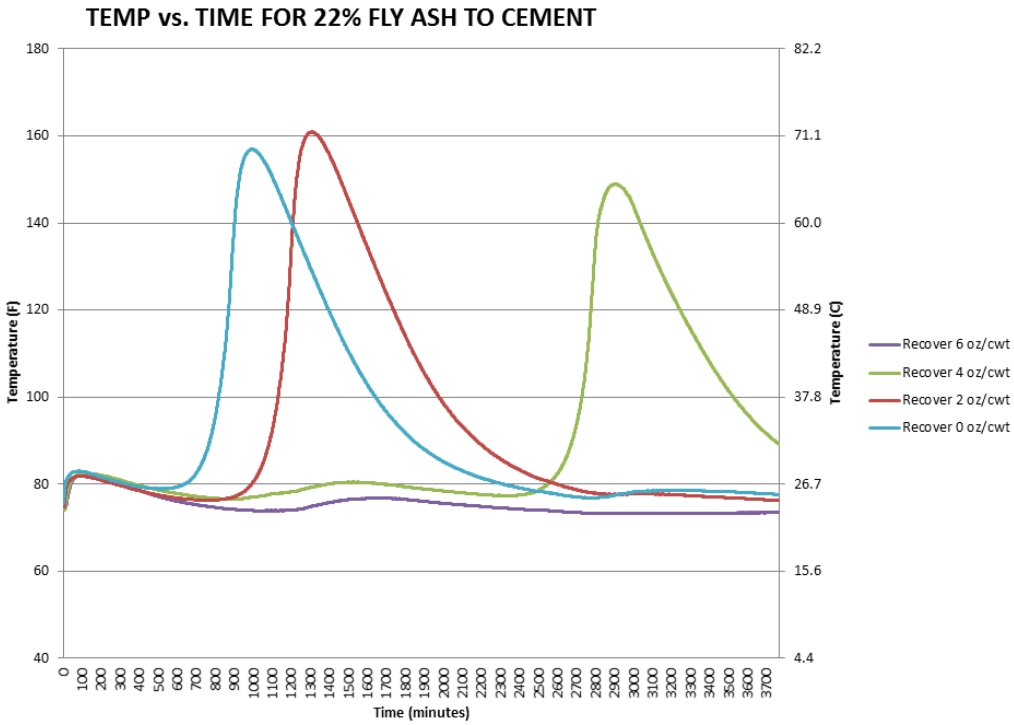


Fig. 6. Calorimetry Data

© 2016 Deep Foundations Institute

D-14 PANEL TESTING

Production Concrete Testing

After casting the first element, D-14, the specifications called for allowing the concrete to cure for 45 days and then coring six holes through the full depth of the element to collect samples for compression testing and to compare to the samples taken from the truck during production. Cross Hole Sonic Logging (CSL) tests were also performed. The core locations were placed to be in line with the CSL tubes to further validate the CSL results as shown in Fig 7. The 247 m (800 feet) of cores were visually observed and thirteen samples were selected by the Owner's engineer for compression testing. The samples averaged 58 Mpa (8,500 psi) nearly double the specification requirement of 31 Mpa (4,500 psi)

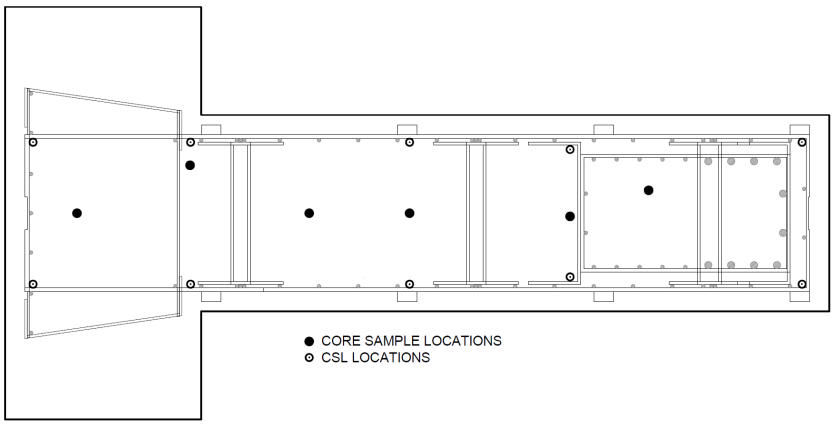


Fig. 7. D-14 Core sample and CSL test locations

INSTRUMENTATION

Layout

Installed in the diaphragm wall are 5 manual inclinometer casings, 5 addressable MEMS in-place inclinometer arrays, 4 vibrating wire push-in pressure cells with built-in piezometers, 4 vented vibrating wire piezometers in open standpipes, and multiple survey monuments. The majority of these instruments were monitored 24/7 by a solar-powered Automatic Data Acquisition System (ADAS), consisting of dataloggers, multiplexers, and cell modems, that automatically recorded, reduced, and presented data on an instrumentation and monitoring system website. Figure 8 shows the real-time display page for the diaphragm wall on the monitoring website, accessible only through a secure log-in webpage. Current instrument readings are displayed on the page, updated every 4 hours when a new reading is recorded. The figure also shows the location of the instruments whose readings are manually collected.

© 2016 Deep Foundations Institute

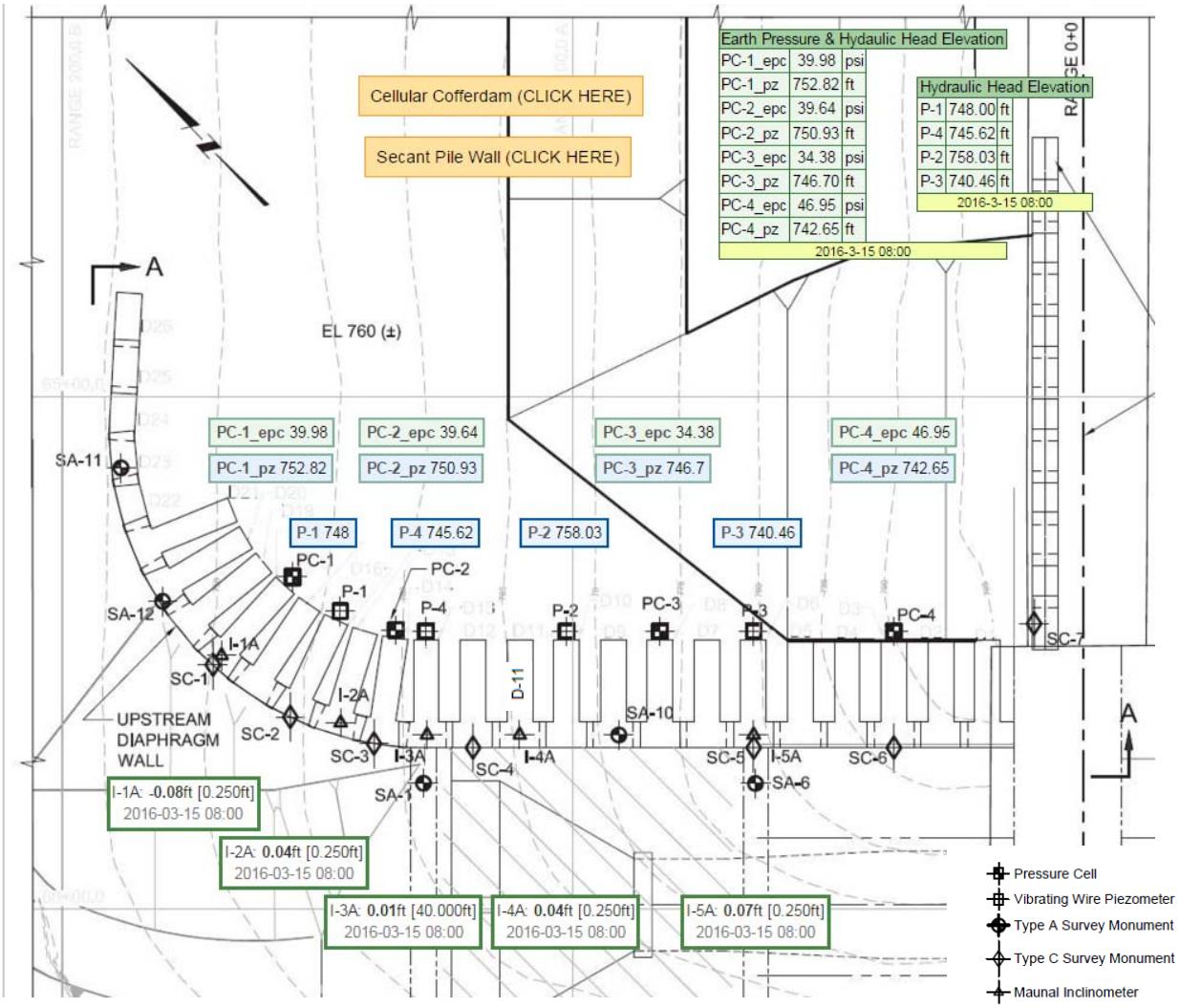


Fig. 8. Instrumentation Layout.

From the website, you can select any specific instrument, and view, scale, table, graph, or download the data for any timeline you desire. The website has also been configured to automatically create and deliver monthly reports (via email to specific project personnel), with data and template formatting tailored to the project. Figure 8 shows over a time period of 90 days, the cumulative displacement of in-place inclinometer array I-4A, which is located in element D-11.

Inclinometer array I-4A extends 6m (20 feet) below the bottom of the diaphragm element into the bedrock. The displacement limits for I-4A of 13mm (.042 feet) and 25.4 mm (.083 feet) are displayed on the graph by the gold and red lines. Displacement limits are greater in value for the other inclinometer arrays farther away from the intake structure. The gold line indicates a threshold displacement value, and if exceeded, an email alert is automatically generated to specific project personnel to perform further investigation in the situation, with corrective actions possible. The red line indicates an action level displacement value, which will call for a corrective action to be implemented if exceeded.

© 2016 Deep Foundations Institute

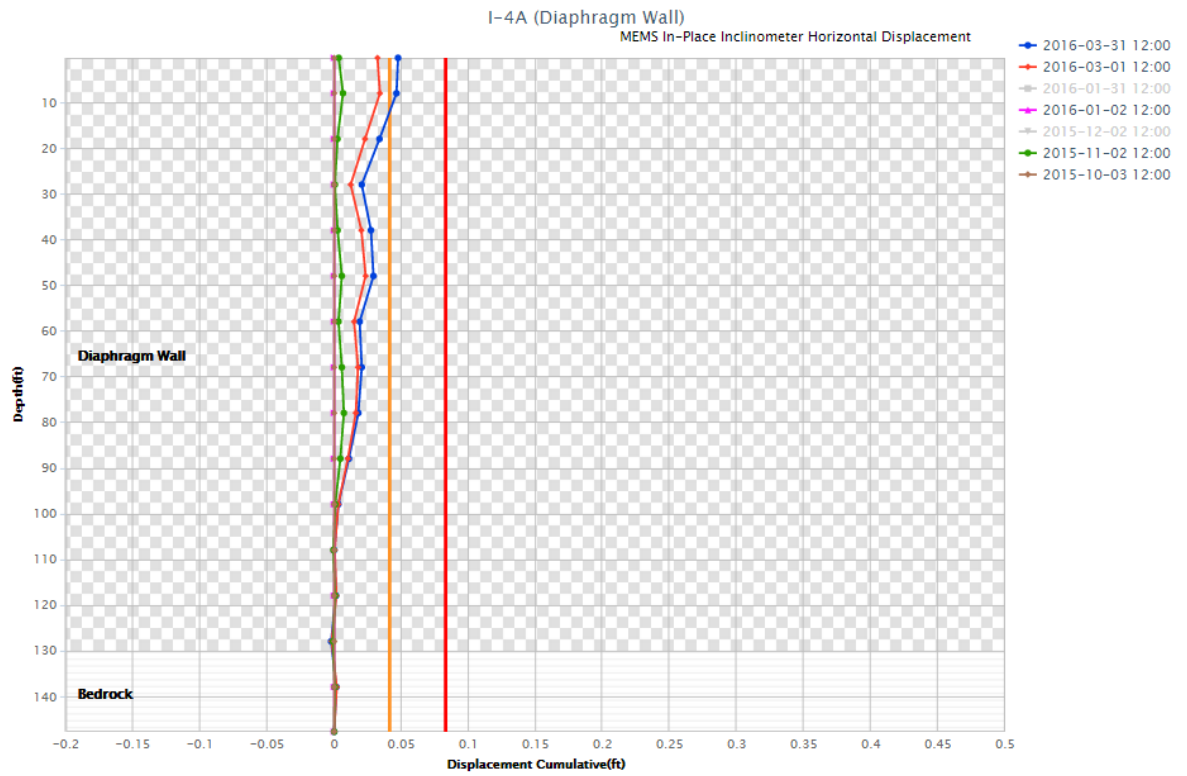


Fig. 9. D-11 displacement graph, as shown on the ADAS website

During the bidding phase, an analysis of the design was performed to establish a base line for providing value engineering alternates. The baseline analysis for this element calculated a maximum deflection of 25.4 mm (1 inch) consistent with the limit set above.

Figure 10 shows the elevation readings and Fig. 11 shows the pressure readings over a six month period for the pressure cells with built-in piezometers. Comparing these graphs shows that as the water levels have fallen the pressure has decreased. Additionally the pressure cells are showing a consistent decrease in pressure as a result of dissipation of pore pressure from their installation. In figure 9 the rapid rise of water in P-1 shown around January 13, 2016 was due to a heavy rain event. Piezometers P-2 through P-4 did not increase in head as rapidly showing the soil did not saturate to those piezometer locations.

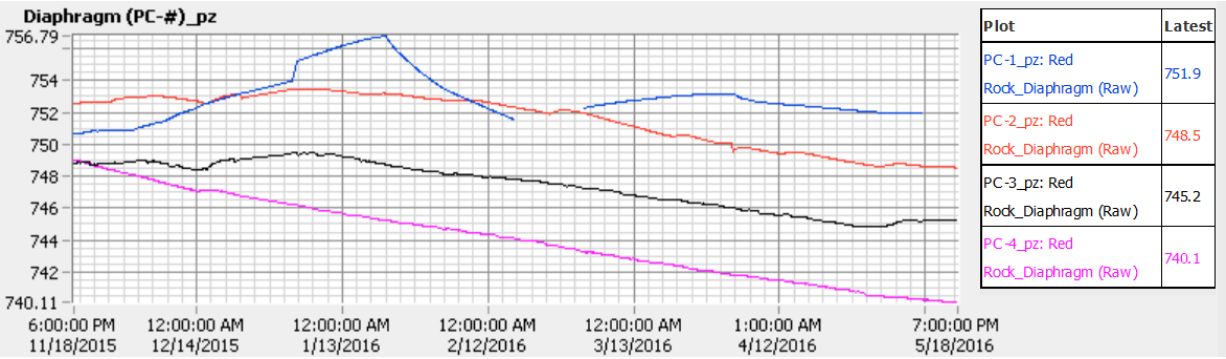


Fig. 10. Piezometer head elevation history readings in feet, as shown on the ADAS website.

© 2016 Deep Foundations Institute

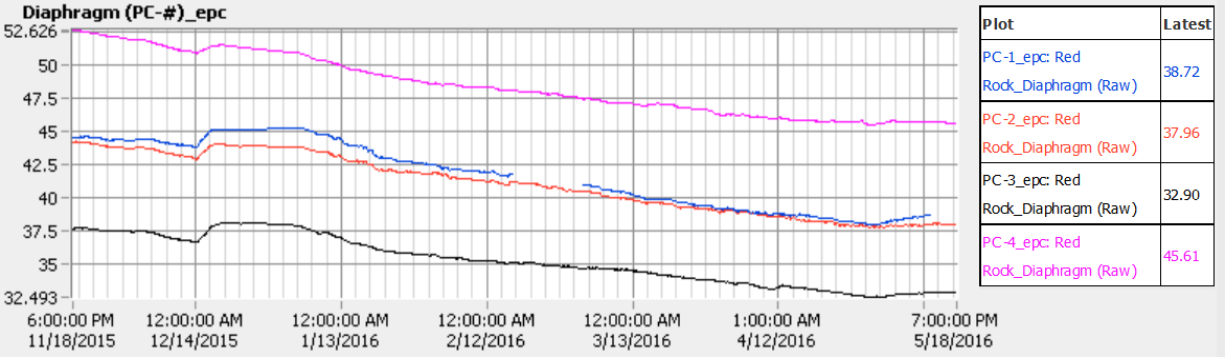


Fig. 11. Pressure cell history readings in psi, as shown on the ADAS website

CONCLUSIONS

The diaphragm wall construction required extensive planning and investigation prior to construction. The concrete mix design was formulated to provide workability and time of set for the contractor's production while meeting the requirements of the specification. Trial batches were tested using calorimetry to determine the appropriate dosage rates. The slurry mix was designed to provide a stable excavation and support the surcharge from the equipment. Load testing was performed on the clamping system for supporting the rebar while it was lifted and placed into the slurry, resulting in a re-design of the connection.

Once the first element was completed it was tested for quality assurance. The concrete was cored and compression tested and also tested using cross hole sonic logging. Then instruments were installed consisting of manual inclinometer casings, addressable MEMS in-place inclinometer arrays, vibrating wire push-in pressure cells with built-in piezometers, vented vibrating wire piezometers in open standpipes, and multiple survey monuments. Some of these instruments provide real time data to a controlled website where team members can see graphical data.

REFERENCES

ACI Committee 318, *Code Requirements for Structural Concrete (ACI 318-11) and commentary*, American Concrete Institute, Detroit, 2011, 503 pp.

EFFC/DFI Joint Concrete Task Group. *EFFC/DFI Best practice Guide for Deep Foundations.* 1st Edition. 2016

Geokon. *Instruction Manual Earth Pressure Cells.* Rev M 3/13. Lebanon, NH, 2013, 35 pp

Geokon. *Instruction Manual Vibrating Wire Piezometers.* Rev Z 2/16. Lebanon, NH, 2016, 32 pp

Geokon. *Instruction Manual MEMS Digital Addressable In-Place Inclinometer.* Rev E 5/16. Lebanon, NH, 2016, 37 pp

Red Rock Hydroelectric Project. *Project Overview.* Missouri River Energy Services. Web. May 18, 2016.

Vista Data Vision, *VDV Online Manual.* Reykjavik, 2014, 75 pp

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

SEISMIC STABILITY ANALYSIS OF SEEPAGE CONTROL MEASURES FOR LEVEES IN CENTRAL VALLEY, CALIFORNIA

Zia Zafir, PhD, PE, GE, Kleinfelder, Rancho Cordova, California, USA, (916)366-2382, zzafir@kleinfelder.com

ABSTRACT

Levees in the central valley of California are primarily for flood control purposes and thus are subjected to infrequent hydraulic loading. The primary concern for these levees is their instability due to through and under seepage. Primary seepage control measures are seepage cutoff walls and/or seepage berms. The cutoff walls primarily consist of soil-bentonite (SB) slurry walls. The seepage berm could be a drainage blanket to allow the seepage with controlled exit gradient or an impervious layer to provide enough cover to protect any adverse effects of seepage in terms of sand boils. Although, Sacramento is an area of moderate seismic activity, levees protecting urban areas require assessment for the potential instability due to liquefaction and associated vertical and lateral deformations. In certain cases, the seismic deformations of the levee may not compromise the stability of the levees, but they may cause damage to the seepage control measures. Depending on the nature and extent of the damage, the mitigation measures may require limited or full replacement of the seepage control measures. This paper will provide a systematic assessment of these different types of seepage control measures under vertical and lateral deformations due to liquefaction and lateral spreading under seismic loadings.

Keywords: seismic, levee, seepage

INTRODUCTION

There are two broad categories of levees in Central California: levees that are subjected to continuous hydraulic loading such as delta levees and are called frequently loaded levees (FLL) and flood control levees that are subjected to seasonal or infrequent hydraulic loading, called infrequently loaded levees (ILL). This paper will focus on the seismic response of ILL. The probability of ILL experiencing an earthquake at the time of the design flood is very low. Therefore, consequences of seismic failure of flood control levees are only significant in terms of response time required to restore the level of flood protection provided by the levee before the next flood season. An example for this restoration is the restoration of the Pajaro River levees in Santa Cruz, California after the 1989 Loma Prieta Earthquake. Nevertheless, failure of flood control levees due to a seismic event could result in loss of the entire levee and/or loss of freeboard. In addition, some of the seismic failure of ILL levees may result in loss of seepage control measures such as the soil-bentonite wall.

The State of California State approved Senate Bill (SB) 5 in 2007 that mandates for 200-year flood protection to be the minimum level of protection for urban and urbanizing areas in Central California (Sacramento-San Joaquin Valley) by the year 2025. This legislation also requires that urban and urbanizing areas protected by Federal project levees cannot use adequate progress as a condition to approve development after 2025. SB 5 requires that the urban level of flood protection be consistent with criteria used or developed by DWR (Government Code § 65007(k)). To avoid delaying urgently needed flood protection, Urban Levee Design Criteria (ULDC, 2012) was developed.

Under a contract from DWR, Kleinfelder was tasked to perform a screening-level seepage and stability of levees in thirteen study areas within San Joaquin Valley. Many of the levees in these study areas are ILL and an example from the Walthall Slough study area (Figure 1) will be presented in this paper.

© 2016 Deep Foundations Institute

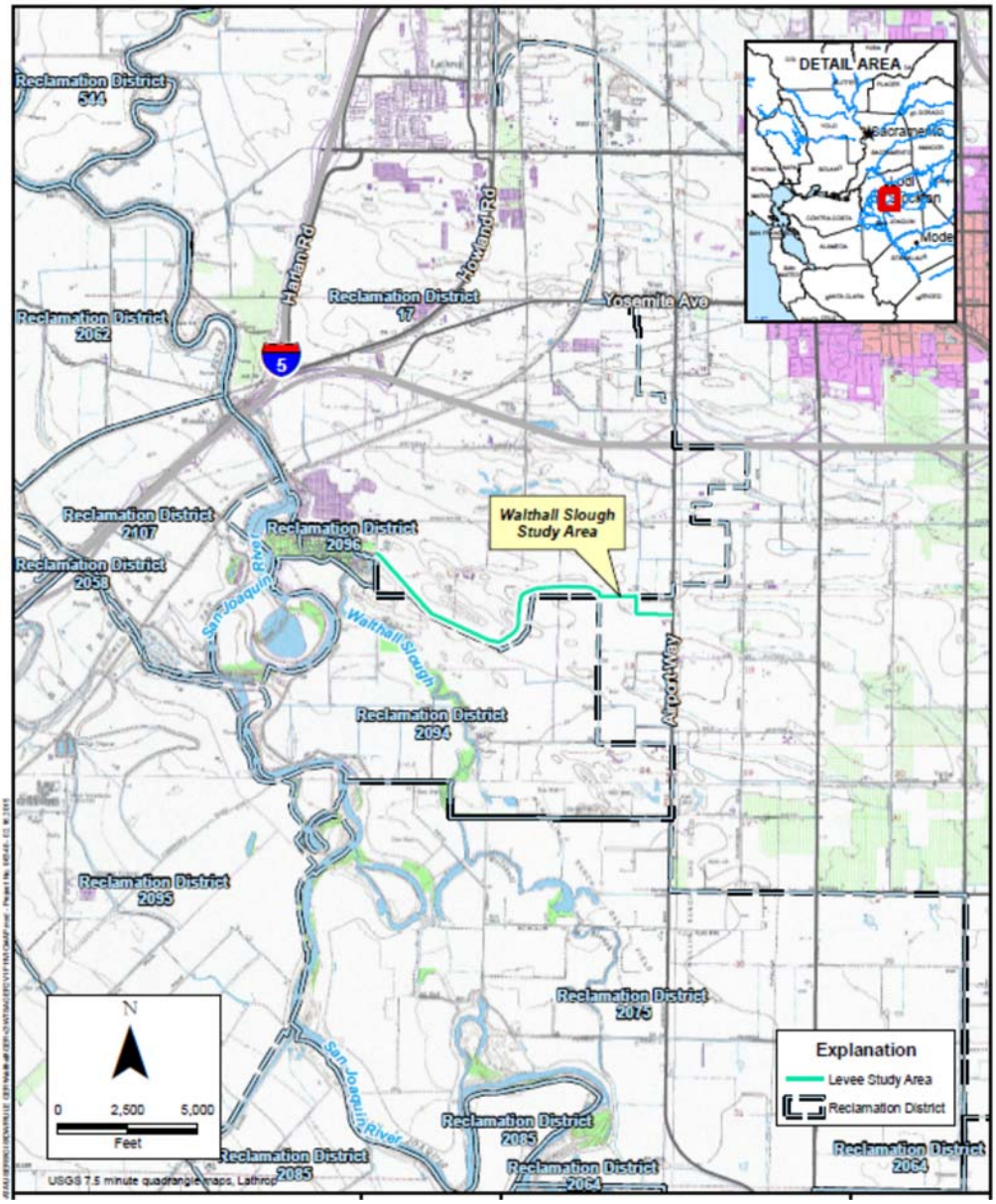


Figure 1: Walthall Slough Study Area

PROPOSED PROCEDURE FOR SEISMIC ANALYSIS

A detailed step-by-step approach for completing the analyses to assess the seismic stability of the ILLs has been presented in DWR Guidance Document Version 14 (URS, 2014). A flow chart showing the proposed approach is presented in Figure 2. The approach is organized into following steps of analyses:

- Perform liquefaction triggering analyses to assess the potential for liquefaction and assess longitudinal extent of liquefaction.
- Estimate post seismic shear strengths for various layers.
- Perform post-earthquake static slope stability analysis to assess flow failure.
- Estimate the yield acceleration (k_y) by performing section-specific pseudo-static slope stability analyses.
- Assess the seismic vulnerability using the vulnerability classification cases.

© 2016 Deep Foundations Institute

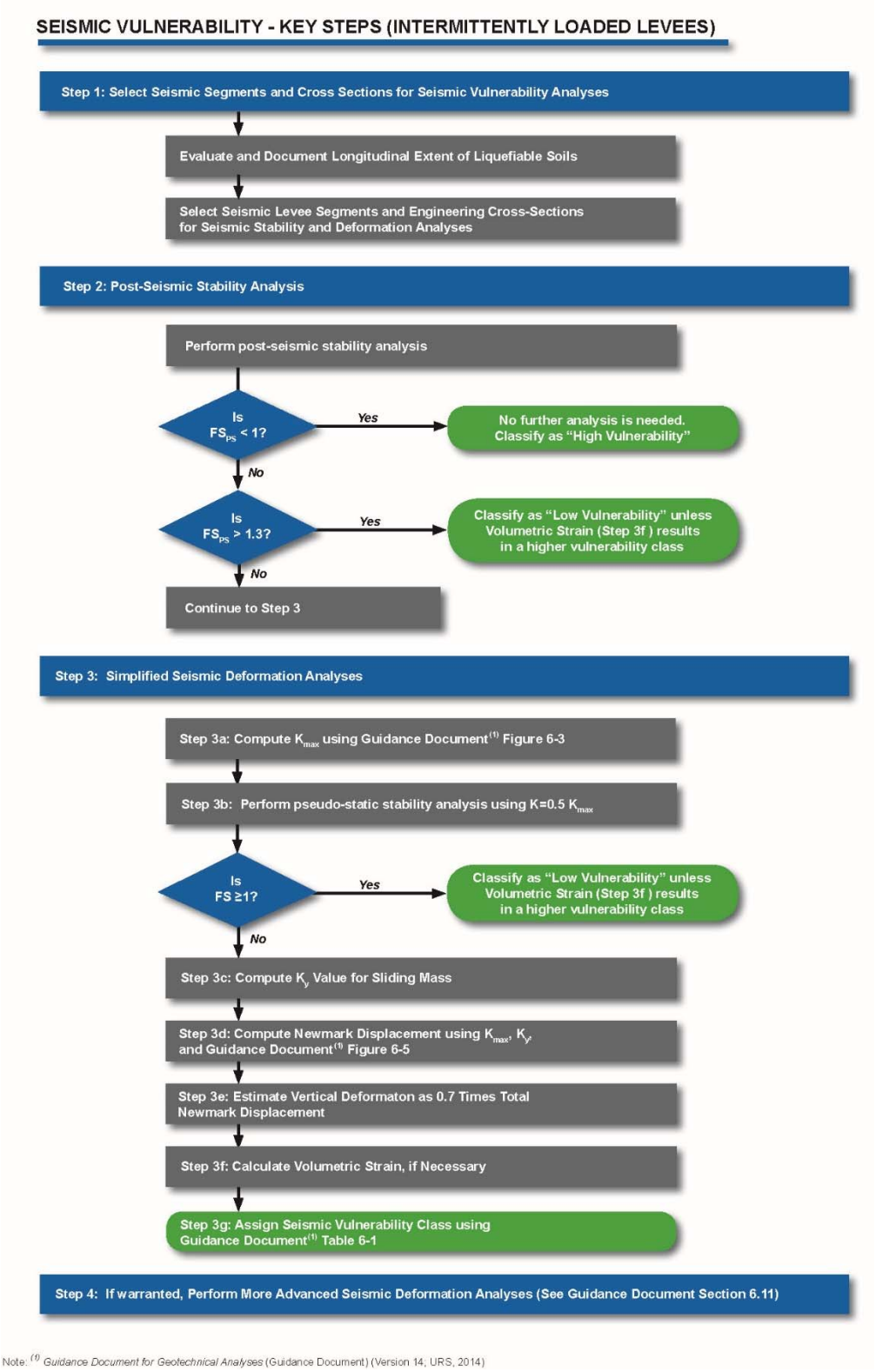


Figure 2: Seismic Analysis Procedure for Intermittently Loaded Levees

© 2016 Deep Foundations Institute

These steps outlined in Figure 2 are further explained below.

Evaluation of longitudinal extent of liquefaction

The entire process of seismic evaluation has been presented in the DWR Guidance Document (URS, 2014) which includes selection of reaches for analysis, development of typical cross-sections from miles of levee topography data, and development of parameters for varying soil conditions within reaches. This process of seismic evaluation involves liquefaction assessment and longitudinal extent of liquefaction, seismic slope stability, and seismic deformation estimates to assess the seismic vulnerability criteria. Since levees induce static shear stresses in the foundation, especially under the toe, free-field methods cannot be used without adjusting for existing shear stresses. Therefore, program-wide site-response analyses for typical levee cross-sections were performed to develop seismically-induced shear stress profiles under the waterside toe, landside toe, and crest of the levee. Shear stress divided by effective overburden pressure is defined as cyclic stress ratio (CSR) and factor of safety (FS) against liquefaction is defined as the ratio between cyclic resistance ratio (CRR) and CSR. The CRR is the soil shear strength divided by the effective overburden pressure.

Evaluation of soil liquefaction triggering potential is performed in two steps. The first step involves evaluating whether the soil meets the compositional criteria necessary for liquefaction (susceptibility). For soils meeting the compositional criteria, the next step is to evaluate whether the design level ground shaking is sufficient to trigger liquefaction given the soil's in-situ fabric or relative density. If it is assessed that liquefaction will be triggered, the engineering consequences of liquefaction are evaluated.

Program-wide susceptibility criteria for ILL whether silty and clayey soils meet the criteria for liquefaction susceptibility were developed. According to these criteria, soils with Plasticity Index (PI) $\geq$ 10 or Liquid Limit (LL) $\geq$ 35 or water content (wc) $\leq$ 0.8 x Liquid Limit (LL) are considered not susceptible to liquefaction. Liquefaction triggering was primarily based on Cetin et al. (2004) method. Both conventional rotary wash soil borings and cone penetration test soundings were used to evaluate soil fabric or relative density for liquefaction triggering analyses. In addition, program-wide map of peak acceleration value was prepared and an earthquake magnitude of 6.5 was established.

Assessment of water surface elevation (WSE) is very critical for both liquefaction and stability analysis. Since groundwater is usually higher during winter months in Northern California, nominal water surface elevations during the winter months are considered for liquefaction evaluation. Water surface elevations for past several years have been obtained by wells maintained by California Department of Water Resources (DWR). However, if groundwater is encountered during the drilling and it is higher than the nominal winter water surface elevation, then higher value was used in the analysis.

Based on the results of liquefaction triggering analyses on selected borings and cone penetration test soundings review of the soil profile beneath the levee, longitudinal profiles showing extent of liquefaction were developed for each reach within each study area. These profiles show likely liquefiable (defined as layers having FS against liquefaction < 1.0) and less likely liquefiable (defined as layers having FS in between 1 and 1.25) areas. An example of longitudinal extent of liquefaction is presented in Figure 3.

Seismic slope stability

Based on the longitudinal extent of the liquefaction profile (an example is shown in Figure 3), cross sections were selected for seismic slope stability analysis. The purpose of the seismic slope stability analyses is to: 1) assess the vulnerability of slopes to flow failures and 2) estimate the yield acceleration (k_y) values for potential failure surfaces, defined as the maximum horizontal acceleration resulting in a just-stable condition equivalent to a Factor of Safety (FS) of 1.0, and is used as an input in estimating deformations of slope due to seismic loading. The seismic slope stability analyses are performed in three steps.

© 2016 Deep Foundations Institute

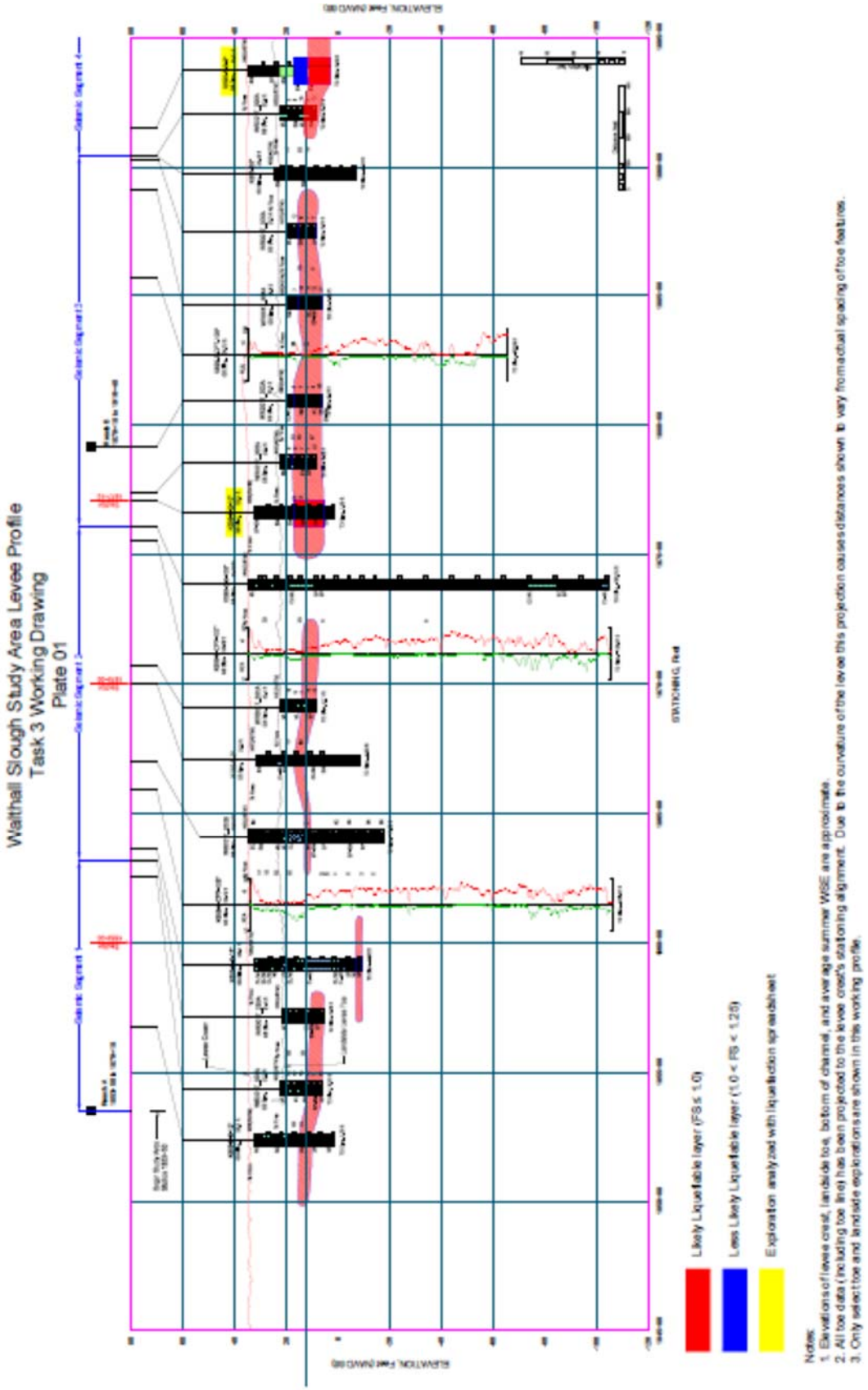


Figure 3: Longitudinal Extent of Liquefaction

© 2016 Deep Foundations Institute

1. A post-earthquake static analysis is performed using strengths of different layers including residual strengths for likely liquefiable layers and 30 percent reduced strengths for less likely liquefiable layers. Residual strengths are based on median $(N_1)_{60\text{-}cs}$ values from all the available SPT data within that cross section and supplemented by CPT data if necessary. Residual strengths were estimated from the lower third value of the Seed and Harder (1990) curve. If FS is less than 1, it means the slope is susceptible to flow failure resulting in large deformations and the levee is classified as "High Vulnerability" for seismic vulnerability. If FS is greater than 1.3, then the levee is classified as "Low Vulnerability" and no further analysis is needed.
2. IF FS is in between 1 and 1.3 in the first step, a pseudo-static stability analysis is performed using the same material properties and assuming a seismic coefficient equals to $\frac{1}{2}K_{max}$ where K_{max} is estimated using Figures 6-2 and 6-4 of the Guidance Document (URS, 2014). If the FS is greater than 1, then the levee is classified as "Low Vulnerability" and no further analysis is needed.
3. If FS is less than 1 in step 2, then yield acceleration (k_y) is estimated by performing pseudo-static stability analysis. Yield acceleration is defined as the seismic coefficient for which the FS is equal to 1.
4. Based on the yield acceleration values, deformations are estimated and levee is classified per the deformation based classification per Table 1.

The ratio between k_y and the K_{max} can be used to estimate deformations using Figure 6-5 of the Guidance Document (URS, 2014) which is based on the Makdisi and Seed (1978) method. Although the Makdisi and Seed (1978) method is not directly applicable to slopes with liquefiable layers, it can be used to estimate approximate deformations for a screening analysis.

Post-earthquake static slope stability and yield acceleration analyses were performed using well-known slope analysis computer programs such as SLOPEW. Both waterside and landside slopes were evaluated using the both circular and wedge-shaped failure modes and the most critical failure surface was assumed. For the yield acceleration analyses, only horizontal seismic coefficient (k_h) is considered.

Seismic vulnerability classifications

Based on the results of analyses, the ILLs are then classified into three classes as presented in Table 1. As shown in this table, levees classified as either medium vulnerability or high vulnerability have the potential of significant damage to the seepage control measure such as the cutoff wall during a design seismic event.

Table 1: Criteria for Seismic Vulnerability Classification

Amount of Crown[2] Displacement Relative to Landside Levee Height (percent)	Potential for Significant Damage to Internal Structures (e.g., Cutoff Walls)	Remaining Freeboard[2] for Post Seismic Evaluation (10-Year Flood WSE +3 feet[3])	Vulnerability Class[4] (Post-Seismic Flood Protection Ability)
< 10	No	>0.3 m (1 foot)	Low Vulnerability
<20	Possibly	>0 and <0.3 m (1 foot)	Medium vulnerability
≥20 (Flow Slide Condition)	Yes	None	High Vulnerability

Note:
1 This table applies to intermittently loaded levees. See Section 6.11 in the Guidance Document for information about frequently-loaded levees.
2 Vertical crown displacements can be estimated at 0.7 times the deviatoric deformation from simplified deformation analysis (such as Newmark type displacement), and by adding volumetric displacement as applicable.
3 Based on ULDC criteria for intermittently loaded levees (Section 7.7.1 of the ULDC [Appendix C]).
4 Analyses results must pass each of the criteria in the first three columns of the table at a given vulnerability class (the last column), or the corresponding segment should be rated at the next higher vulnerability class.

© 2016 Deep Foundations Institute

EXAMPLE PROBLEM

The procedure described above will be explained through an example problem from the Walthall Slough Study Area. This problem illustrates an assessment of high vulnerability classification.

The analyzed section is presented in Figure 4 and is within Seismic Segment 3 as shown in Figure 3. Using a mapped peak horizontal acceleration of 0.17g and performing liquefaction analysis, longitudinal extent of liquefaction plot was prepared as shown in Figure 3. Seismic Segment 3 extends from Station 1876+10 to 1890+40 and is located within portions of Reach A and B in the Walthall Slough Study Area. A summary of geometry and liquefaction characteristics of the cross section in Seismic Segment 3 is presented in Table 2.

Table 2: Seismic Segment 3 Cross Section Geometry and Liquefaction Characteristics

Levee geometry	Levee crown width	15 feet
	Approximate waterside slope	3H:1V
	Approximate landside slope	3H:1V
	Approximate levee height	13 feet
Likely Liquefiable Layers	Depth from landside toe	9 feet
	Depth from waterside toe	1 feet
	Thickness	5.5 feet
	Median $(N_1)_{60\text{-}cs}$	7
	Residual undrained strength	Sur: 110 psf

A post-seismic static slope stability analysis was performed at Station 1877+10. Data to identify the liquefiable layer included information from one historic borings and four toe borings advanced for this study. Results of liquefaction analyses show that there is a 5-foot thick likely liquefiable layer with a residual undrained shear strength of about 110 psf.

At this time, static remediation for Reach A and B consists of a partial waterside levee replacement with a cutoff wall. Post-seismic stability analyses were performed on both landside and waterside slopes using circular and wedge search methods. As shown on Figure 5, the minimum post-seismic factors of safety (FS) for waterside and landside slopes are:

- Waterside Post-Seismic FS = 0.79 (wedge) and 0.82 (circular)
- Landside Post-Seismic FS = 1.09 (wedge) and 1.00 (circular)

Since the minimum post-seismic FS is less than 1.0 on the waterside, Segment 3 is classified as “High Vulnerability” with respect to post-seismic flood protection and no seismic deformation analysis or volumetric settlement calculation is needed.

© 2016 Deep Foundations Institute

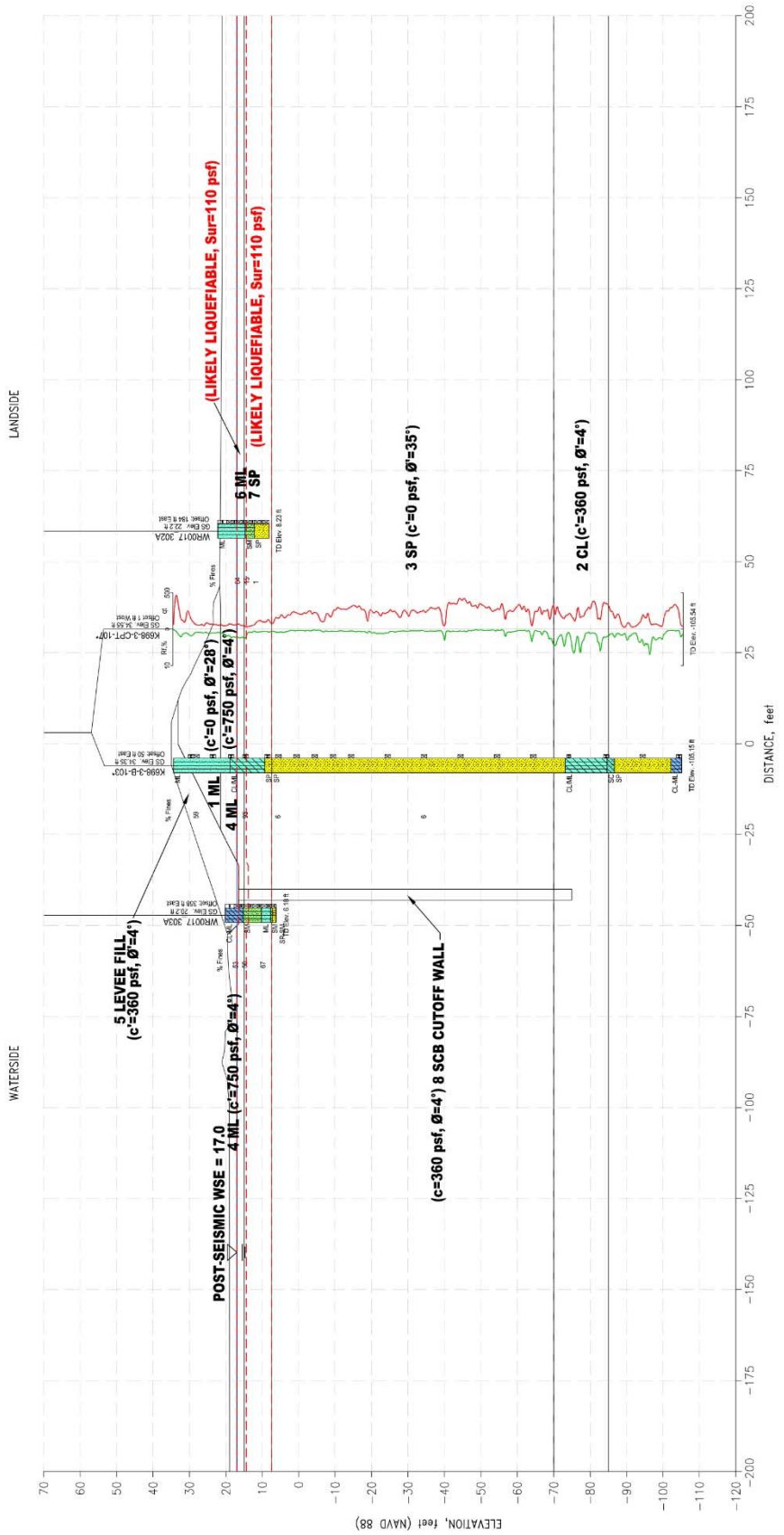


Figure 4: Transverse Cross Section for Seismic Segment 3

© 2016 Deep Foundations Institute

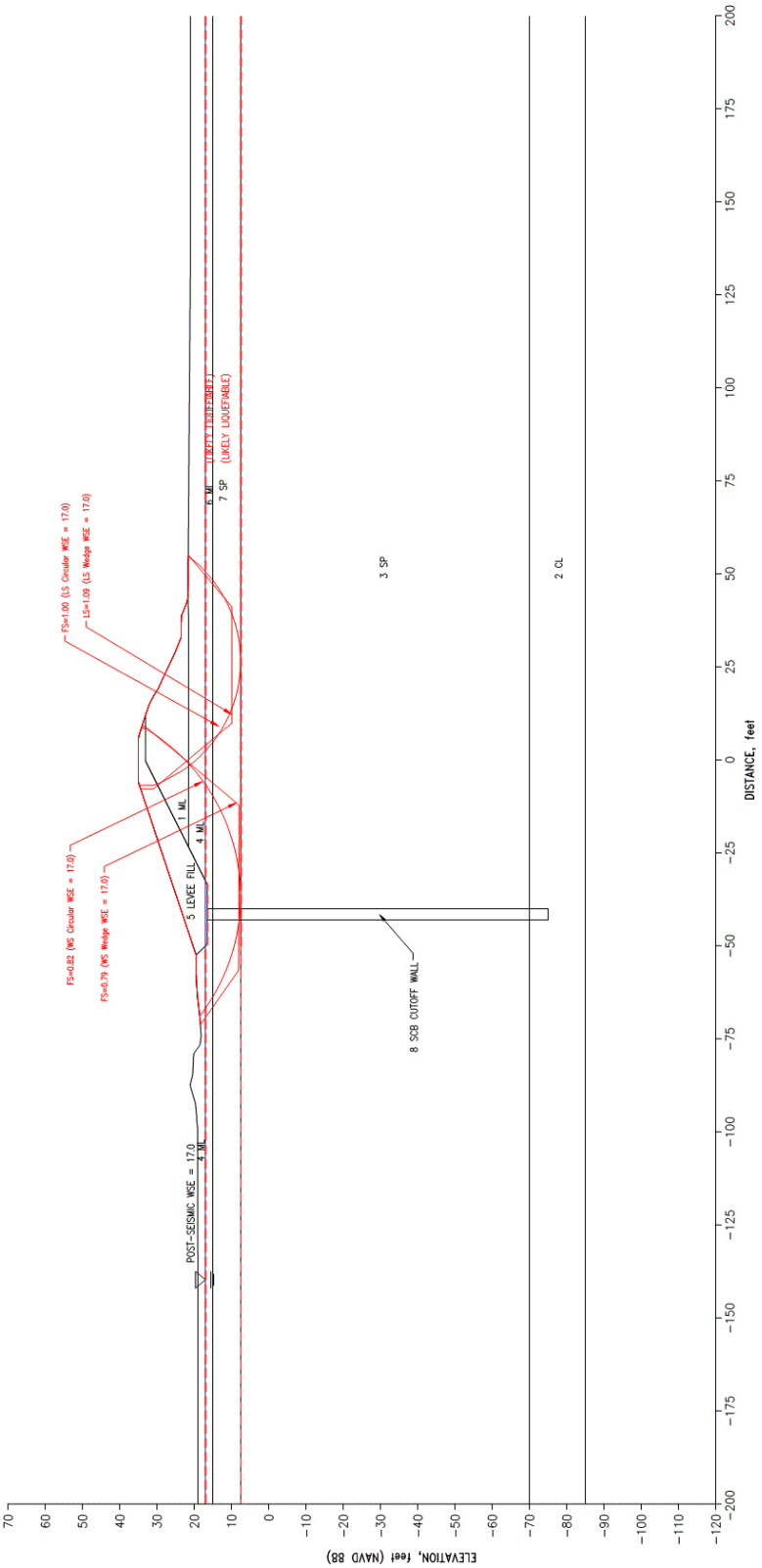


Figure 5: Post Seismic Stability Analysis

© 2016 Deep Foundations Institute

The example problem shows that during a seismic event, this levee will have a flow failure resulting in large deformations. The failure plane on the water side goes through the top portion of the cutoff wall making the levee vulnerable for seepage failure. This requires not only repairing the levee after a seismic event against slope failure but also for seepage protection.

CONCLUSIONS AND DISCUSSION

A step by step method to assess seismic stability of levees is presented which can be used relatively easily for the screening purposes. Estimation of seismic material properties is very important for this kind of analysis. Based on the results of stability analyses, vulnerability classes for the intermittently loaded levees are also defined for the screening purposes. Seismic vulnerability classes of moderate and high vulnerability result not only in failures of levee slopes but also in failures of seepage control measures. This would require remedial measures in terms of both slope stability and seepage control measures.

ACKNOWLEDGEMENTS

The author would like to thanks Steve Mahnke of California Department of Water Resources and Justin Kempton of Kleinfelder for providing valuable assistance for this project.

REFERENCES

Cetin, K. O., Seed, R. B., Der Kiureghian, A., Tokimatsu, K., Harder, L. F., Jr., Kayen, R. E., and Moss, R. E. S. 2004 "Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential." J. Geotech. Geoenviron. Eng., 130(12), 1314–1340.

California Department of Water Resources (DWR). 2012. *Urban Levee Design Criteria*.

Makdisi, F.I. and Seed, R.B. (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *Journal of Geotechnical Engineering Division*, v. 104, p. 849-867, ASCE.
URS Corporation (URS). 2014a. *Guidance Document for Geotechnical Analyses*. Version 14.

© 2016 Deep Foundations Institute

ANALYSIS AND BEHAVIOR OF DOUBLE SHEET PILE COFFERDAM

Bashar S. Qubain, GeoStructures, Inc., King of Prussia, PA, USA, 610-265-1818 bqubain@geostructures.net
Jianchao Li, GeoStructures, Inc., King of Prussia, PA, USA, 610-265-1818, jcli@geostructures.net
Kamil Nuzha, GeoStructures, Inc., King of Prussia, PA, USA, 610-265-1818, knuzha@geostructures.net

ABSTRACT: The Philadelphia Water Department is rehabilitating Belmont Raw Water Basin. As part of this work, a 500-ft long, temporary cofferdam was designed to retain 35 ft of combined water and soil at the western half during the reconstruction of the eastern half. A cofferdam that comprises two integral rows of sheet piles was selected to take advantage of the soil's weight between the sheet piles for increased stability. To fine tune the design, a geotechnical investigation was carried out to verify the subsurface conditions and, in particular, the top of rock variations along the centerline of the cofferdam. A detailed finite element analysis was performed based on the subsurface findings and incorporating the construction sequence to evaluate the behavior of the cofferdam at the following stages: installation of tie rods to connect to the two rows of sheet piles; partial excavation and installation of tiebacks; complete excavation of the east basin, and demolition of the cofferdam. Inclinometers and survey markers were installed every 100 ft along the cofferdam and were monitored throughout construction. The deformation measurements were in good agreement with the finite element results.

Keywords: double row sheet pile, cofferdam, excavation support, tieback anchors.

INTRODUCTION

The Philadelphia Water Department (PWD) Belmont Raw Water Basin is located at the intersection of City Line and Belmont Avenues in Philadelphia, PA. The facility actually contains two raw water basins, east and west, separated by an earthen dividing dike. Water leakage at the bottom of the basins and through the dividing dike together with increasing demand for a larger storage capacity dictated the replacement of the wide earthen dike with new narrower concrete structure. The temporary cofferdam was designed for a two-year period to facilitate excavation support, water cutoff, removal of the existing earthen dike and replacing it with a new concrete dike. During this construction sequence, the water level in the west basin had to be maintained at or near its full capacity.

Various excavation support systems were considered for this project prior to the selection of a double wall sheet pile cofferdam. Fig. 1 shows a plan view of the cofferdam and the new dividing dike. A typical cross section of the two structures is shown in Fig. 2. Double row sheet pile walls are commonly used as both temporary and permanent retaining systems in open water applications such as construction works, quaywalls, breakwaters and revetments for storing industrial waste as indicated by Ohori, et al. (1984), Ohori et al. (1988), and Khan, et al. (2001). There are many advantages to double row sheet pile systems as recognized by Ohori, et al. (1984; 1988): (1) It can be installed and removed in the presence of water easily and quickly; (2) it provides stability with small width to height ratio, utilizing the existing soils between the installed sheet pile rows; and (3) it serves well as a means for water cutoff. These advantages meet the specific project requirements.

An earlier example of a double wall sheet pile cofferdam by the US Army Corps of Engineers in 1908 at Black Rock Lock in Buffalo, NY is documented by O'Bannon (2009). The cofferdam consisted of two parallel rows of sheet piles 30 ft apart with 30 ft spaced transverse sheet piles. Horizontal double channel wales were installed at top of the longitudinal and transverse sheet piles to increase the structure stiffness.

© 2016 Deep Foundations Institute

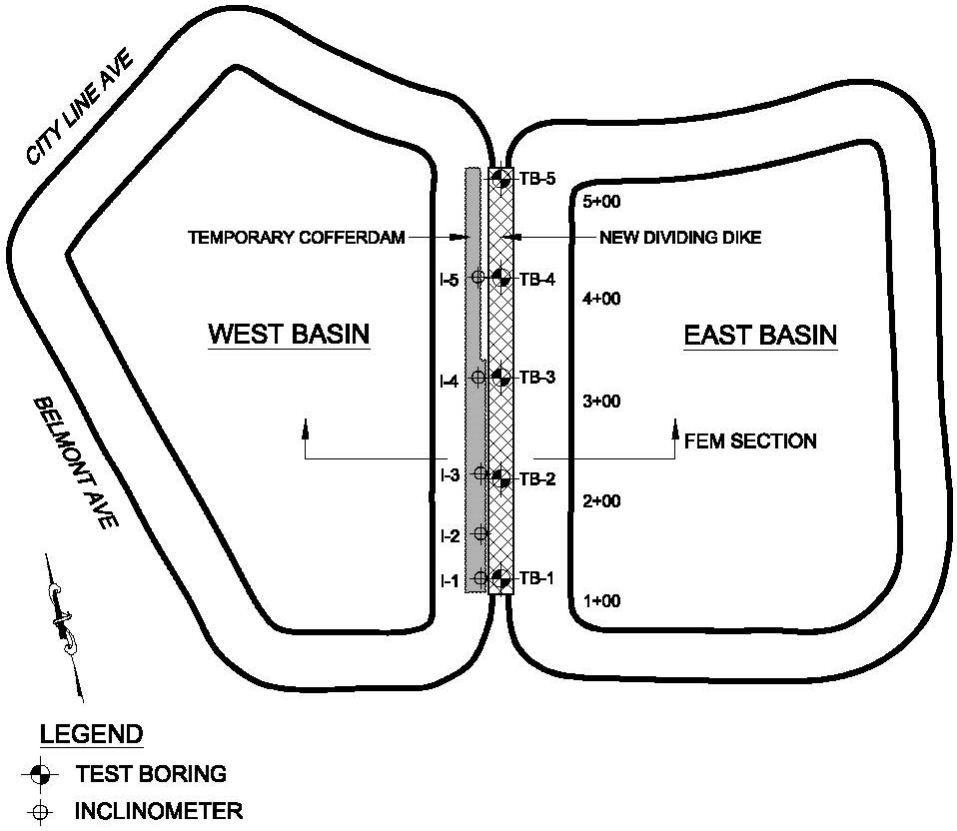


Fig. 1 - Plan showing new dividing dike and temporary cofferdam

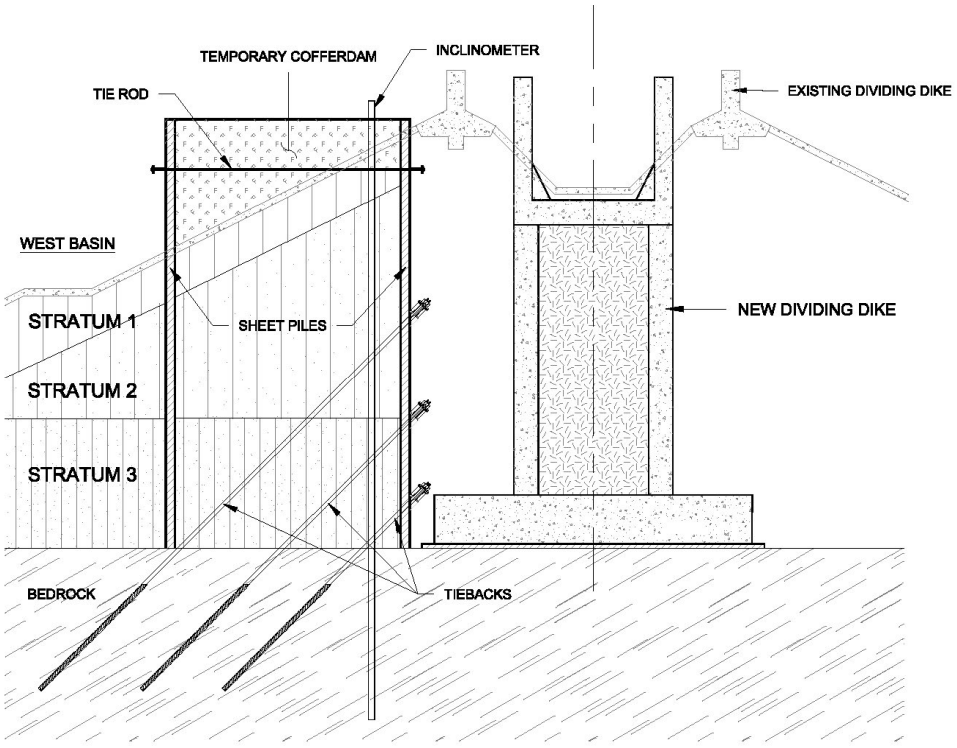


Fig. 2 - Cross section showing existing and new dikes and temporary cofferdam

© 2016 Deep Foundations Institute

This system was recognized by Ohori, et al. (1984) to reduce the sheet pile deflection and provide rigidity. The transverse sheet piles also can be replaced with trussed or internal braces to accomplish the same goal. The US Army Corps of Engineers utilized another type of double sheet pile cofferdam at Montgomery Lock and Dam on the Ohio River in 1932 which featured cross tie rods in addition to the two sheet pile walls (O'Bannon 2009). There is also an engineering design guide published by the US Army Corps of Engineers (1989) for cellular cofferdams. It uses limit equilibrium methodology to evaluate the overall structural stability and size the sheet piles. In addition, the finite element method (FEM) is introduced in the manual as a mean to refine the existing techniques, checking failure modes that cannot be captured by other methods, and determining stresses and deformations.

SUBSURFACE INVESTIGATION AND CHARACTERIZATION

A subsurface investigation was carried out to: (1) gain an understanding of the subsurface conditions and establish soil and rock properties for finite element analysis; and (2) define the top of rock elevation along the cofferdam. The exploration program consisted of 6 test borings as indicated in Fig. 1. The test borings were drilled along the existing dividing dike centerline and were advanced through the overburden soils and 10 ft into underlying bedrock. Laboratory tests were also performed on selected soil and rock samples.

In general, soils in the vicinity of the temporary cofferdam can be grouped into three natural soil strata below a 9 ft thick ballast layer filling the existing dike flume. *Stratum 1* is the first residual soil below the flume. It is wet and described as stiff to very stiff, brown and grayish brown sandy silt (ML). The thickness of this stratum is very limited, measuring only 2 to 6 ft in the test borings. *Stratum 2* is encountered directly below Stratum 1 and consists of loose to medium dense, brown and grayish brown, silty sand with gravel (SM) in moist to wet state. The bottom of this stratum varies from 16 to 28 ft. *Stratum 3* is a completely weathered product or saprolite of the underlying *mica schist* and *granitic gneiss* bedrock. It is described as dense to very dense, brown and reddish brown, silty sand with gravel (SM) in a moist to very moist state. Its thickness varies from 10 to 15 ft. at the south and only 3 ft in the north.

The primary influencing subsurface factor revealed by the test borings is the presence of shallow top of rock across the length of the cofferdam — within a few feet of the bottom of the proposed excavation. This demanded a modified configuration to conventional double row sheet pile cofferdams by incorporating tiebacks to compensate for the lost embedment and provide the necessary stability.

COFFERDAM CONFIGURATION

The cofferdam configuration consisted of PZC sheet piles driven to refusal at the top of rock. The existing soils between the sheet pile rows remained in place and additional ballast and crushed stone were backfilled to raise the cofferdam to elevation 280 ft. One inch diameter all-thread tie rods were installed at 20 ft on center to connect the two sheet pile rows, 4 ft below the top of the cofferdam at elevation 276 ft. The tie rods were pre-tensioned to 40 kip (as discussed later in the analysis results) and the excavation of the dividing dike started at elevation 280 ft and progressed in stages with multiple rows of tiebacks installed at their designated elevations. Tiebacks were drilled at 45° angle and anchored in competent bedrock as shown in Fig. 2. Tiebacks installed at the same elevation were joined by steel channel wales welded to the east row sheet piles to evenly distribute the loading. Each tieback was then tested and pre-tensioned to a design load of approximately 150 kip.

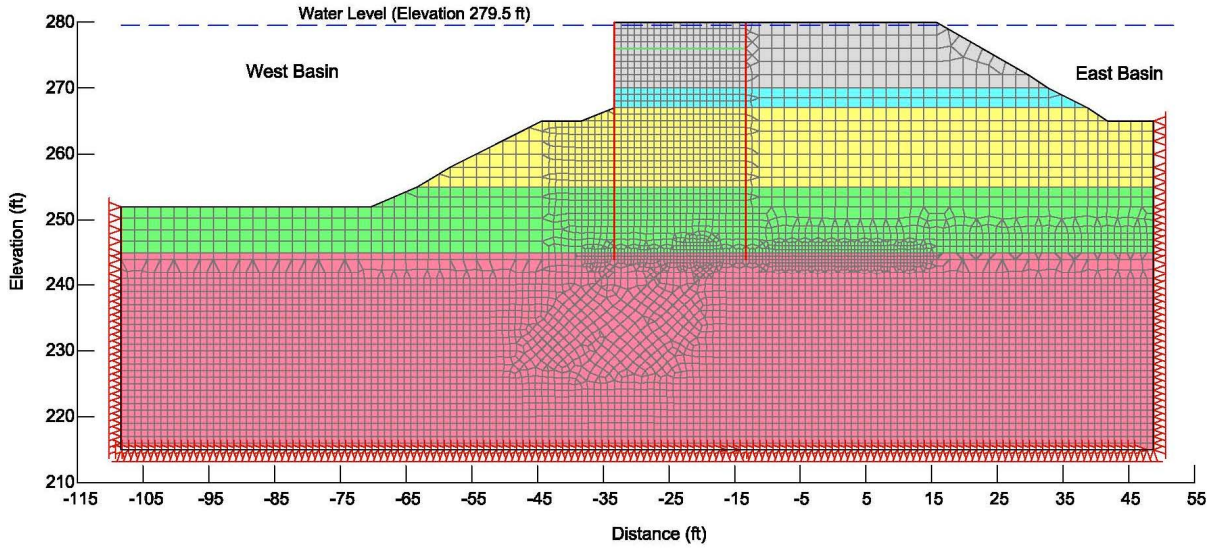
FINITE ELEMENT MODELING

Due to the uniqueness of the cofferdam configuration and the use of tie-rods and tiebacks, there are no widely accepted design guides that can be directly adopted with confidence. The limit equilibrium method that is typically used for design of cofferdams would require many simplifications of the model as well as

© 2016 Deep Foundations Institute

new assumptions, which would make the design unreliable and difficult to evaluate. Therefore, we relied on the finite element method to bridge the gaps in available design methods and to develop a better understanding of the behavior of the cofferdam.

SIGMA/W (Geo-Slope, 2012), a 2-D finite element analysis program was used to model the cofferdam system. The program incorporates both linear elastic and elastoplastic material consecutive models. The sheet piles and the bonded segments of the tiebacks were modeled as *beam elements* while the tie rods and the unbonded segments of the tiebacks were modeled as *bar elements*. In-situ analysis was first performed in order to define the initial state of stresses before construction based on the existing site geometry and water levels in the east and west basins. *Quadrilateral* finite elements were used to model the subsurface soil and rock.



Material Description	Linear Elastic Parameters			Mohr-Coulomb Parameters	
	E (ksf)	ν	γ (pcf)	$\emptyset'$ (°)	c' (ksf)
Fill: Ballast fill and recycled crushed concrete.	1,500	0.3	105	38	0
Stratum 1: Stiff to very stiff, sandy silt (ML).	800	0.3	125	30	0
Stratum 2: Medium dense, silty sand (SM).	1,500	0.3	135	35	0
Stratum 3: Dense to very dense, non-plastic, silty sand with gravel (SM) - saprolite.	2,000	0.3	135	40	0
Highly to completely weathered schist and granitic gneiss	5,000	0.2	150	0	50

Fig. 3 - Finite element mesh and model soil parameters

© 2016 Deep Foundations Institute

The elastic and elastoplastic soil properties shown in Fig. 3 were assigned to each material based on published data in Bowles (1988) and U.S. Navy (1982) for the types of soils and rocks encountered during the subsurface investigation. Load-deformation analysis was then performed considering 6 excavation steps with *elastoplastic* consecutive model assigned incrementally to the soils in the vicinity of the cofferdam. East and west sheet piles along with the tie rod were added to the model first and then each excavation step was followed by installing and pretensioning of a tieback row as shown in Fig. 4. Stress redistribution was performed at the end of load-deformation analysis.

It was recognized at the outset that the analysis and design of the cofferdam components is an iterative process in which the tie rod and tieback pre-tensioning forces play a central role. Higher pre-tensioning yields smaller deflection in the sheet piles but increase the axial forces in the tiebacks and tie rods. Basically, the lower the pre-tensioning, the larger the sheet pile deflection, which ultimately causes system failure. The elevation of tieback rows and tie rods determines the distribution of moment and shear stresses in the east row of sheet piles. The optimization process involved placing the tieback rows at their proper elevations and pretensioning them to a value such that there is no overstressing in any component of the system (sheet piles, tiebacks and tie rods), and the deflection in the sheet piles is maintained *below 1 in*. The shear stresses of the *infill soil* and surrounding elements in the model were checked according to *Morh-Coulomb* failure criterion using the soil parameters in Fig. 3.

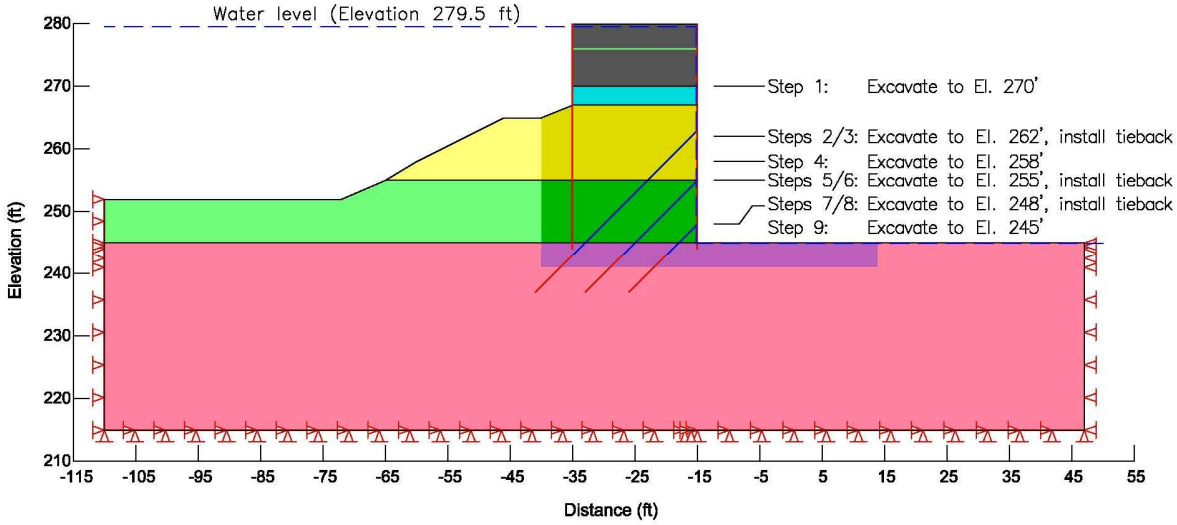


Fig. 4 – FEM model showing construction steps

ANALYSIS RESULTS

The maximum shear stress in the infill soil (1000 psf) as illustrated in Fig. 5 is well below the shear strength of the soil and is indicative of a well behaving model. Fig. 6 presents the axial forces in the three tiebacks at different analysis steps. As shown, the tieback forces narrowly fluctuate between 155 and 160 kips — approximately a 5% increase over the pre-tensioning design load. Fig. 7 depicts the variation in tire rod force above the pre-tensioning design load of 40 kips. The maximum tie rod force is about 46 kips, which represents a 15% increase over the design load.

The bending moment and shear forces in the east row sheet piles are depicted in Fig. 8 and Fig. 9, respectively. The relatively equal distribution of positive and negative moments and shears is certainly indicative that tiebacks are positioned at optimum locations.

© 2016 Deep Foundations Institute

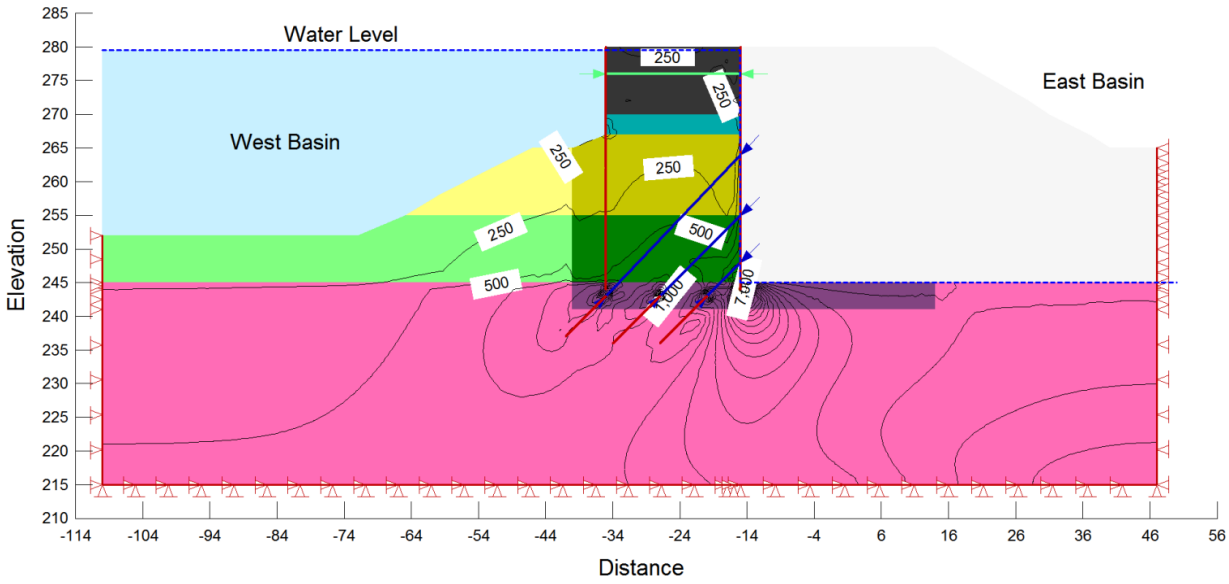


Fig. 5 - Maximum shear stress in (psf)

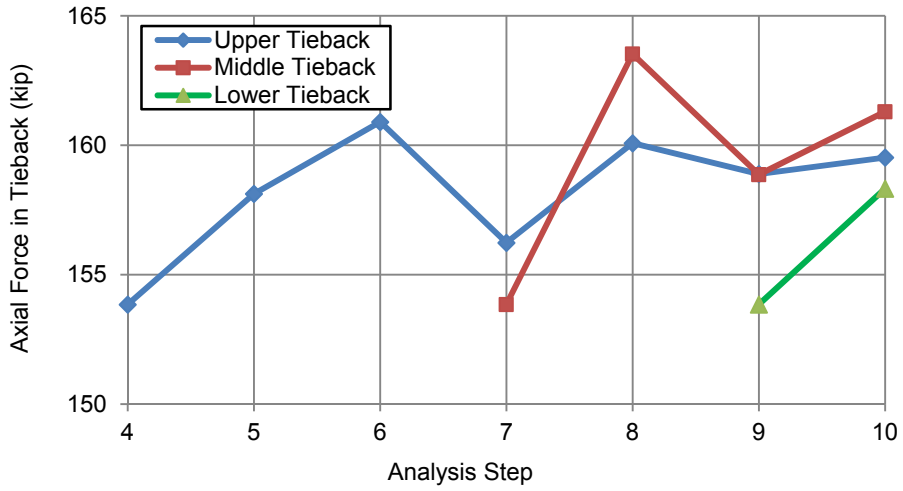


Fig. 6 - Axial force in tiebacks

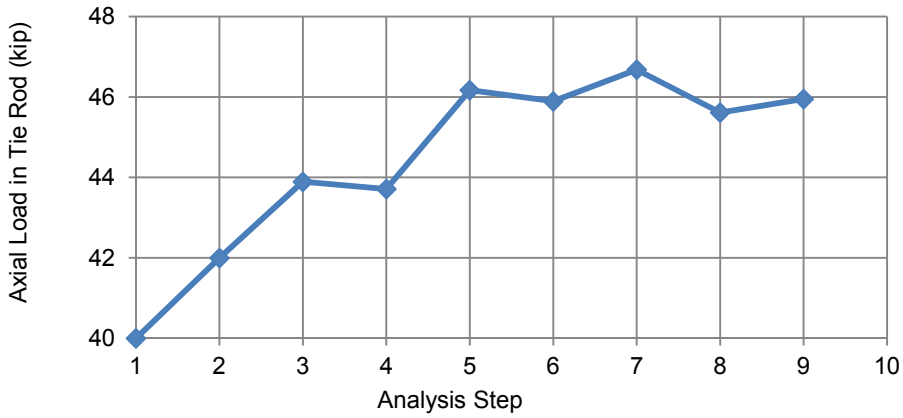


Fig. 7 - Axial force in tie rods

© 2016 Deep Foundations Institute

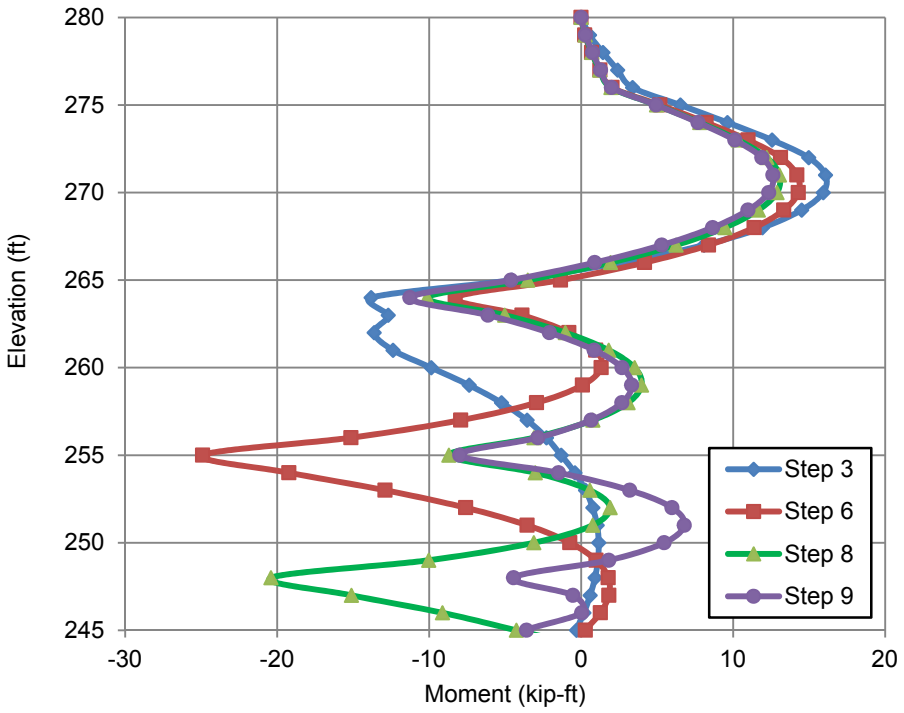


Fig. 8 - Moment diagram of east row sheet piles

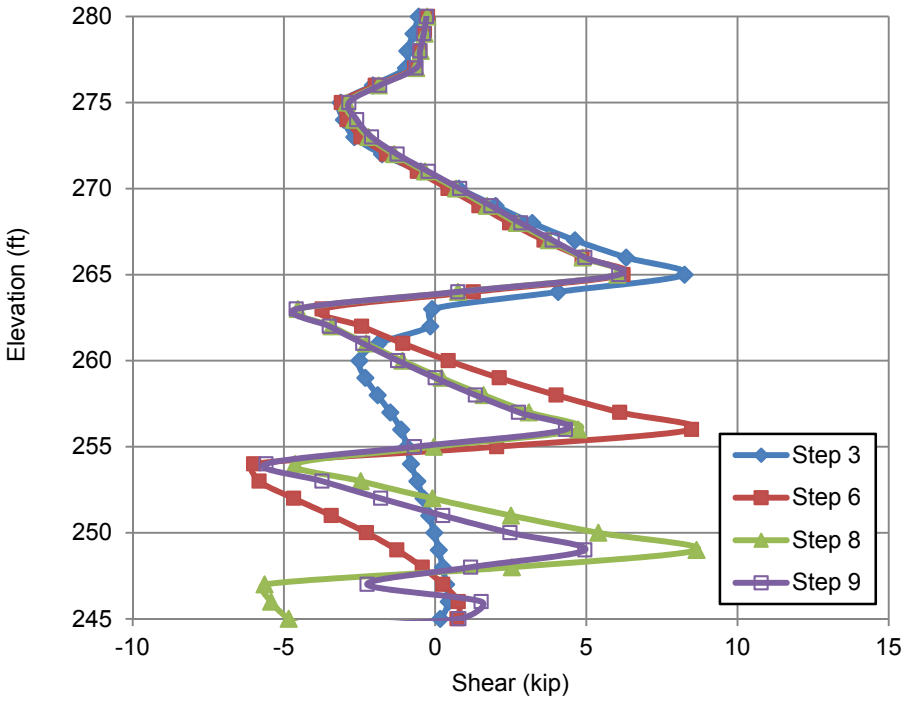


Fig. 9 - Shear diagram of east row sheet piles

© 2016 Deep Foundations Institute

INSTRUMENTATION PROGRAM

At completion of the sheet pile driving and tie rods installation and pretensioning, inclinometers and survey markers were installed every 100 ft along the east row sheet piles as indicated in Fig. 1 to monitor the deflection of the sheet piles as excavation progressed. The inclinometers were drilled through the cofferdam infill soil and 10 ft below the proposed bottom of excavation. Fig. 10 presents a comparison of the deflected shape of the east row sheet piles based on FEM as well as the inclinometer measurements at the end of excavation. The effects of tieback forces are visibly apparent as bends in the sheet pile deflection profile at both inclinometers. Clearly, the inclinometers captured the upper and lower bound deflections in the vicinity of the analyzed section. It is also worth noting that the deflection profile for inclinometer I-2 show negligible lateral movement below elevation 258 ft due to localized high top of rock there. The deflection of the upper part is still consistent in shape with that of I-1.

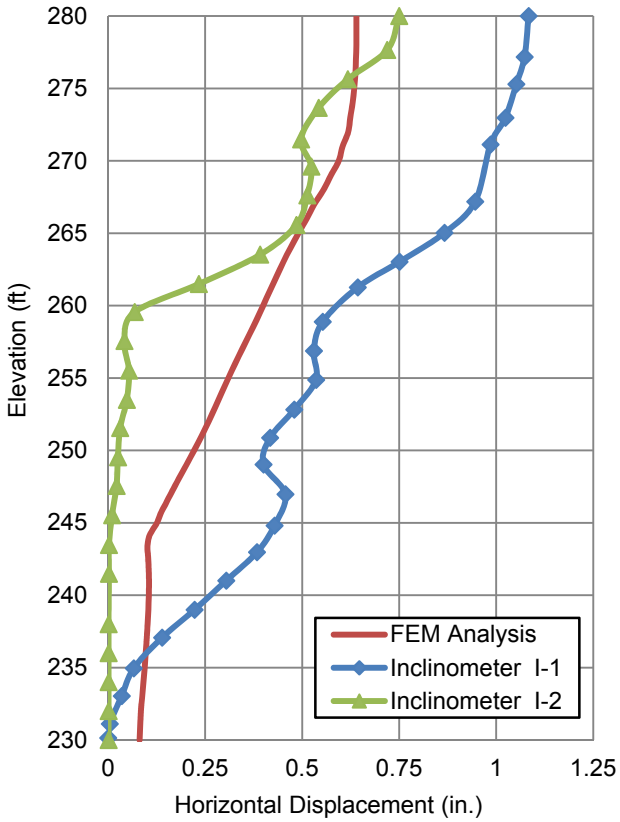


Fig. 10 - Horizontal displacement of east row sheet piles (FEM vs. Inclinometers)

CONCLUSIONS

Due to high top of rock and lack of sheet pile embedment, a hybrid double row sheet pile cofferdam was installed at Belmont Raw Water Basin. This system incorporates conventional system components as well as tiebacks to compensate for the lost embedment and provide the necessary stability. The system was constructed and instrumented using inclinometers and survey markers. Evidently, the deflections of the sheet piles based on FEM were in general agreement with those measured in the inclinometers as illustrated in Fig. 10.

© 2016 Deep Foundations Institute

When embedment is restricted due to shallow rock, a double wall system can significantly improve stability, and provide a superior water cutoff when backfilled with soil or even crushed stone. It can also accommodate heavy equipment on top which is an advantageous during construction to install seepage countermeasures such as grout curtains, if necessary.

The finite element method offers a deeper insight and better understanding of the deformation behavior and interaction between tiebacks, sheetpiles, infill soil and retained earth/water during various excavation and installation stages. Traditional limit equilibrium methods do not offer information about the deformation behavior of the system and may end up with an overly conservative design or, in some cases, undesirable deformations.

ACKNOWLEDGEMENT

GeoStructures, Inc. was retained by JPC Group, Inc. (Construction Manager) to design a temporary cofferdam and monitor its installation and performance. O'Brien & Gere is also acknowledged for valuable technical reviews during the design and construction phases of the project.

REFERENCES

Bowles, J., 1988. Foundation analysis and design. McGraw-Hill.

Geo-Slope International, 2012. Stress-deformation modeling with SIGMA/W 2012, an engineering methodology, version 8.13, Calgary, AB, Canada.a.

Khan, M. R. A., Takemura, J., Fukushima, H., & Kusakabe, O., 2001. Behavior of double sheet pile wall cofferdam on sand observed in centrifuge tests. International Journal of Physical Modeling in Geotechnics, 1(4), 01–16.

O'Bannon, P., 2009. Working in the dry: cofferdams, in-river construction, and the united states army corps of engineers. U.S. Army Corps of Engineers, Pittsburgh District.

Ohori, K., Shoji, Y., Hara, M., Kawai, Y., & Shiota, K. 1984. Elasto-plastic analysis of the double sheet pile wall structure (No. 9) (pp. 79–88). Kawasaki Steel Incorporation. Retrieved from http://www.jfe-steel.co.jp/archives/en/ksc_giho/no.09/e9-079-088.pdf

Ohori, K., Shoji, Y., Ueda, H., Hara, M., Kawai, Y., & Shiota, K., 1984. Static behavior of double sheet-pile wall structures with high rigidity partitions (No. 9), pp. 79–88. Kawasaki Steel Incorporation. Retrieved from http://www.jfe-steel.co.jp/archives/en/ksc_giho/no.09/e9-079-088.pdf

Ohori, K., Takahashi, K., Kawai, Y., & Shiota, K., 1988. Static analysis model for double sheet-pile wall structures. Journal of Geotechnical Engineering, 114(7), 810–825.

U.S. Army Corps of Engineers, 1989. Design of sheet pile cellular structures. EM 1110-2-2503, Department of the Army, Washington, D.C.

U.S. Navy, 1982. Design manual - Soil mechanics, foundation and earth structures, NAVFAC Design Manual DM-7.2, Washington, D.C.

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

CUT-OFFS IN TEMPORARY COFFER DAMS AS SEEPAGE BARRIERS FOR CONCRETE DAM CONSTRUCTION –A CASE STUDY OF TEESTA –V H E PROJECT IN SIKKIM (INDIA)

I D DAYAL, ANIL KUMAR JAIN

EXECUTIVE DIRECTOR (DESIGN & ENGINEERING), NHPC LIMITED
CHIEF ENGINEER, (DESIGN & ENGINEERING), NHPC LIMITED

ABSTRACT

Cutoff walls are frequently used to slow down or prevent seepage of water through and/or around dams. Cutoff wall is a type of seepage barrier structure designed to reduce the flow of water underneath temporary or main dam. There are number of techniques for the construction of cutoff walls including concrete, Jet grouting and permeation grouting. NHPC LTD, a premier Govt. of India undertaking has been instrumental in the hydro-power development within the country & in nearby countries like Nepal & Bhutan since last four decades. NHPC LTD has completed more than 20 hydro-electric projects with total installed capacity of 6587MW (including capacity addition by JV- NHDC). In all these projects, different types of dams and spillways were constructed in not so benign to extreme challenging geological set up. There has always been challenge associated with the temporary diversion works. The project sites sometimes incur huge river bed excavations in case of concrete gravity dam founded on rock foundation at large depth. The challenges for minimizing the seepage have been well taken care by constructing the dedicated seepage barriers. The construction of cut-off walls have progressed from "conventional continuous trench backfilled with conventional concrete or plastic concrete" to "grout curtain", to "jet grouting" to the "concrete cut-off wall" , the last being the most sophisticated & credible seepage barrier.

The efficiency of the cut-off walls are monitored by the seepage quantity observed (reduction of the seepage flow), uplift pressures or exit gradients after construction of cut-off walls. For construction of the main concrete dam, required space is created by the river bed excavation for placing the dam foundation on sound rock. Sometimes there is deep river bed excavation from 30 to 40 m depth or even more as per the available rock profile. Temporary coffer dams are constructed for diverting the river flow through diversion tunnels / channels etc. In deep river bed excavation; there are high chances of the significant seepage through the coffer dam foundation / river bed material due to high hydraulic gradients. Seepage and piping are the main causes of the failure / damage of the temporary coffer dams. If the seepage quantity is too high, then it becomes difficult to handle it by dewatering in the dam pit. Dam concrete is required to be placed unhindered to have a monolithic construction of dam blocks & to have faster construction in the working season. Cut-off walls increase the seepage path considerably thus reducing the exit gradient and further reduction of the seepage thus helping in maintaining the seepage free area for working.

© 2016 Deep Foundations Institute

As the diversion flood values are increasing due to uncertain storm phenomenon, adverse climatic conditions and to have long working period for faster construction of main dam which is resulting in the substantial increase in the height of temporary coffer dams. The suitability of the proper cut-off barrier needs to be designed in accordance with trial testing and credibility as per the available geological condition. Seepage analysis by FEM software helps to find out the efficacy of the seepage barrier in terms of the flux and exit gradient.

1. TEESTA- V PROJECT –A BRIEF INTRODUCTION

The Dam site is located in a narrow gorge near Dikchu where the left bank was having relatively gentle scope of 45º and the right bank was a rocky escarpment made up of competent Phyllitic quartzite and quartzite inter bands. The riverbed comprised of river borne material mainly of boulders of quartzites, gneisses and phylites. The thickness of river borne material was found to be around 40-42 m. A 90 meters high and 180 m long concrete gravity dam consisted of a central spillway abutted on both sides with non-over flow concrete blocks. Fig. 1 shows the general arrangement of dam area & Fig. 2 shows the upstream elevation of dam of Teesta V dam spillway:

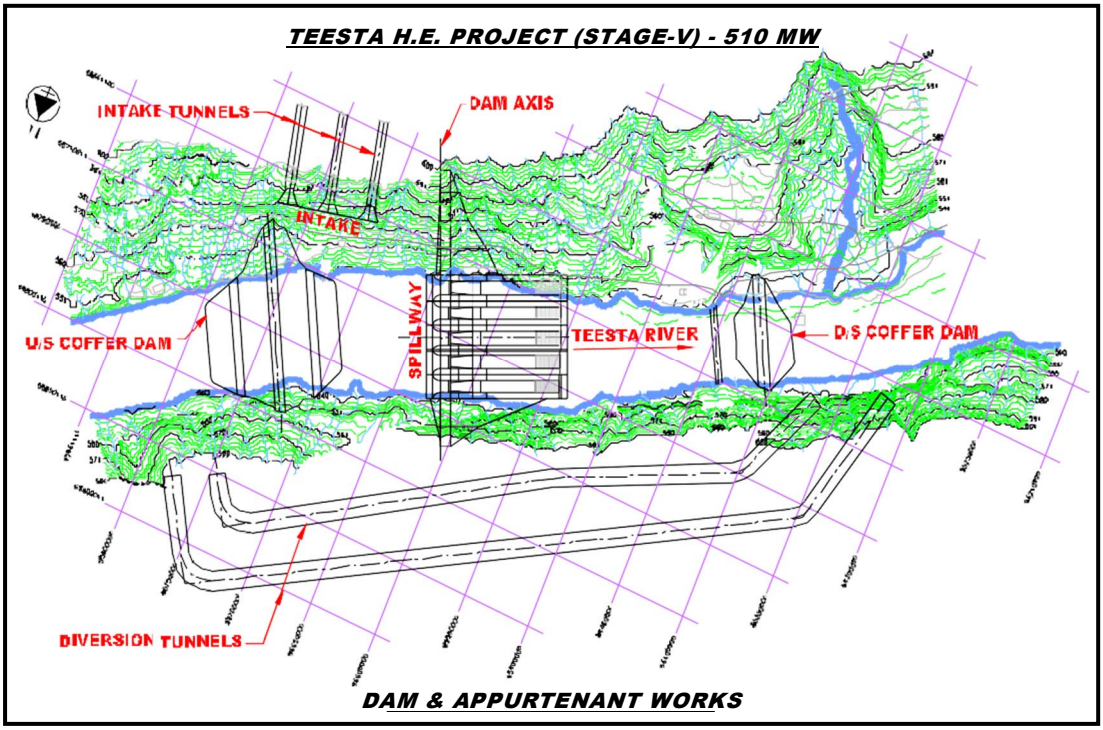


Fig. 1 Showing Dam area layout plan

© 2016 Deep Foundations Institute

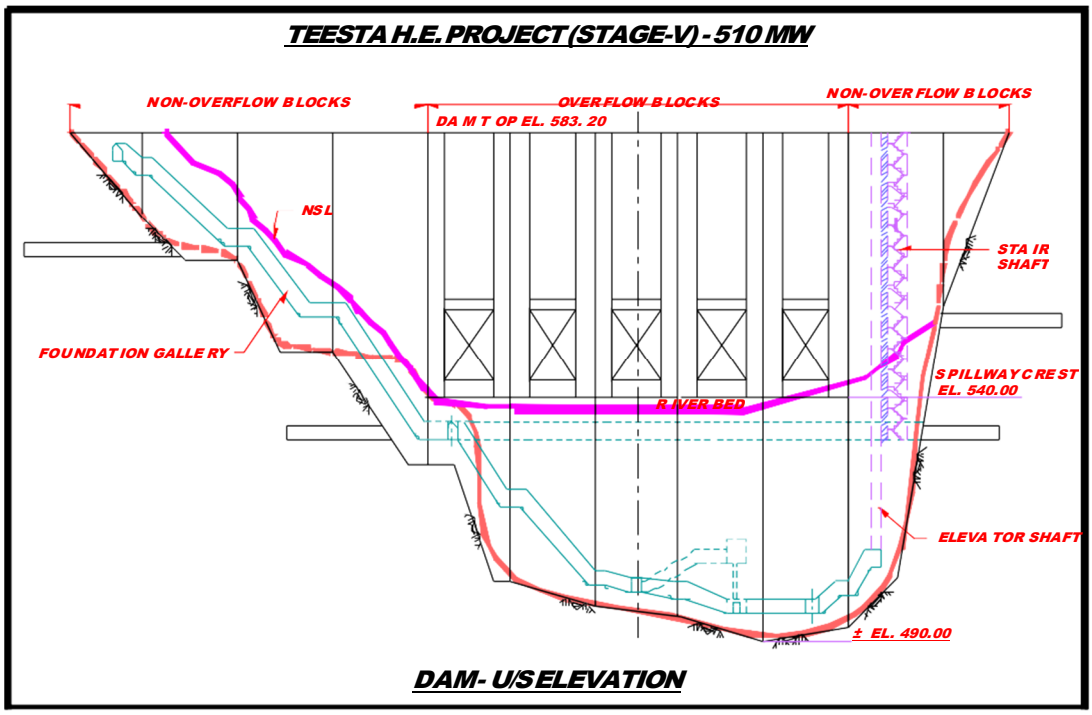


Fig. 2 Showing Dam Upstream Elevation

2. GENERAL FEATURES OF TEESTA DAM & RESERVOIR

HYDROLOGY:

- Catchment area — 4307 sq.km
- Design flood — 9500 m3/s
- Temporary River diversion design flood — 3251 m3/s (1 in 25 year monsoon flood)

DAM & RESERVOIR:

- Maximum reservoir level — EL 580.72 m
- Full reservoir level — EL 579.00 m
- Minimum draw down level — EL 568.00 m
- Gross storage — 13.52 million m3
- Live storage — 6.3 million m3
- Dam top elevation — EL 583.20 m
- Max. height above river bed level (El.531m) — 52.20 m
- Max. height from deepest level — 88.60 m
- Length at dam top — 176.50 m

© 2016 Deep Foundations Institute

SPILLWAY:

- Crest of spillway — EL 540.00 m
- Type — Ogee shaped with gated sluices and flip bucket
- Design flood — 9500 m^3/s
- Number and size of sluices — Five (Each 12.0m high, 9.0m wide)
- Energy dissipation — Flip bucket

DIVERSION TUNNELS:

- Number, shape & dia. — Two , Horse –shoe ,12.2 m
- Length — 473 m & 610 m

3. NEED OF CUT-OFF BARRIERS BELOW COFFER DAMS

The new project sites are posing challenges in terms of adverse geological consideration and other site constraints. Therefore, it is becoming necessary to implement innovative measures for satisfactory performance of the structures. In deep excavations, carried out for construction of the dam, seepage is main concern for having the work space suitable for uninterrupted working. Also, stability of the coffer dam in high head conditions is of utmost importance for the safety of the man and machine working in deep pit. Jet grouting is one such technology that caters to the need for making an impermeable curtain / wall to limit the seepage and hydraulic gradient.

In case of Teesta –V project, concrete dam was founded on the rock at El.±490 m with deep river bed excavation of around 40-42 m below the river bed level (El.±532m). For protecting the dam pit, two coffer dams were constructed on the upstream & downstream to contain the monsoon floods during dam constructions, after diversion tunnels are excavated through the right bank for transferring the river flows.. The cut off was required to be provided beneath the Upstream & Downstream Coffer Dams to facilitate Dam pit excavation inside the two Coffer Dams without relevant water infiltration. The main issues associated with deep dam pit excavation was prevention of seepage in dam pit, construction of coffer dam along with u/s & d/s cut-off in 6 months working period and sliding stability of the steep left bank during construction. Below fig. 3 & 4 shows the view of u/s coffer dam and dam pit excavation.

© 2016 Deep Foundations Institute

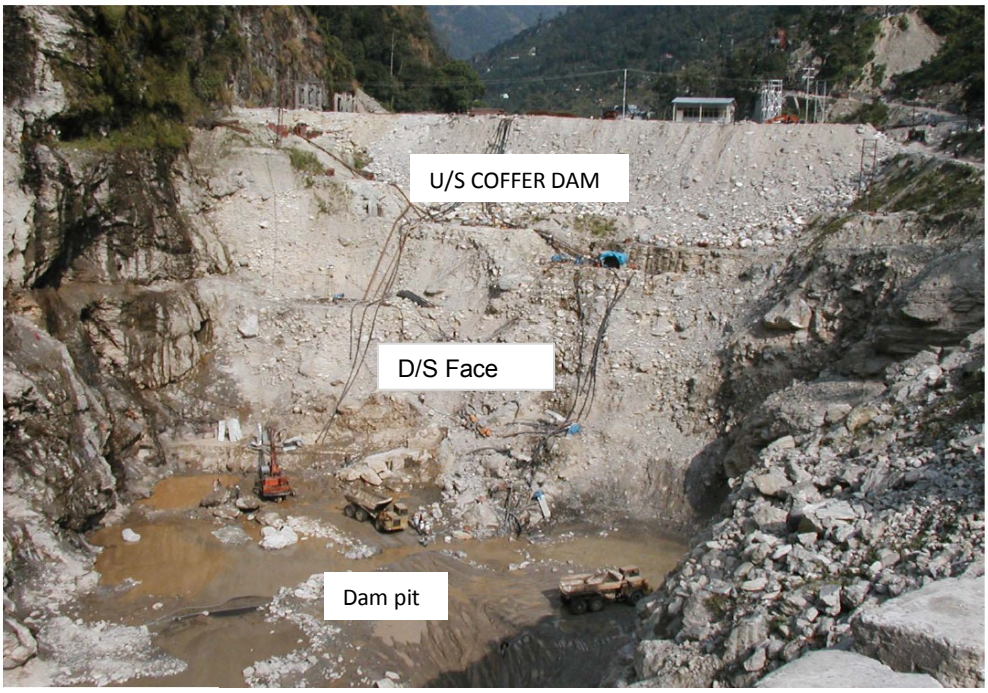


Fig. 3 Picture showing the u/s coffer dam and dam pit



Fig. 4 Picture showing the dam pit excavation

© 2016 Deep Foundations Institute

At the location of the coffer dams, permeable bouldary material was encountered up to 50 m depth and the construction of a jet grouted cut-off wall below each coffer dam was challenging task and timely construction was key feature which could otherwise disturb the schedule of the entire project.

At the upstream coffer dam, the length of the cut-off wall was about 145 m and its area worked out to about 4000 m².At the downstream coffer dam, the length of the cut-off wall was about 120 m and its area worked out to be 2500m².For both cases, the jet grouted cut-off wall was extended to the alluvial / bedrock contact (1 m inside bedrock).

4. JET GROUTED CUT-OFF

4.1 GENERAL

Jet grouting has increasingly become one of the acceptable ground improvement technologies used to address seepage concerns and to provide strength improvements of soils. Use of jet grouted walls as a barrier to water flow under dams has been used increasingly in recent times. If installed correctly, jet grout columns may provide a continuous wall of relatively impermeable material. However, the results and effectiveness of jet grout construction can be difficult to achieve and access. Field trials to establish jet grout parameters, column diameter measurements, energy correlations and assessment of quality of treated soil including inclusion of untreated soil within the column are required. This method has been attempted in many parts of the world successfully for depth up to 50 m.

Jet grouting uses a high energy jet of fluid to erode a soil and mix it with cement grout. In the single jet grouting system, erosion and soil cementation are achieved with the same fluid (cement grout) forming a jet stream when directed at the ground (one fluid employed, grout). In the double jet grouting system, an air jet envelops the grout jet for improving the grout jet stream erosive efficiency (two fluids employed, grout and air). In the triple jet grouting system, erosion is achieved with a double jet of water surrounded by an air jet and cementation is simultaneously obtained by a separate grout jet (three fluids employed, water and air and grout).

A jet grouted column corresponds to the volume of soil eroded and cemented from a single borehole. When rotating and simultaneously translating the erosive jet stream in the borehole with constant speeds, a column is formed having the shape of a cylinder in homogeneous soil. In heterogeneous ground, with obstacles, the jet grouted column obtained has an irregular shape. The column diameter varies all along the borehole as a function of the resistance opposed by the soil to jet erosion and/or as a function of the parameters of execution (jet grouting parameters).The jet grouted material is the soil-cement mixture left in the ground by the process, and constituting the body of a jet grouted element after setting. When jetting in sandy soils, the jet grouted material is like

© 2016 Deep Foundations Institute

a cement mortar. Fig. 5 shows general single row jet grouting column plan at top and bottom showing drilling deviation.

During construction of a jet grouted column, a mixture of soil and injected fluid arises at hole collar via the annulus of the jetting borehole. This spoil return normally flows on the working platform as long as fluids are sent into the ground for constructing the jet grouted column. This is called as spoil return.

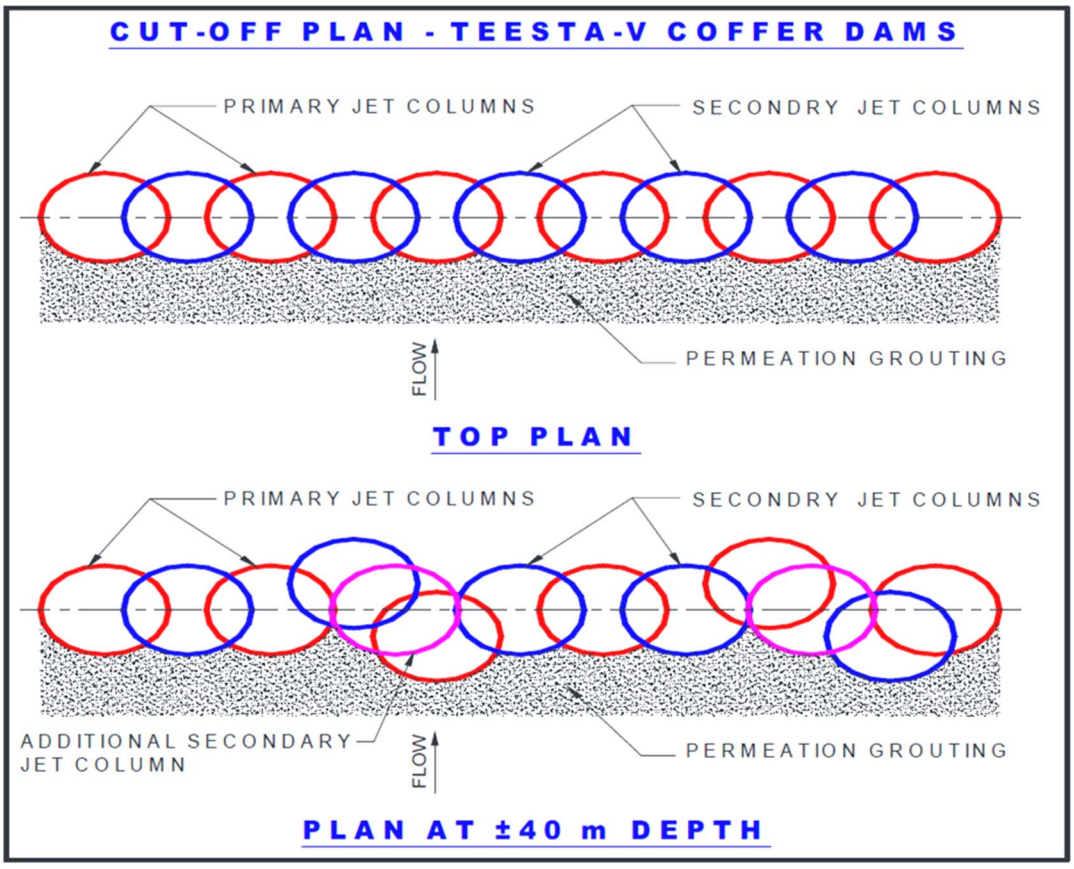


Fig. 5 showing jet grouted column plan

4.2 JET GROUTING METHODS

There are three Jet Grouting methods. Selection of the most appropriate method is generally determined by the soil conditions, the geometry of the element to be constructed and the application:

Single fluid Jet grouting

The grout directly fulfils three distinctive functions of breaking down the soil matrix, removal of excess material and mixing with the soil.

© 2016 Deep Foundations Institute

Double fluid jet grouting

A two-phase internal fluid system with a coaxial air-jet supply line around the grout-jet supply in order to increase the efficiency of the mixing process and range of action of the slurry. Air-jetting also improves, due to an air-lifting effect, the extraction of the spoils along the drilling rod.

Triple fluid jet grouting

Coaxial air and high-velocity water are used to erode the soil structure with an additional improvement through partial substitution of the finest soil particles. Cement grout is then injected through independent nozzles located below the water/air jets.

5. CHOOSING JET GROUTED CUT-OFF IN TEESTA- V PROJECT

Jet Grouting cut offs as impervious barriers were selected in combination with permeation grouting as it provides fast and efficient technique to bring the seepage within the manageable limits. The other methods of providing impervious barriers like diaphragm wall and concrete cut off could have taken longer time due to presence of large boulders and conventional grout curtain could not have provided as impervious barriers as desired for the job. As there was approx. 60 m head difference between the dam pit and coffer dam upstream water level & coffer dams were founded on river bed material, it was apprehended that it may cause a lot of seepage in the dam pit area and stability of coffer dam may be endangered. Jet grouting was selected due to its advantages in present site conditions and fast, flexible & efficient technique for improving the soil characteristics & impermeability & less sensitive to big boulders as compared to concrete cut-offs .

For the present case of application, the cut-off wall composed of a series of interlocking columns of jet grouted material within the alluvium. It was ensured that the jet grouting process do not have any critical windows amongst the columns (windows caused by boulders for instance), the jet grouted cut-off walls were complemented by permeation grout curtain and conventional grouting through rock at the foundation below jet grouting was made.

At Teesta-V dam site, each hole was first drilled using casing and down the hole drilling techniques to ensure the best verticality and advancement rate through boulders , in what is called the pre-drilling phase. The casing was then retrieved and the hole filled with stabilizing mix. The hole was re-drilled to full depth using the jet grouting string.

Various methods of providing impermeable curtains like jet grouted cut-off, diaphragm wall, permeation grouting, concrete cut-off walls etc. were available for consideration.

© 2016 Deep Foundations Institute

All the pros and cons were analyzed & finally jet grouted cut-off was chosen as seepage barrier. The feature which made Teesta H.E. Project Stage – V such an important works is that this was the first Project in the country having an impermeable cut off made of Jet Grouting method. Other important considerations for choosing jet grouted cut-off wall apart from providing an efficient technical solution were strict adherence to construction schedule for concrete dam, most challenging task to complete the cut-offs and coffer dams in one non-monsoon season of 6 months.

6. FACTORS CONSIDERED FOR SUCCESFULL COMPLETION OF JET GROUTING IN TEESTA –V PROJECT

6.1. SUBSOIL CONDITION & GEOLOGY

The ground to be treated by jet grouting was composed of permeable fine dense sand containing blocks and boulders of fresh rocks of various size and metamorphic nature. The bedrock itself was mainly composed of quartzo-phyllite and noticeable fissure permeability. In general terms, boulders or blocks of every size were recognized in the investigation drillings (with pieces up to 3.5 m) at every location between the river bed and the bedrock. As per statistical analysis of the boulder / block frequency in the cores, 18% of the coring lengths were pertaining to boulders or blocks. The majority of boulders/ block pieces were considered to be 60 cm long, only some core pieces were longer (these pieces represent about 45% of the boulder / block lengths).

The geological investigation gave the values of UCS ranging from 70 to 120 MPa for the quartzo-phyllite bedrock material. However, some tests gave lower results in the order of 20 MPa.

The alluvium matrix between boulders (that is composed of fine dense sand) was not of very high permeability, as no critical loss of water experienced during core drillings. As per sieve analysis of the sandy matrix, a fine to medium sand (d50 about 0.3 mm) was considered with very little silty (less than 5%) and no gravel.

The permeability of this type of sand was expected to be in the range of 2e-5 m/s to 2e-4 m/s in accordance with results of head water loss tests performed in the exploratory drillings.(these water test also demonstrate the absence of high permeability zones throughout the alluvium).Additional exploratory holes (6nos.) were carried out to precisely confirm the nature and permeability of the soil strata along the alignment of the coffer dams to ensure the required permeability of the jet grouted cut-off walls i.e. 1e -6 m/s. Existing permeability of the overburden material was of the order of 2 e-5 m/s to 2e-4 m/s which was improved to 1e-6 m/s with jet grouted cut-off. Instead of multiple row jet grouting which could have taken longer time, single row jet grouting along with permeation grouting was adopted for coffer dams.

© 2016 Deep Foundations Institute

6.2 DOUBLE JET GROUTING SYSTEM

Double Jet System was recommended in the Contract Technical specifications & the same was employed for jet grouting. The jetting / drilling string of tools used for double jet grout +air composed of:

- Double injection head swivel to which the air and grout delivery hoses are connected.
- String of 90 mm double rods designed with two coaxial passages, (the inner for the grout, the outer for the air)
- Double monitor placed at the base of the rods, bearing one or two sets of nozzle through which grout and air will flow out.

Each set of nozzles was specially designed in order to encapsulate the grout jet by an air jet. By this process, the grout jetting erosion power was further maintained by an air envelope into the ground and the soil was removed at greater distances. Another effect of employing air was to assist the lifting of the jet grouted material up to the surface via the annulus between the drill hole and the jet rods during the jetting operation.

6.3 GROUT TYPE

The grout was composed of cement and water mixed together, plus (possibly) bentonite addition. The technical specification for a 28 days UCS of the jet grouted material comprised between 0.5MPa and 3MPa.

7. SEQUENCE OF JET GROUTING CONSTRUCTION AT TEESTA- V SITE

7.1 TRIAL JET GROUTING AND PROCESS PARAMETERS

Before taking up the main jet grouting works, trial jet grouting was done at site to finalize grouting parameters. The trial site was located 5m u/s of upstream jet grouted cut off wall on left bank of river at EL 541.0m. Cut-off wall was 12.5m u/s of coffer dam Axis. The trials were done to assess the following:-

- Test the drilling, deviation measurement and jet grouting equipment.

- Fixing the basic design parameters like input energy, flow rate of grout, Grout pressure, Uplift speed, Rotation speed and jet column diameter.

- Establish the construction procedure.

- A trial shaft was made to examine the treatment from inside, window formation, if any and measure the permeability.
- Initially four nos of individual columns were constructed as per the jet grouting parameters given below in Table.1

© 2016 Deep Foundations Institute

TABLE 1.

Column	1	2	3	4
Grout pressure (MPa)	40	40	40	40
Grout flow (Lpm)	225	225	225	225
Air pressure (Mpa)	0.7	0.7	0.7	0.7
Jet nozzle diam. (mm)	4.5	4.5	4.5	4.5
Uplift speed cm/min.	18	11.30	8.2	5.5
Energy (MJ/m)	50	80	110	165

TRIAL SHAFT

Based on this trial, the trial shaft comprising of 9 primary columns @ 2.4mc/c and 9 secondary columns in between two primary columns forming an annulus were constructed on the left bank. Jet grouting was done in all the holes with cement bentonite solution of Cement-500Kg, Bentonite 25Kg, water-825L and having density of grout being 1.36Kg/m3. After this, permeation grouting was done by making 22 holes at 1.2m away from trial shaft. After this, two pump wells, one inside the shaft and the other outside the shaft was done and then pumping test was done to ascertain the permeability of the grouted cut-off wall which was found to be less than the minimum requirement of 1X10-6m/s.Fig.6 & 7 shows the trail shaft details.

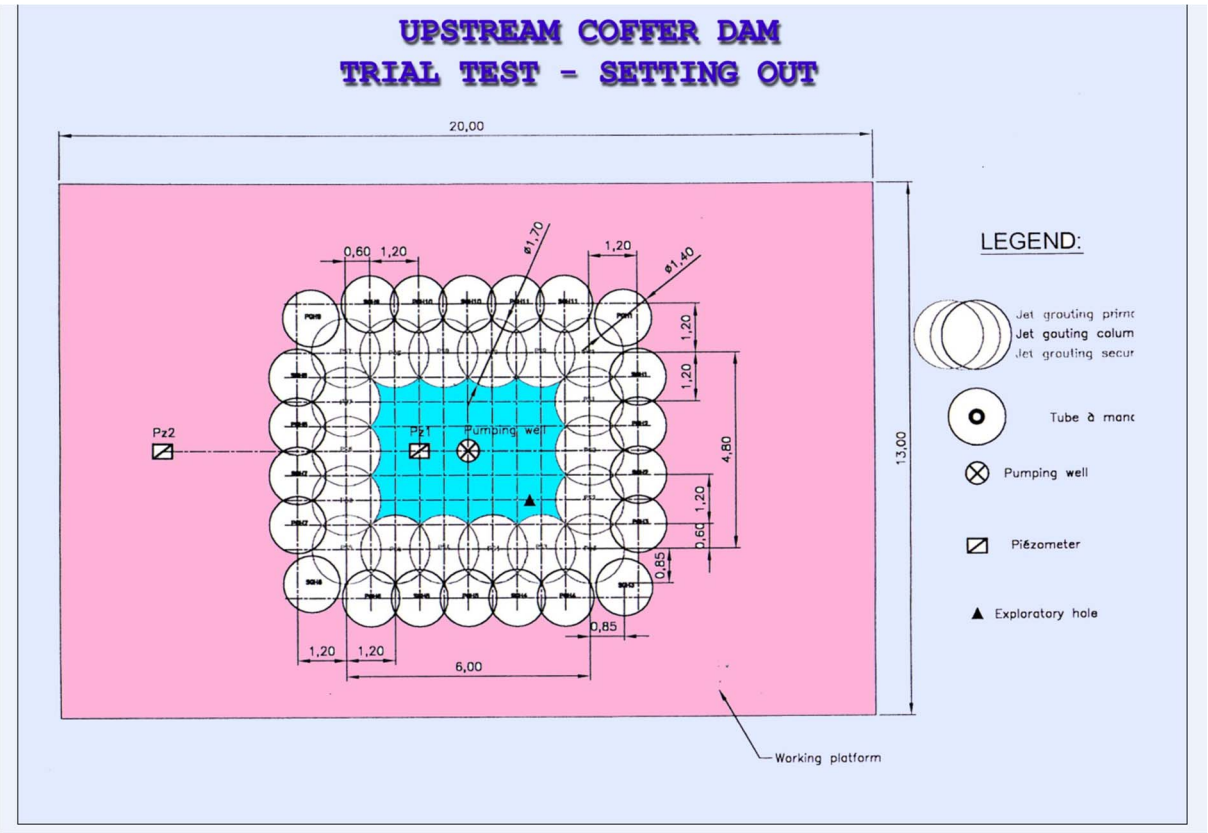


Fig.6 showing trial test plan

© 2016 Deep Foundations Institute

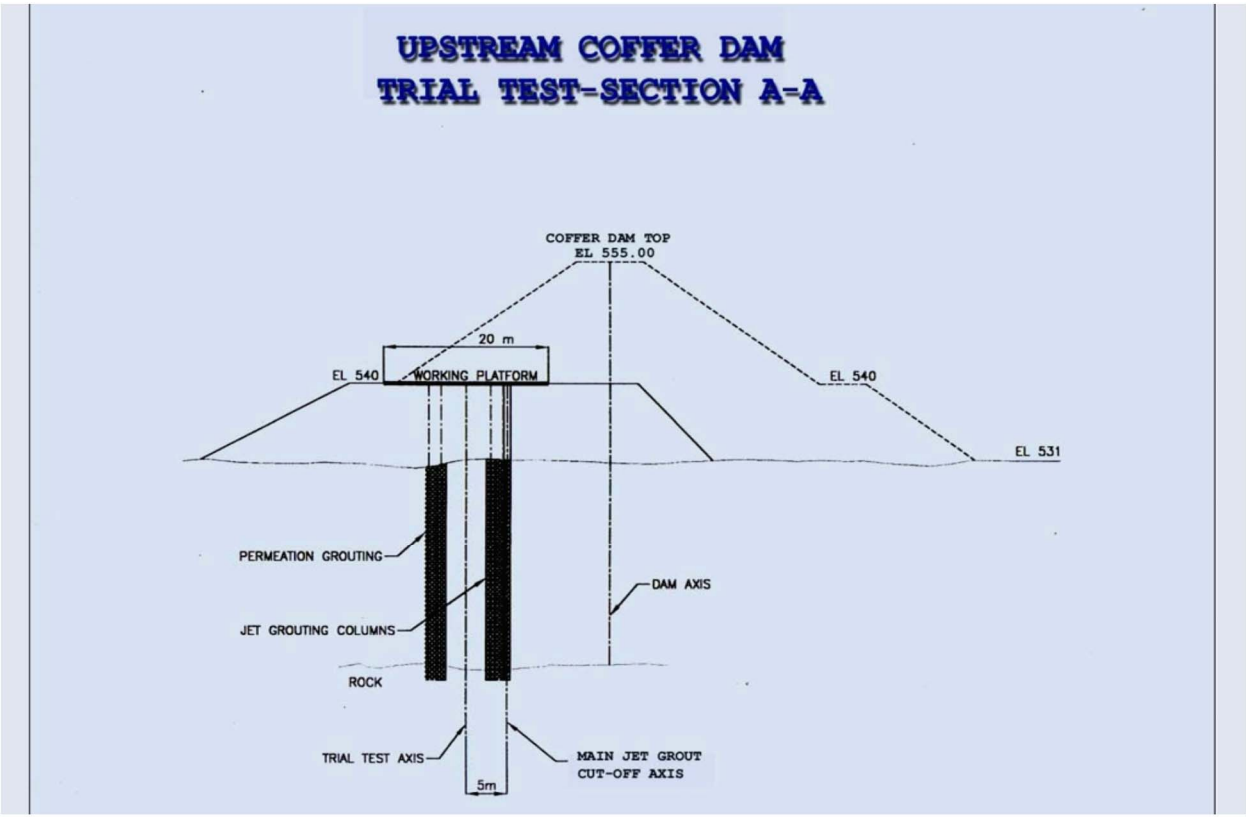


Fig.7 showing trial test section

7.2 BASIC DESIGN PARAMETERS OF JET COLUMNS FOR TEESTA- V

After the trial, following technical parameters were finalized for jet grouting:

- Types of jet grouting– Single row-double fluid technique
- Maximum cut off wall depth: 40 m
- Design Head: 60 m
- Dia. of Jet columns …… Minimum 120 cm
- Minimum overlap of 10 cm for adjacent columns along the line
- **Normal ranges for double fluid jet grouting parameters are :**
- Jet grout pressure …….. 35 to 45 MPa
- Grout flow …. 150 to 300 lpm
- Air Jet Pressure …….. 1 to 1.5 MPa
- Air Flow …….. 3 to 12 m^3/min
- Rotation Speed …….. 8 to 35 rpm
- Uplift Speed … 3 to 20cm /min
- Total Specific Energy …. 30 to 200 MJ/m
- Permissible Average drilling deviation verticality … 1.5 %
- Maximum drilling deviation in verticality ……2.5%
- Permeability after jet grouting ……$1x10^{-6}$ m/s or better

© 2016 Deep Foundations Institute

7.3 MAIN JET GROUTED CUT-OFF

The specialized agency was given contract for construction of the jet grouted cut-off. The agency mobilized jet rigs and other equipments for main cut off in the month of November 2002. The sequence of work activities undertaken at site is given below:

i. First Stage upstream coffer dam construction up to EL 539 m followed by jet grouting platform construction from RD 0.00-30 m up to EL 540 m.

ii. Marking of centerline of cut off wall on individual primary column locations which was kept 12.5m upstream of centerline of coffer dam.

iii. Drilling of 200 dia. holes by roto-percussion method

iv. Measurement of inclination before withdrawal of casing.

v. Stabilization of drill holes with cement and bentonite grout and simultaneous withdrawal of casing.

vi. Re-drilling of holes up to final depth with jet grouting string having tricone drill bit.

vii. Jet grouting from bottom of the hole through jet nozzles at pre-designed uplift speed and energy.

The process was repeated for secondary column-pre-drilling and jet-grouting. After completion of jet grouting in cut off, permeation grouting holes were drilled at 1.2m upstream of jet grouting cut off axis. Permeation grouting is a grouting process in which grout was placed under pressure without appreciable damage to the soil structure or triggering soil displacement. It was carried out using T.A.M.(Tube-a-Manchette) technique. It was done on the upstream side of the jet grouted cut off. As compared to jet grouting, it was a low pressure grouting up to 5Mpa. It was mainly used for window treatment as the chance of window formation could happen in such boulder matrix. The details of the jet grouting are summarized below in fig. 8 & Table2.

© 2016 Deep Foundations Institute

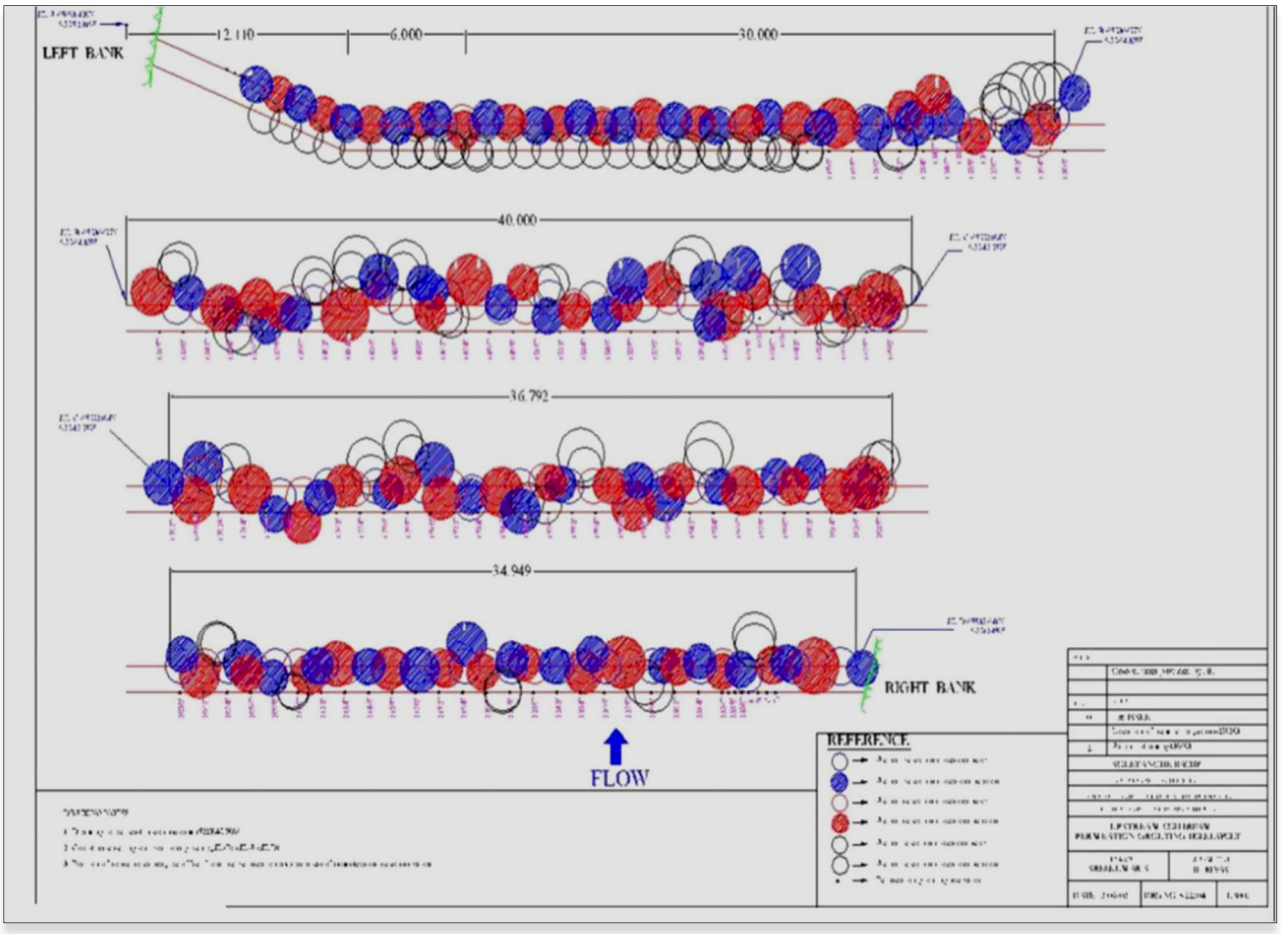


Fig.8 showing constructed jet grouted column plan at Teesta V site

TABLE2. ABSTRACT OF JET GROUTING

Sr. No.		Upstream coffer dam	Downstream coffer dam
1.	Total length	161 m	159 m
2.	Total area of treatment	4900 m²	3700 m ²
3.	Total no. of holes	184	142
4.	Total cement consumed	117376 bags	93437 bags
5.	Bentonite	293440 kgs	233592 kgs.
6.	Sodium Silicate	98000 ltrs	98000 ltrs

© 2016 Deep Foundations Institute

8. PLANING OF CONSTRUCTION OF COFFER DAMS AND CUT-OFF

In a narrow valley, to create working space for construction activities, Teesta River was diverted through two no. concrete lined horse-shoe shaped diversion tunnels of diameter 12.2 m on the right bank. These diversion tunnels were kept operational till the completion of dam and erection of radial gates. Afterwards, 2 sets of closure gates was kept for blocking the tunnels for construction of permanent concrete plugs in the diversion tunnels.

The monsoon in Sikkim sets in early May and lasts up to late October, the construction of coffer dam in all respect was to be completed in 6 months i.e. from Ist Nov. to 30th April. Therefore coffer dam design was made to take care of, apart from slope stability & seepage control, construction methodology so that the coffer dam can come up within 6 months.

The diversion scheme including the coffer dam was planned to be designed for maximum monsoon flood of 1 in 25 year frequency which is 3251 cumecs.For diversion and construction of main dam, Upstream coffer dam having 32 m height above river bed level and downstream coffer dam having height of 21 m above river bed level was designed & constructed for keeping the working space available during the entire year for construction of main concrete gravity dam. Fig.9 & 10 shows the cross section of the u/s coffer dam.

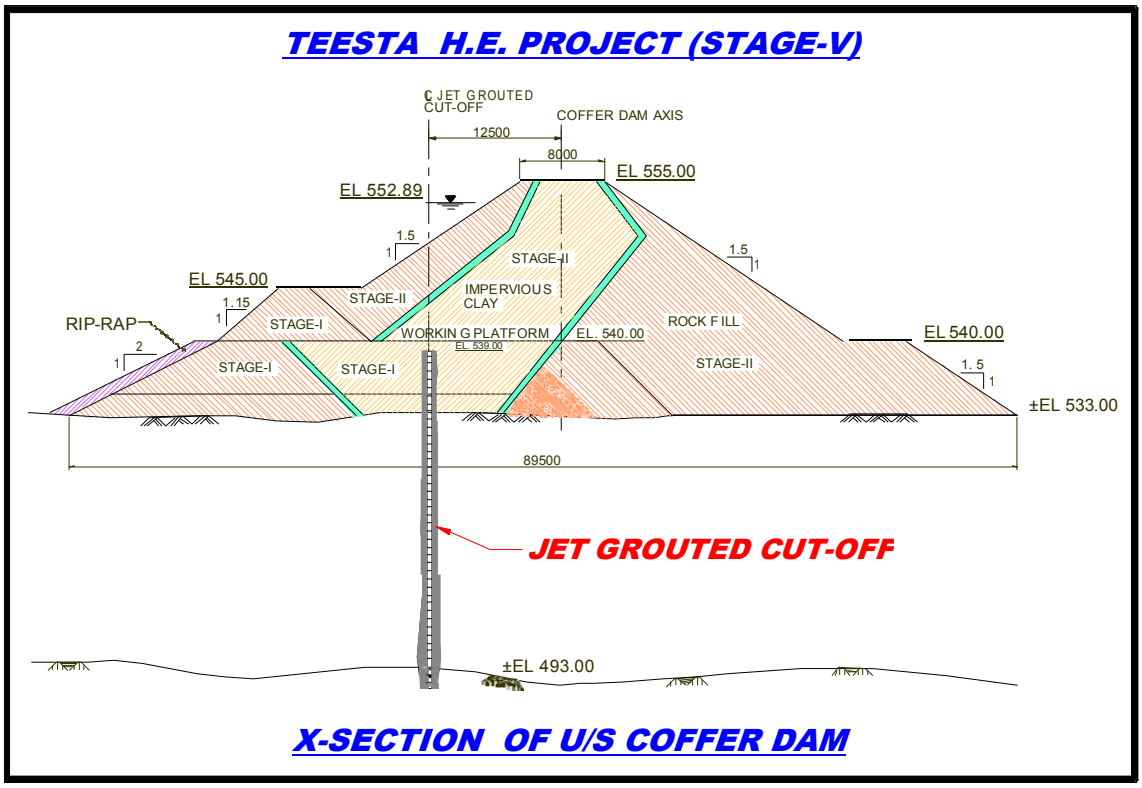


Fig. 9: U/S Coffer Dam Section

© 2016 Deep Foundations Institute

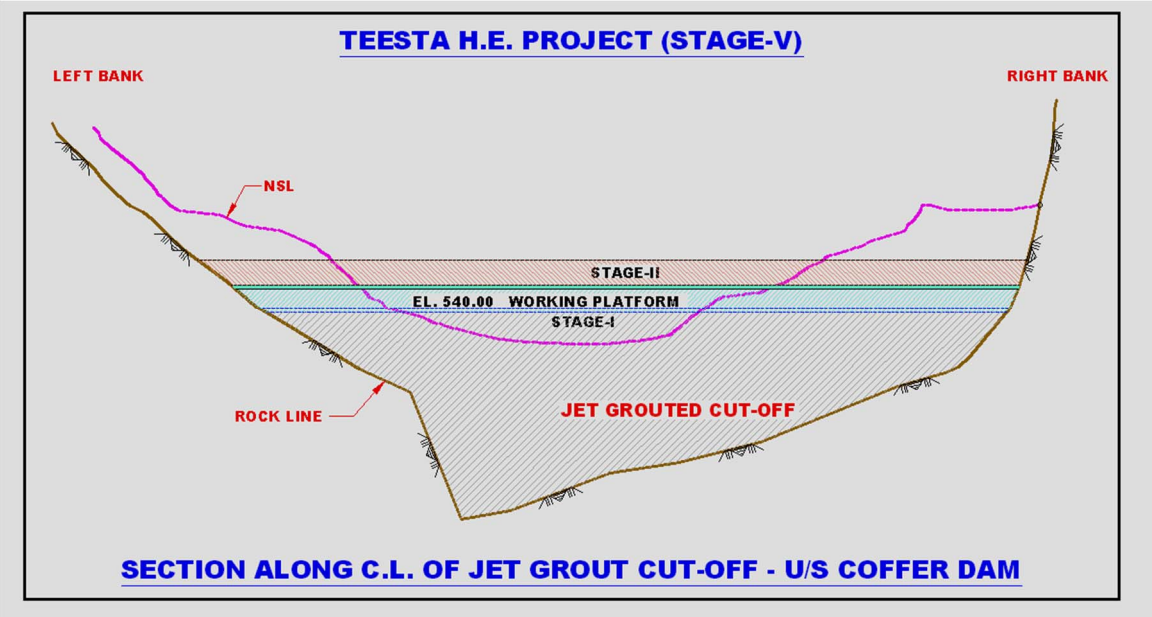


Fig. 10 U/S Coffer section with Jet granted cut off Elevation

The timing of construction of coffer dam & jet grout curtain within available working season of 6 months required meticulous planning of design & construction activities, conducting trials, phase -I coffer dam construction, construction of jet grout curtain & completion of stage-II coffer dam & start of dam pit excavation.

a.) Diversion of the river was done in Ist week of Nov. only to take optimum advantage of the construction period. The average discharge in the river during Ist week of Nov. is approx. 200 m3/s. Therefore the underwater rock fill placement was required to be done up to El.535.00 (The water level corresponding to 200 m3/s is 534.5 m as per rating curve at inlet of Diversion Tunnel).

b.) Phase- I of the coffer dam was raised up to El 540.00 m (corresponding to 1 m 25 year non-monsoon flood i.e. 684 cumecs,) so that the jet grouting work was accomplished safely during non-monsoon period. The deck level of the bridge 200m D/S of dam was also 540.m. Therefore the 6m bench about 100m U/S of dam axis was utilized for dam foundation excavation. This work of phase- I coffer dam was completed in 1-1/2 months.

c.) Location of jet grouting was kept in such a way that it is as upstream as possible so that the filling work of the other part of coffer dam can also be taken up simultaneously. The location of jet grouted cut-off will was approx. 27m from U/S bottom point of coffer dam and approx 140m U/S of dam. The jet grouting was completed within 3 months.

d.) The final phase of cofferdam was completed in balance 1-1/2 months including rock grouting if required. The u/s coffer dam was provided with u/s face concrete cladding for minimizing the entry of water and building of pore pressure.

© 2016 Deep Foundations Institute

e.) The axis of cofferdam was kept at 120m u/s of dam axis. The top width of the cofferdam was kept 6.0 m to 8.0 m. The river bed level at this location was 532.00 mts and top of coffer dam was to be kept at 554.5m.The rock level at this location was presumed at El 487.00 m on the basis of geological data. The U/S and D/S slope of the coffer dam was kept as 1V: 1.4 H and 1V: 1.6H respectively with a bench of 6.0 m on both sides at El 540.00m. The U/S and D/S slope of the impervious core was kept as 1V: 0.70H and thickness of filter was taken as 1.0m. The total base width thus worked out to approx. 90.00 m. Therefore, even with these slopes and excavation slope of 1.25H: IV for dam foundation, a bench of about 20 m was made available at 532.00 m. for construction facilities. The rock fill material was used from the available excavated muck of diversion tunnels and filter material was used from river bed itself which had sufficient quantity of sand and gravel. Approximately 16000 m3 of impervious material for central core was brought from quarry to be identified for the purpose. Below fig. 11 & 12 shows the details of the construction of the jet grouted cut-off at Teesta V dam site.



Fig. 11 showing construction of jet grouted cut-off at site

© 2016 Deep Foundations Institute

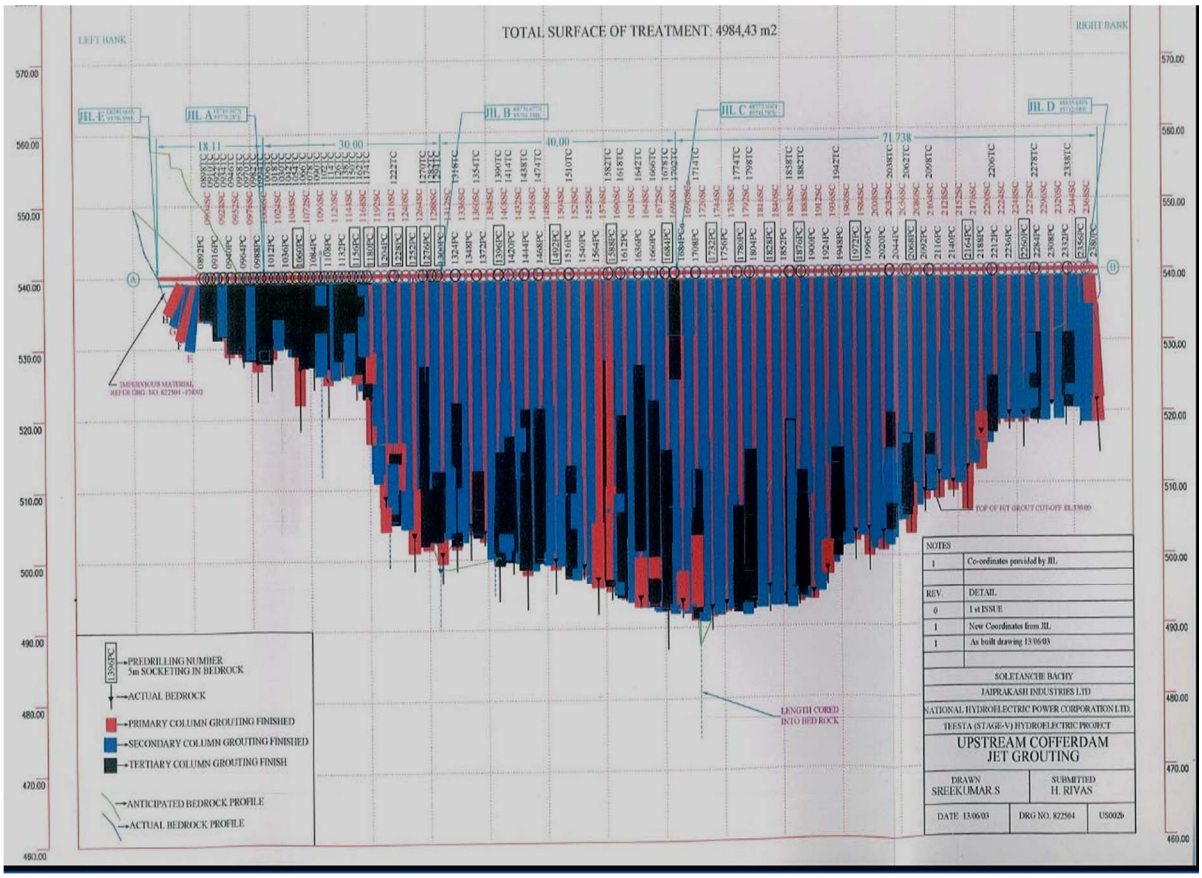


Fig.12 showing the constructed u/s cut-off wall in u/s coffer dam

9.HYDRAULICANALYSISFORPERMEABILITYOFCONSTRUCTEDJETCUT-OFF

9.1 LOCAL PERMEABILITY

The local permeability of the cut-off wall was tested by means of water test in check holes drilled within the jet grout wall from the working platform. Three check holes for the upstream jet grouted cut-off were carried out. Each bore hole of diameter 101 mm was drilled along the centre axis of the wall with a predetermined inclination of 5° to 12° according to the hole. Target permeability value of k =3 e -6 m/s as an average results target value was kept when using the check hole test.

Two types of water tests were performed for checking the local permeability:

1. Permeability test –constant head taking into consideration the position of the bore hole into the cut-off

© 2016 Deep Foundations Institute

2. Permeability test- variable head without taking into consideration the position of the bore hole into the cut-off

The average value of the permeability from constant head and variable head method was found to be 5.9 e-7 m/s and 5 e-7 m/s respectively. Therefore, the permeability measured from water tests performed in check holes is better than the target value of k = 3e-6 m/s.

9.2 GLOBAL PERMEABILITY

The global behavior / testing of the cut-off were carried out by pumping test with a network of piezometers and wells u/s and d/s of cut-off wall. The pumping started for one day and recovery completed in next two days. Before pumping commenced, the cut-off was already subjected to flows caused by

-The ground water level difference between the upstream coffer dam and the downstream coffer dam
-Pumping in the pond between the coffer dams
-The piezometric situation before pumping commenced

Seepage analysis was carried out by FEM software for simulation of the hydraulic behavior of the cut-off during pumping test. The upstream potential was taken as El.532.70 m. The permeability of the alluvium was determined with a small pumping test using the well and piezometers. An average value of alluvium of the order of 1.5 e-4 m/s was used for the simulation. Average permeability value of the cut-off was found to be 0.7 e-6 m/s which was within acceptable limit. Also, the check holes tests done through the cut-off gave an average value between 0.5 and 0.6 e-6 m/s.Fig. 13 & 14 shows the details of the permeability calculation.

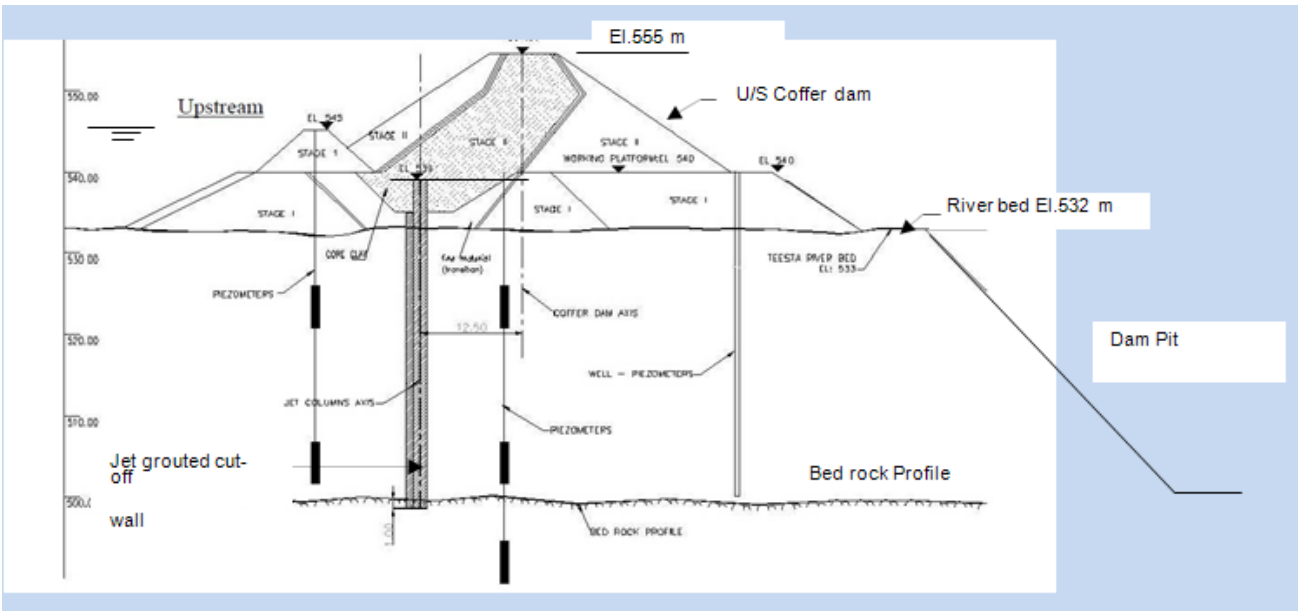


Fig. 13 showing permeability test details

© 2016 Deep Foundations Institute

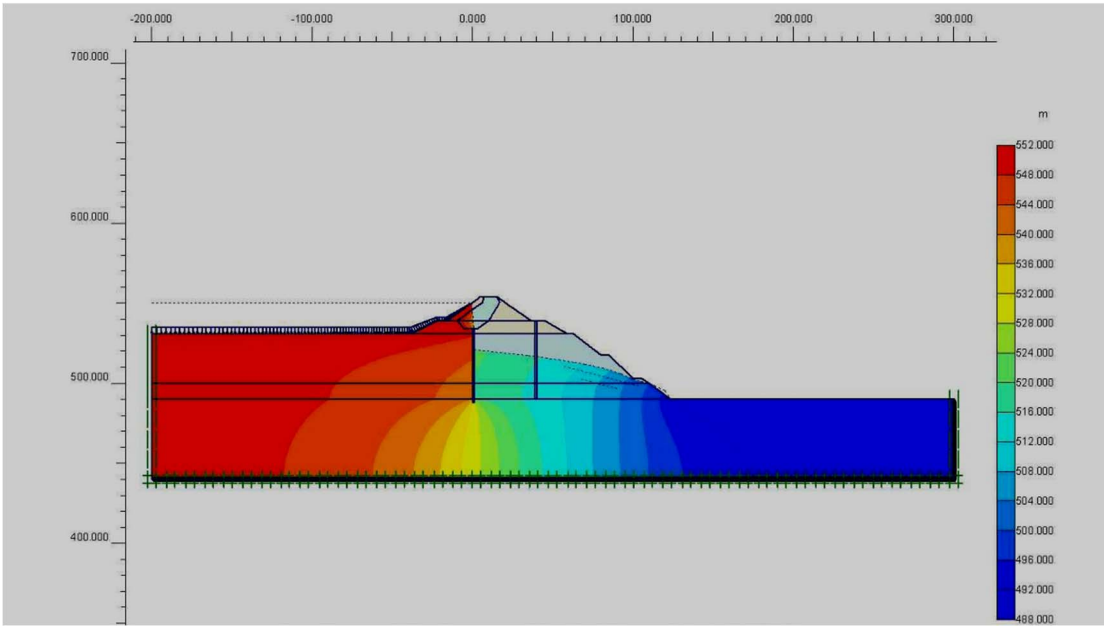


Fig. 14 Seepage analysis by FEM software

10. CONCLUSION

For deep foundation excavations, especially in the middle of mighty river, creation of seepage free space for construction of concrete dam was a real challenge. Seepage free working space was made possible by providing positive jet grouting cut-offs in river bed (approx 40 m deep) material below coffer dams and clubbing it with impervious core within coffer dam body. The main lesson learnt during construction of jet grouting cut-off was to mitigate the drilling deviations & window formations due to presence of large boulders. This was attended by supplementing the jet grouted cut-off with permeation grouting wherever required. The double fluid jet grouting technique was chosen because of its successful application in widely different material available in river bed and for faster construction. Choosing jet grouting cut-off over concrete cut off and grout curtain proved to be right decision as it was least sensitive to big boulders as compared to concrete cut-offs and the Jet grouted cut-off was more credible than the grout curtain. Jet grouted cut-off beneath upstream & downstream coffer dam helped in having the insignificant seepage in the dam pit from the coffer dams even during monsoon season.

As no significant seepage was observed during the dam pit excavation & dam construction, therefore dam concreting continued without any interruption. Jet grouting helped to create almost a seepage free barrier and enhanced the stability of the coffer dam. The successful design & construction of coffer dam, jet grouting cut-off and dam pit excavation was implemented by continuously interacting among the site group, design group & agency constructing the jet grouted cut-off.

© 2016 Deep Foundations Institute

River bed excavation up to 40- 45m depth especially in a place where almost six months are wet (monsoon), was challenging task as conventional working could have incurred huge dewatering problem. Deep Jet grouting of the order of 40ms in such difficult conditions being implemented for the first time in NHPC proved its worth and its result was better than expected. The construction of jet grouted cut off wall below each coffer dam helped in reducing the water flows across the alluvium to an amount that was manageable by pumping. Rather the seepage from the upstream and downstream end was almost negligible and only the seepage water from the right and left abutment was the main contributing factor and was easily managed by dewatering pumps. As the jet grouting and raising of coffer dams was completed in almost six months, the selection of jet grouting cut-off proved more useful & worthy than any other methods.

After having a successful experience of the jet grouting in Teesta V, this method of the providing seepage barrier is being provided in other hydro projects of NHPC also. Fig. 15 shows the completed view of the Teesta-V dam.



Fig. 15 view of the completed Teesta V dam

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

DESIGN AND CONSTRUCTION OF THE CANNELTON HYDROELECTRIC PLANT COFFERDAM AND CUT-OFF WALL

James M. Tantalla, Mueser Rutledge Consulting Engineers, New York, NY, USA, 917 339-9300, jtantalla@mrce.com
Roderic A. Ellman, Jr., Mueser Rutledge Consulting Engineers, New York, NY, USA, 917 339-9300, rellman@mrce.com
Dwight Metcalf, Kiewit Foundations Group, Woodcliff Lake, NJ, USA, 201 571-2500, Dwight.Metcalf@kiewit.com

ABSTRACT

The construction of a new low head hydro-electric plant at the Cannelton Lock and Dam required the design and construction of a cofferdam to accommodate a deep excavation extending approximately 150 feet below the Ohio River levels. The new plant, which generates about 390 GWh of electricity annually, is owned by American Municipal Power (AMP). A specialist team was assembled to design and construct the cofferdam, marine dike and CB slurry wall that helped make the project successful and on time for project delivery.

Because the new plant diverts water from the locks and dam through a powerhouse to generate power, it would need to be partially in the river to operate properly. To accomplish this, approximately 250,000 square feet of land was reclaimed from the river by construction of a marine dike. The dike, combined with the design and construction of a 150 foot deep cement bentonite (CB) cutoff wall enabled construction of the new plant in the dry. Extensive dewatering analyses were performed using Modflow to evaluate pumping requirements and the number of wells needed to maintain ground water at required levels within the side slopes and final subgrade. A monitoring program consisting of inclinometers, piezometers and movement monitoring points were used to monitor cofferdam stability and performance throughout construction. Detailed two and three dimensional stability analyses that included a parametric study of fluctuating river levels and slurry unit weight were performed to ensure stability and suitable low permeability of the trench during cut-off wall excavation both on land and in the marine dike. The wall was constructed with both a hydromill excavating primary panels and a clamshell following behind to construct secondary panels. Data acquisition systems onboard the excavating equipment allowed for detailed confirmation of the continuity of the wall at each interface between panels. Another challenging aspect was the CB mix design for the trench. The cofferdam required the construction of a 1,300-foot long marine dike downstream of the existing dam up to 100 feet tall founded on existing river sediment to enclose the site and cut-off the seepage into the excavation from the river. Marine dike construction utilized vibro-compaction to pre-densify granular fills placed underwater through which the CB trench extended.

KEYWORDS: excavation, river, CB Cutoff wall, seepage cutoff, cellular cofferdam, hydromill, Marine dike, stability analyses

PROJECT DESCRIPTION

The Cannelton Lock and Dam is a USACE facility located 720 miles downriver from Pittsburgh, PA on the Ohio River. It was built between 1963 and 1974 with a main lock 1,200 feet long by 110 feet wide, and includes one auxiliary lock structure. The dam is immediately adjacent to the lock and consists of twelve tainter gates totaling about 1,400 feet wide. The head differential between the upper and lower pools under normal operating conditions is about 25 feet.

© 2016 Deep Foundations Institute

American Municipal Power (AMP), a non-profit, 128-member power producing co-op that owns and operates electric generation facilities for its member communities, is in the process of constructing hydroelectric power plants adjacent to several existing navigation locks and dams on the Ohio River. The Cannelton Hydroelectric plant has been constructed at the Dam on the Kentucky bank of the Ohio River. The renewable power will be produced by three turbine generating units with a total generating capacity of 84 MW at a 25-foot head differential. Ground breaking on the project took place on August 5th, 2009.

The project was let out in separate contracts based on "special expertise and schedule constraints required to bring the generating plant on-line as reasonably soon as possible." A contract was awarded for the design and construction of the cofferdam, including the construction of a marine dike up to 100 feet tall, a 500,000 square foot perimeter seepage control cement bentonite cutoff wall, approximately 1.25 million cubic yards of excavation for the future powerhouse, dewatering and emergency flooding systems, and instrumentation. These key components of the cofferdam are identified in Figure No. 1.

The design was reviewed by the Owner's Engineer, and was reviewed and approved by the US Army Corps of Engineers (USACE) and a Board of Consultants selected by the Federal Energy Regulatory Commission (FERC) through a series of design submittals and review meetings.

GEOLOGY

A total of 38 borings, five of which were located in the Ohio River were provided in the GDR. In addition, 11 test pits between 3.5 and 20-feet deep were completed. Rock cores were extracted as part of the investigation program. As part of the design build contract, an additional 7 borings were taken to confirm the information presented in the Contract Geotechnical Design Report.

The project site is situated within the Ohio River valley in Kentucky. The valley was formed in glacial times. During the ice age, several glaciers approached the Ohio River valley. Retreat of the early glaciers scoured the bedrock into a U-shaped valley. The width and depth of the valley in any particular area depended on the hardness of the bedrock and its resistance to scour by the debris laden water. The valley extends several thousand feet into Kentucky.

The valley was filled with sediment carried by melt water during the retreat of subsequent glaciers. This alluvial soil is comprised of two distinctly different deposits. The lower deposits are coarse-grained and consist primarily of boulders, cobbles, gravel, and coarse sand that form a consistent layer atop the bedrock across the width of the valley. Clay, silt, and fine-grained sand are often intermixed with the coarser material or exist as discrete lenses. These finer grained materials were deposited during periods of slack water or temporary ponding of the glacial stream. The upper deposits primarily consist of finer grained material (clay, silt, and sand) deposited during the waning of the glacier. Of particular concern for this project was the presence of some occasional layers of stiff, high plasticity clay that had a tendency to grab the very wide (4.2M) clamshells used for this project.

The bedrock consists of shale, sandstone, and limestone with an average compressive strength of about 7.500 psi. A layer of weathered and decomposed rock was encountered in many of the borings at the top of rock. The cut-off wall extended to bedrock, however the excavation for the powerhouse left forty feet of overburden on the rock. Overburden varied from soft, fine grained river deposits to glacial till.

© 2016 Deep Foundations Institute

DESIGN

The cofferdam design consisted of different components, including a marine dike, excavation, cut-off wall, instrumentation, dewatering system, a cofferdam tie-in to the existing dam, slope stability analysis, finite element deformation analysis, and an emergency flooding structure. Design reports were prepared for each of these components for review and approval by the Owner's Engineer, FERC's Board of Consultants, and the Army Corps of Engineers. Key components of the cofferdam for groundwater and seepage control are identified in Figure No. 1.

Marine Dike

The marine dike represents one-third of the overall perimeter of the cofferdam as shown in Figure No. 1. Settlement of the river bottom foundation for the marine dike was a concern due to the compressibility of the natural soils. Slope stability of the dike was also a concern due to the weak founding soils and sloping bedrock into the river away from the dike. To assess the river bottom a bathymetric survey was performed was well as a penetration test into the river bottom silt consisting of a 6-foot diameter x 10-foot long steel pipe filled with rock being lowered on the silt and measuring the penetration. If it penetrated more than 6-inches additional foundation rock was required. This testing was performed on a 100-foot x 100-foot grid.

A schematic cross section of the marine dike is shown in Figure No. 2. The marine dike consisted of two outer containment berms of large diameter rip-rap (sized to prevent scour during possible 100 year flood events) placed in the river from a maximum depth of over 70-feet to above the minimum pool elevation, with two inner transition layers of progressively smaller stone covered by filter fabric. The central portion of the dike was composed of sand with a maximum of 10 percent passing the #200 sieve, partially placed underwater and partially placed in the dry to the level of the cutoff wall working platform. Once the cut-off wall was installed, the dike was built up to its final elevation with an impervious clay cap. The completed dike was over 100 feet tall at its largest.

Due to the need to excavate a cut-off wall through the dike, the competence of the "dumped" sand placed underwater was a major concern. Trench stability calculations established a minimum friction angle of 30-degrees along with slurry unit weights maintained at 70 pcf or greater for acceptable stability of the trench. Trench failures in the marine dike in the Ohio River would have been catastrophic to the project. The project team opted for a program of vibroflotation on a regularly spaced pattern to improve the sands placed underwater. After vibroflotation was complete, a cone penetration testing verification program was undertaken to confirm the improvement.

Cutoff Wall

The cement bentonite cutoff wall's performance was critical to effective groundwater control by the cofferdam. The wall was approximately 150 feet deep. The wall was excavated in two different ways depending on location: a continuous trench was excavated on the landside, where the excavation was primarily in natural soils and demonstrated good stability, and the wall was excavated in panels on the marine dike, where looser soil was anticipated. The greatest risks that were considered in the design and construction of the cutoff wall included the permeability of the wall, the quality of the closure to bedrock, continuity between panels, and trench stability.

The wall was required to have a permeability of not greater than $1x10^{-6}$ cm/sec. Permeability was confirmed as part of the project's QA/QC program that verified the strength and permeability of the wall materials, and included 2,300 samples taken in total, of which 890 were from the batch plant and 1,410 cast from grab samples taken from various elevations in the trench.

© 2016 Deep Foundations Institute

The presence of more permeable till and weathered rock above the bedrock caused concern for excessive leakage at the wall to bedrock interface, which lead to great consideration of whether the cutoff wall should be keyed several feet into rock or just to the top of rock with verification that the wall actually reached rock.

Results of the groundwater modeling and a review of the geotechnical conditions at the top of rock established that a key would not be needed if the excavation could properly clean the top of rock. The final specification requirements were that when the hydromill was touching rock, field engineers and Owner's reps would review the spoils collected at the desanding unit and mutually agree when "fresh" rock fragments were being produced; at this point the panel was sufficiently excavated. For secondary panels, when the 20 ton clamshell was no longer bringing up decomposed or fractured rock then bottom was achieved.

Trench stability calculations were carried out as an infinite 2-D trench where longer open trenches were utilized (landside), and as a 3-D analysis where shorter panel lengths were planned due to small differential head and weaker soils (marine dike). Stability analyses were developed for both varying slurry unit weights and varying river levels. Slurry unit weights in the trench from 72-80 pcf were considered, along with ground water levels anywhere up to 25 feet above normal pool levels (work was to be stopped if higher water levels were predicted. The results of the analyses were implemented in construction through tables that specified the required unit weight of slurry for a given river stage to maintain the required factors of safety as shown in Figure No. 3.

Dewatering

A deep well dewatering system was designed for the site that utilized twenty wells arranged around the perimeter of the cofferdam as shown in Figure No. 4. The presence of a consistent deposit of coarse grained soils above bedrock across the site increased the efficacy of this type of dewatering system by performing as an underdrain to continually feed water from the lower permeability upper sediments to the pumps around the site.

The system was sized to allow for up to 5,000 gpm pumping capacity if difficulties were encountered during dewatering. A 3D finite difference groundwater model was developed in MODFLOW to simulate the performance of the dewatering system and predict pumping quantities from individual wells. The finite difference model consisted of a 3,000 ft. x 3,000 ft. area subdivided into a grid of approximately 250 rows and columns, with variable spacing to allow for increased detail on groundwater flow at the cutoff wall and wells. A transient analysis was performed to allow for estimates of initial flow to the system in addition to steady state quantities.

Results of the analysis indicated that the maximum anticipated initial pumping quantities absent any substantial discontinuities in the wall would not exceed 2,600 gpm, with expected steady state pumping quantities on the order of 400 gpm. During construction, the cofferdam exceeded performance estimates, and the dewatering system averaged about 150 gpm to maintain the water levels required for construction. Drawdown was rapid and successfully maintained below excavation subgrade during construction, as shown in Figure No. 5.

Excavation

Construction of the powerhouse required an excavation subgrade more than 105 feet below the top of the cofferdam. The excavation was in existing land and adjacent to the marine dike. Stability of the slopes and long-term performance of the cofferdam were important concerns from the Board of Consultants during review of the proposed work. The excavations were analyzed for stability by limit equilibrium analysis

© 2016 Deep Foundations Institute

including transient, rapid drawdown cases. Steady state analyses were performed using assumed groundwater conditions based upon the results of the dewatering analysis. In addition, the maximum phreatic surface that maintained acceptable slope stability factors of safety was iteratively determined to establish threshold values for piezometers located throughout the cofferdam. Deformations were estimated by performing finite element analyses. The limit equilibrium analyses resulted in excavated slopes cut at approximately 2.5H:1V slope. Results of the finite element analyses were used to develop threshold levels for an extensive system of instrumentation including inclinometers and surface mounted movement monitoring points around the site to confirm performance during excavation.

Emergency Flooding Structure

The cofferdam has been designed to maintain a dry excavation for river levels up to the 100 year flood. An emergency flood gate and spillway were designed and installed in the marine dike to allow filling of the cofferdam from the downstream side if needed to prevent potential damage due to overtopping or slope stability if water levels were ever predicted to exceed the 100 year flood levels.

The flooding structure consisted of two primary components: a floodgate and a spillway. The floodgate consisted of a group of four box culvert sections that were closed off with timber needle beams and a steel king pile for initial opening. If the cofferdam needed to be flooded, the needle beams could be removed from above. The spillway was constructed of grouted rip-rap down to the bottom of the excavation to prevent erosion of the cofferdam interior during filling. The flooding structure is shown in Figure No. 6.

The flooding structure was designed so that the entire cofferdam could be filled to three-quarters full in 4 hours. Various rates of rise on the river based on observations during previous floods at the Cannelton Lock and Dam were considered to determine the river levels when the gates needed to be opened to meet the filling requirement. Maximum flow through the system in the event of flooding would exceed 2.5 million gallons per minute - the entire gate and spillway were designed hydraulically to prevent erosion and protect the cofferdam under these high flows during a flooding event.

CONSTRUCTION

Vibroflotation

A Vibro Systems Inc. V23 300 KN 23mm amplitude vibroflot was used to densify the dumped marine sands in the marine dike. The probe itself is 3.57 m long and 350mm in diameter. The project used a 21m follower to achieve maximum depth of 18m (60 ft.). The test program consisted of 5 different patterns ranging from 3.8m x 3.8m to 6m x 6m. CPT testing confirmed that the 5.3m x 5.3m pattern provided sufficient densification. The team made the decision to use a 3.8m x 3.8m pattern within 9.1m of the cutoff wall to further improve trench stability. A total of 866 vibroflotation holes were completed on the project (photo 2)

Cutoff wall excavation

The excavation of the cutoff wall was completed with a combination Soilmec CH8-10 HWD 58 Hydromill (800mm x 3000mm) Cougar model mounted on a SC120 crane. This hydromill configuration offers a hydraulic motor within the gear box and the additional feature of articulation of the wheel gearbox itself to aid in steering in conjunction with the extendable shoulders. The hydromill was equipped with a full instrumentation package and download capabilities to include x, y, and z deviation. The hydromill served as the primary excavation tool and excavated all the primary bites while the clamshell excavated

© 2016 Deep Foundations Institute

secondaries. The hydromill and clam shell, shown working in tandem in Figure No. 7, was the combination that allowed the job to progress quickly enough to beat the rising waters of the Ohio River.

Continuity was defined as a minimum 10-inch section at any given point in the nominal 31.5-inch wall, and was controlled through the articulation of the instrumented hydromill while excavating primary panels, which also aided in controlling the positioning of the clamshell while excavating secondary panels.

CB Mixing

Kiewit's Foundation Group performed the cutoff wall construction for the joint venture. As part of that task they designed and had fabricated a custom mix plant consisting of side by side 5 cubic yard vortex mixers fed by six 350 barrel silos. A 6x8x15 mission pump provided the shearing mechanism to blend the components. The mix design was formulated to allow pumping up to approximately 2,000 feet prior to placement in the trench.

Throughout the construction of the 500,000 square foot cutoff wall 2,300 samples were collected, with about 40 percent taken at the batch plant and the remaining samples taken from the trench. Unconfined compressive strength and permeability tests were run on site by employees of the Kiewit Foundation Group to verify the performance of the cutoff wall.

CONCLUSIONS

The design and construction of the Cannelton Hydroelectric Project cofferdam was successful due to the close coordination, planning and communication of all parties. The cofferdam, shown completed in Figure No. 8, performed well during the entire construction process. The permeability achieved was one-half an order of magnitude lower than required and the amount water pumped by the dewatering system was much less than expected during design (150 gpm vs 400 gpm predicted). The power house construction has experienced no issues with water infiltration. Two of the three turbines are currently producing power, with the third in final testing and expected to come online in the near future.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the efforts of the entire team that made the project a success. The cofferdam was constructed by a joint venture of Kiewit Infrastructure Co. (formerly Kiewit Construction Company) and Traylor Bros. Inc. Subcontractors for the work included Liquid Earth (CB wall mix design), and Moretrench America Corporation (dewatering system). Mueser Rutledge Consulting Engineers was the Engineer of Record for the design of the cofferdam.

© 2016 Deep Foundations Institute

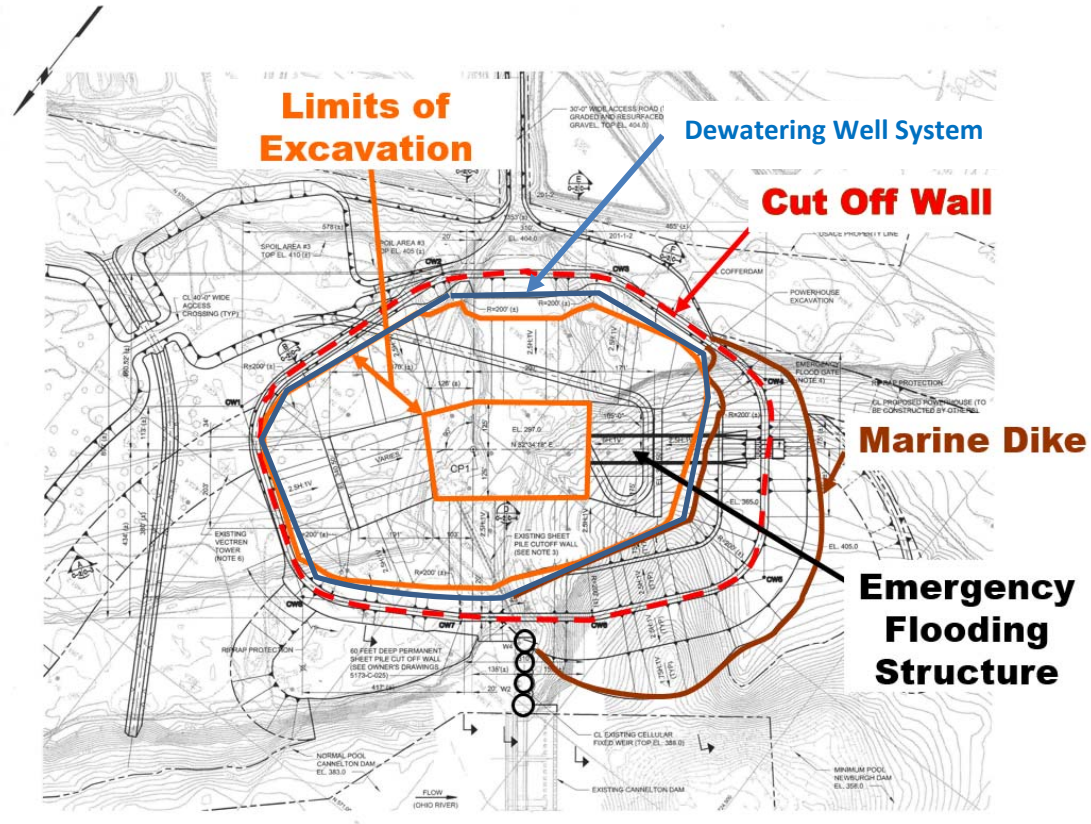


Figure No. 1 – Major cofferdam components

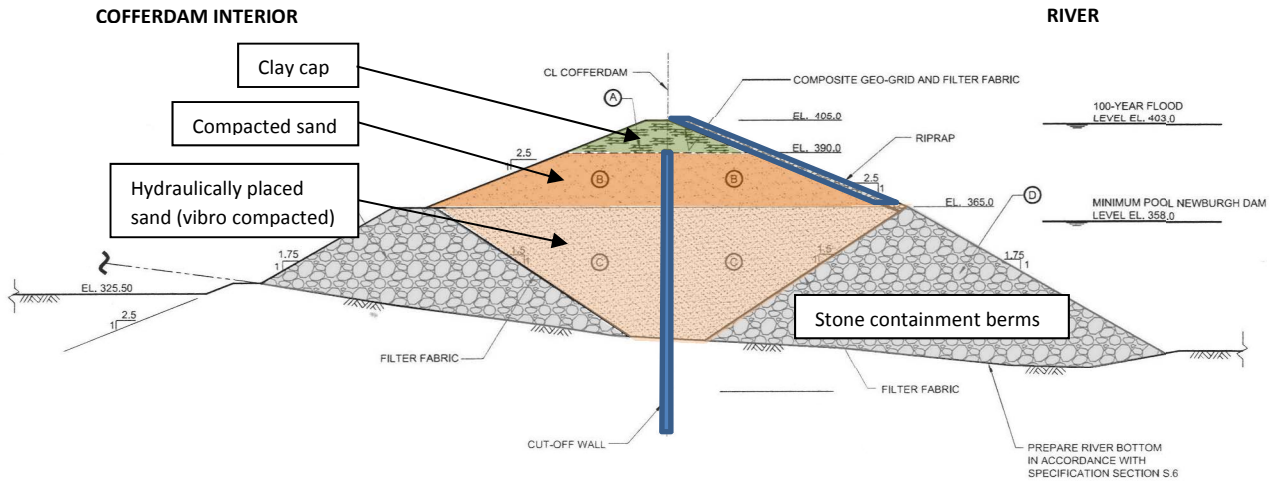


Figure No. 2 – Marine dike cross section

© 2016 Deep Foundations Institute

WATERSIDE (TYP.)

GW EL.	Minimum γslurry (pcf) with FS=1.25	GW EL.	DEC-MAR days above last 30 yrs
359	<68	359	99.4%
361	<68	361	90.3%
363	<68	363	78.5%
365	<68	365	68.3%
367	<68	367	58.9%
369	<68	369	50.9%
371	<68	371	43.6%
373	<68	373	36.3%
375	<68	375	30.1%
377	<68	377	23.9%
379	<68	379	18.8%
381	<68	381	14.8%
383	69	383	11.4%
385	75	385	8.5%

Figure No. 3 – Sample cut-off wall construction guide table

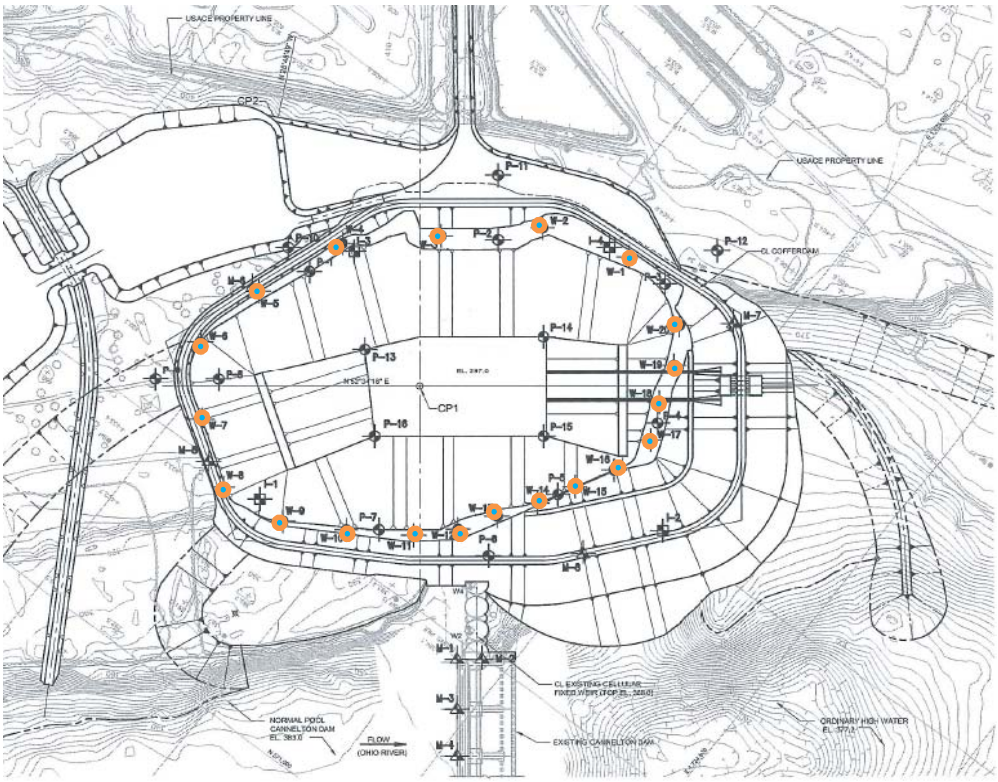


Figure No. 4 – Proposed dewatering system layout

© 2016 Deep Foundations Institute

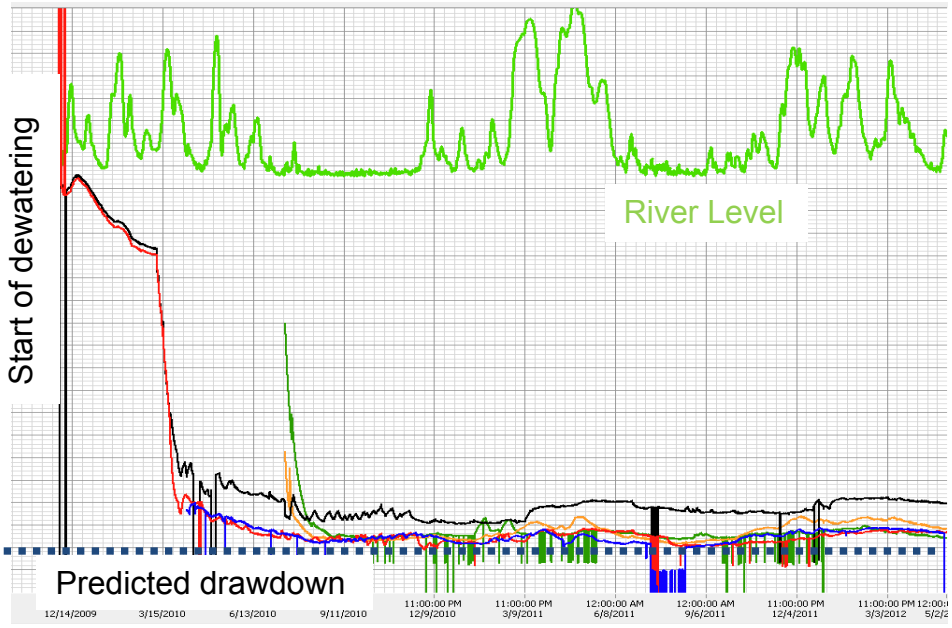


Figure No. 5 – Observed drawdown during dewatering


Figure No. 6 – Cofferdam showing emergency flooding structure

© 2016 Deep Foundations Institute



Figure No. 7 – Hydromill and clamshell working in tandem to construct cut-off wall


Figure No. 8 – Aerial photo of completed cofferdam

© 2016 Deep Foundations Institute

DRAWDOWN OF GROUND WATER CONTROL SYSTEM IN DEEP EXCAVATION SURROUNDED BY CUTOFF WALLS: NUMERICAL INVESTIGATION

Dalia. R. Mohamed, Housing and Building National Research Centre, Egypt, daliaa_ragab@yahoo.com
Marawan M. Shahien, Tanta University & Hamza Associates, Egypt, marawan.shahin@f-eng.tanta.edu.eg
Hisham H. Hafez, Cairo University, Egypt, hisham.hafez@egec-xprt.com

ABSTRACT

Construction activities in major projects that involve deep excavations necessitate the use of vertical walls surrounding the construction site to play an important role as earth retention system. The walls are usually partially penetrating impervious walls which can increase the efficiency of the ground water control system that usually consists of number of partially penetrating wells. The analysis and design of such system requires 3D numerical simulation. The aim of this study is to develop simplified semi-empirical approach to analyze and design such system. The analyzed model is a single well in a square site that is surrounded by impervious walls along the perimeter of the site in an unconfined aquifer. The results of the parametric analysis are presented in this paper aiming to assess and understand the factors that influence dewatering in terms of ground water or drawdown profile surrounding the well. The parametric analysis is carried out using the 3D finite difference software. The parametric analysis comprises with more than 550 cases calculated. The study involved the parameters; well penetration depth, impervious wall penetration depth, size of the construction pit, coefficient of permeability, well yield and well location or distance between well center to impervious wall edge.

KEYWORDS: **Deep wells, Dewatering, Drawdown, Ground water control, Partially penetrating cutoff walls, Well penetration depth, Well location.**

INTRODUCTION

The presence of groundwater in a site may have impact on the construction processes involved during the course of project construction, especially those projects that involve deep excavations. Deep excavations, especially in urban areas, require the construction of supported walls all around the site perimeter to ensure the stability of the excavation. In most of cases, the constructed walls are relatively impervious. Depending on the ground and site conditions, the walls are not penetrating all the way down through to the bottom of the aquifer and thus the walls are partially penetrating walls. The presence of ground water necessitates the lowering of the groundwater to levels below the bottom of the excavation to ensure working in dry. This paper focuses on groundwater control systems that consist of deep wells inside square areas enclosed by impervious walls partially penetrating unconfined aquifers. The wells are partially penetrating the aquifer, as well. There are many factors or parameters that are related to the configurations of the components of the system that may influence the drawdown of a well in such groundwater control system. This paper is a part of a comprehensive study that aims to development of a simple method for estimating the drawdown of dewatering system of the described configuration. In order to achieve such a major goal, a 3D finite difference model is developed for the problem in hand. A comprehensive parametric analysis was then carried out to investigate the influence of all the parameters on the drawdown of a deep well in such groundwater control system. This paper describes the problem, the model, the parametric analysis including the involved parameters. The paper, then, demonstrate the results of the analysis showing the influence of the studied parameters on the drawdown of a single well.

© 2016 Deep Foundations Institute

MODEL OF GROUNDWATER CONTROL SYSTEM

The model of groundwater control system is described in plan and in cross section in Figs. (1) and (2), respectively. The dewatering system under consideration consists of a single partially penetrating deep well having a penetration depth of lw and well yields of Q. The well is located inside a construction pit with a square shape that is W in width. The construction pit is surrounded by impervious walls from all sides. The walls have equal penetration depth of lc. A quadratic coordinate system is considered with origin located at one of the corners of the construction pit. The x and y axes of the coordinate system is as shown in Fig. (1). The well is located at X and Y distances from the origin of the coordinate system in Fig. (1). The well is located at any radial distance D between well center and wall edge. The distance between well center to any point within the construction pit along any radial pass around the well is distance r.

GROUND CONDITIONS

The ground condition of the model is a continuous homogeneous isotropic fine to medium sand layer with coefficient of permeability of k. The ground water level exists at 3.0 m below ground surface. The sand layer constitutes an unconfined aquifer with a depth H below the ground water level. The aquifer is underlain by an impervious layer.

NUMERICAL MODELING

The problem in hand described above is simulated using a three-dimensional model that is solved using finite difference approach. The program used is MODFLOW developed by McDonald and Harbaugh (1988) and Harbaugh and Mc-Donald (1996). The pumping well is represented using the well package (WELL) supported in VISUAL MODFLOW4.2®. The pumping rate and levels of the top and bottom elevations of the well screen are assigned through WELL. The vertical cutoff is represented using the horizontal flow barrier package (HFB), through which the cutoff penetration depth and the hydraulic characteristics of the barrier are assigned.

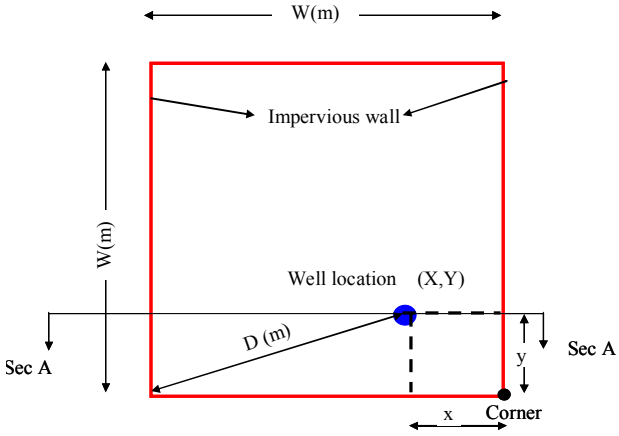


Fig. 1. Plan of model area

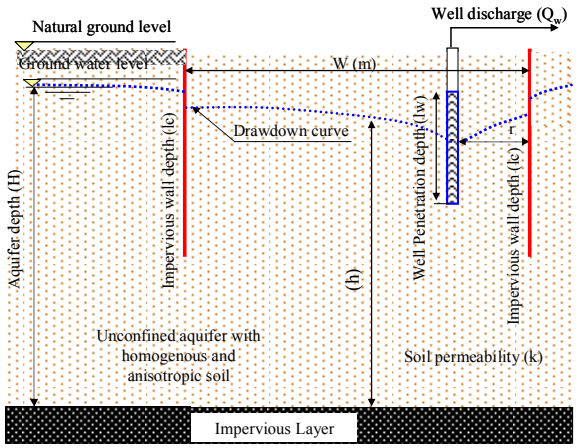


Fig. 2. Section A-A of model area

PARAMETRIC STUDY

A comprehensive parametric study is carried out using VISUAL MODFLOW software. The parametric study considered the variables shown in Table (1). The analysis comprised with over 550 runs. In order to investigate the drawdown profile around the well, the drawdown computed from visual Modflow is

© 2016 Deep Foundations Institute

extracted in eight different directions from well center to wall edges (d1, d2, d3, d4, d5, d6, d7 and d8) as shown in Fig. 3 and listed below:

d1, d5 : Drawdown in y-axis direction
d3, d7 : Drawdown in x-axis direction
d2, d4 : Drawdown in directions making an angle of 45° with +ve x-axis
d6, d8 : Drawdown in directions making an angle of 45° with –ve x-axis

Table 1. Variables Considered in Parametric Study

Variable	Symbol	Unit	Range of values used
Coefficient of permeability	k	m/s	0.5, 1, 2.5, 5.0, $8.0x10^{-4}$
Well pumping rate	Q	m^3/hr	30, 45, 60 and 90
Impervious wall penetration depth	lc	m	20, 25 and 30
Well penetration depth/ Impervious wall penetration depth	lw/lc		0.75, 1.00 and 1.25
Depth of aquifer	H	m	40, 50 and 80
Area surrounded by square impervious wall WxW	W	m	21, 31, 41, 71 and 100
Location of well (distance from well center to closest impervious wall edge in direction of X, Y axis)	X, Y	m	Several values covering entire area from 0 to 0.5 W

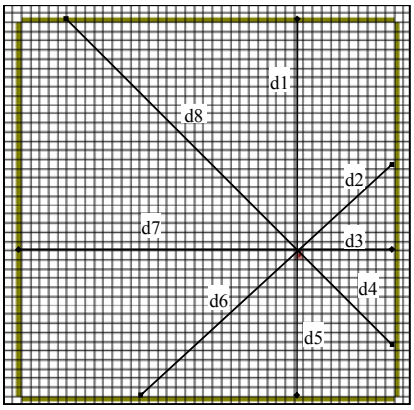


Fig. 3. Different directions for presentation of drawdown profiles around the well

TYPICAL DRAWDOWN CURVE ACROSS THE WALLS

Fig. 4 shows typical result of the parametric analysis that show drawdown curves along a certain direction starting from outside the impervious wall, across the wall, to well center, to the other wall in the direction d1, across the other wall and beyond the wall outside the construction pit. The drawdown curve when crossing the impervious wall shows a sudden change as stated by (Discrol, 1986, Freeze et al., 1979, Powers, 1992, Preene, et al., 2000, and Abdel Haleem, 2006).

© 2016 Deep Foundations Institute

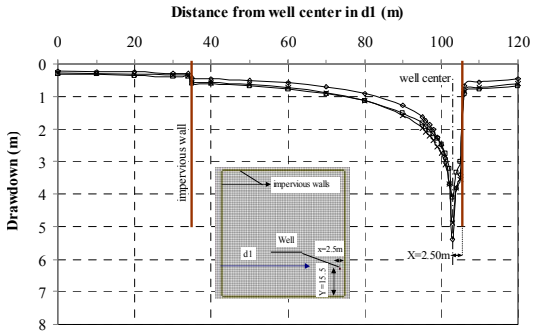


Fig. 4 Typical Drawdown curve along different distances from well center

EFFECT OF WELL PENETRATION DEPTH

Typical results of the parametric study are shown in Figs. 5 and 6 that correspond to two construction pits with sizes of 21x21m^2 and 71x71m^2, respectively. In each pit, a single well is located near the wall 2.5 m from the wall in the X axis and at the center of the construction pit in the direction of the Y axis. In each Figure, three ratios of well penetration depth to impervious wall depth (lw/lc) of 0.75, 1.00 and 1.25 are presented. The ratios correspond to well penetration depths that are shallower, equal and deeper than the wall penetration depths, respectively, considering constant wall penetration depth of 25m. Figs. 5 and 6 show the same data presented in Figs. 7 and 8 in the form of relationship between drawdown versus the ratio lw/lc for different distances from the center of the well.

In general in the distances near the well center for r less than 2.5 m to 5.0 m, the drawdown increases with the decrease in well penetration depth as reflected by the ratio lw/lc. In distances more than about 5.0 m for construction pit with large size, the well penetration or lw/lc seems to have less significant influence on the drawdown.

In construction pits with small size and for distances more than 5.0 m away from well center, there is a tendency for the drawdown to slightly increase with the increase in well penetration from lw/lc of 0.75 to 1.0. For further increase in well penetration, the drawdown seems to level off.
The sudden change in drawdown across the impervious boundary is influenced by well penetration or lw/lc in same manner described above in 1) and 2).

The construction pit size has significant influence on the amounts of drawdown in general. As the size of the construction pit increases, the drawdown decreases. It is interesting to observe that with the increase in construction pit size, there is increase in the rate of decrease in drawdown inside the construction pit with the increase in well penetration.

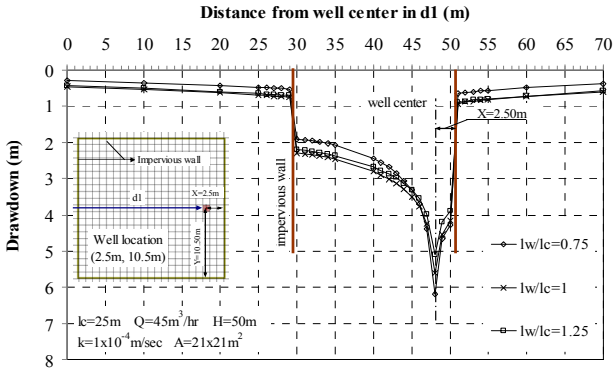


Fig. 5. Drawdown curves for various lw/lc (21x21m^2)

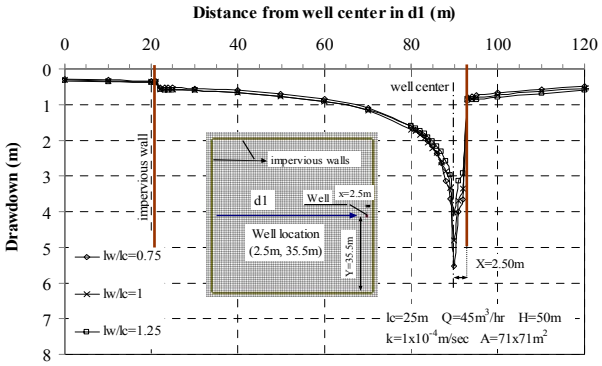


Fig. 6. Drawdown Curves for various lw/lc (71x71m^2)

© 2016 Deep Foundations Institute

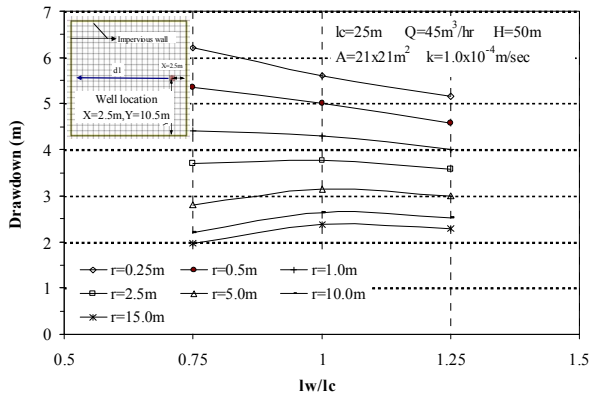


Fig. 6. Drawdown versus lw/lc (21x21m^2)

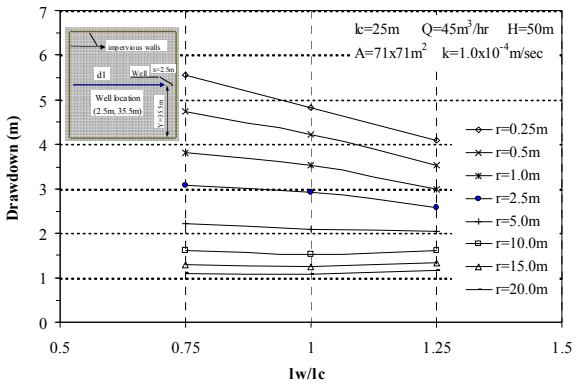


Fig. 7. Drawdown versus lw/lc (71x71m^2)

EFFECT OF IMPERVIOUS WALL PENETRATION DEPTH

Figs. 8 and 9 show drawdown curves for single well in construction pits of the sizes 21x21m^2 and 41x41m^2, respectively. In each Figure, three ratios of impervious walls depth to aquifer depth (lc/H) of 0.312, 0.5 and 0.625 are presented. The ratios correspond to aquifer depths 40m, 50m and 80m considering constant wall penetration depth of 25m. The presented data correspond to lw/lc ratio of 1.0. Figs. 10 and 11 show the same data presented in Fig. 8 and 9 in the form of relationship between drawdown versus the ratio lc/H for different distances from the center of the well. As the aquifer depth decreases as reflected in an increase in lc/H, the drawdown, in general, increases. The sudden change in drawdown across the impervious boundary is influenced by aquifer depth or lc/H.

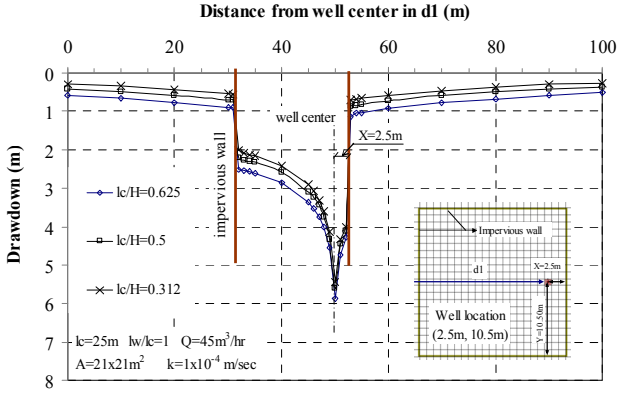


Fig. 8. Drawdown Curves for various lc/H (21x21m^2)

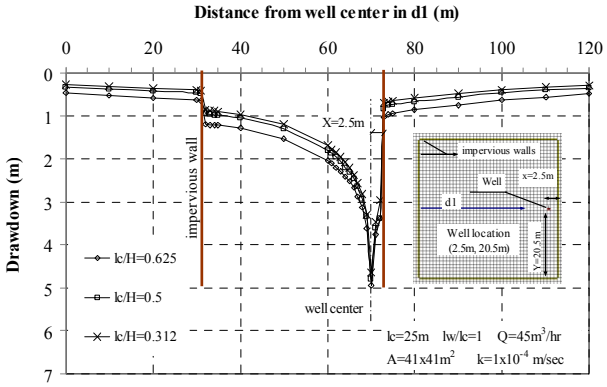


Fig. 9. Drawdown Curves for various lc/H (41x41m^2)

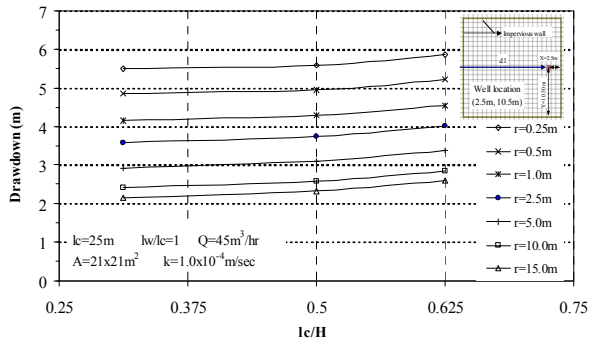


Fig. 10. Drawdown versus lc/H for lw/lc=1 (21x21m^2)

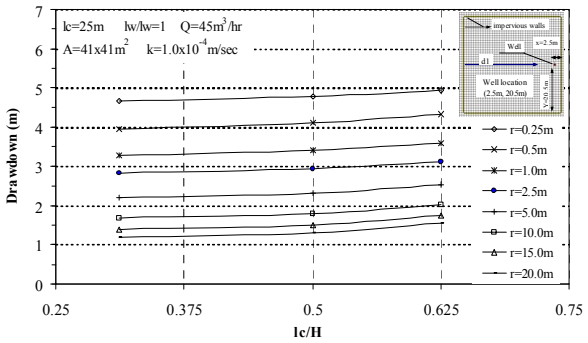


Fig. 4. Drawdown versus lc/H for lw/lc=1 (41x41m^2)

© 2016 Deep Foundations Institute

EFFECT OF COEFFICIENT OF PERMEABILITY

Figs. 12 and 13 show drawdown curves for single well in the center of construction pits of the sizes 41x41m^2 and 71x71m^2, respectively. In each Figure, three drawdown curves are presented for three coefficients of permeability. The difference in the drawdown scales in both figures should be noted. As expected, the drawdown decreases with the increase in coefficient of permeability. It is noted that the sudden change in drawdown across wall decreases with the increase in coefficient of permeability.

EFFECT OF WELL DISCHARGE

Figs. 14 and 15 show drawdown curves for single well in the center of construction pits of the sizes 21x21m^2 and 71x71m^2, respectively. In each Figure, three drawdown curves are presented for three discharge rates. The difference in the drawdown scales in both figures should be noted. As expected, the drawdown increases with the increase in discharge rate of the well. The sudden change in drawdown across wall increases with the increase in discharge rate.

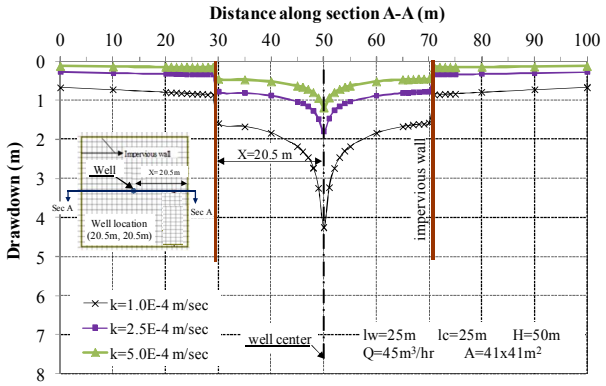


Fig. 12. Drawdown curves for various coefficients of permeability for construction pit (41x41m^2)

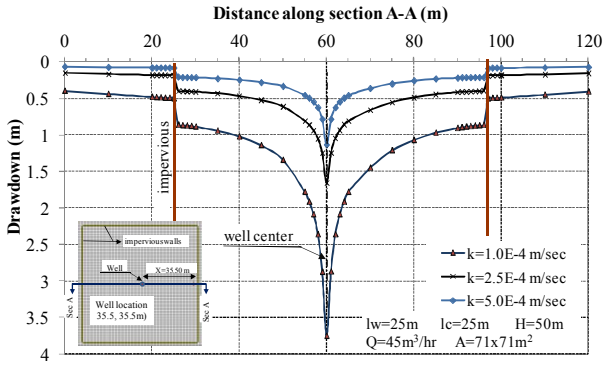


Fig. 13. Drawdown curves for various coefficients of permeability for construction pit (71x71m^2)

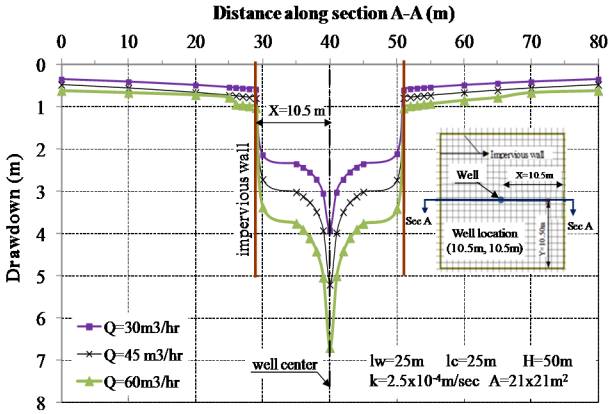


Fig. 14. Drawdown curves for various well discharge rates for construction pit (21x21m^2)

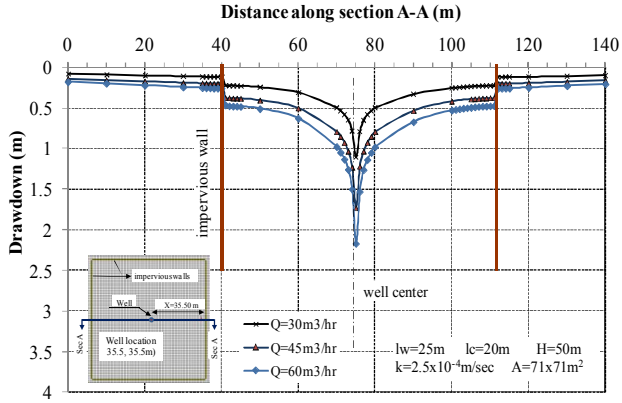


Fig. 15. Drawdown curves for various well discharge rates for construction pit (71x71m^2)

EFFECT OF WELL LOCATION

Figs 16, 17, 18 and 19 show results of drawdown curves for wells in construction pits with sizes 21mx21m, 31mx31m, 41mx41m and 71mx71m, respectively. In each figure, the drawdown profiles of two cases of individual wells of different locations are presented along the directions of d3 and d7 (Fig. 3). One of the two locations is near the impervious boundary at x=y=2.5m from the corner of the construction pit, while the second location is at the center of the construction pit.

© 2016 Deep Foundations Institute

From Figs. 16, 17, 18 and 19, it can be observed that the closer the well location to the impervious wall, the greater is the drawdown. This is more pronounced as the size of the construction pit decreases. It is anticipated that the observed trend may change if the ratio of well penetration to wall penetration changes as discussed earlier.

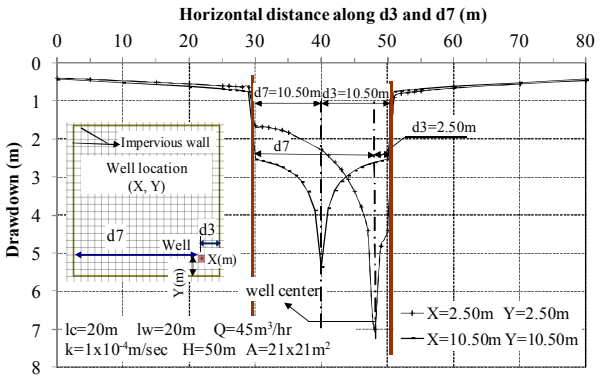


Fig. 16. Drawdown curve for different well location in A 21x21m²

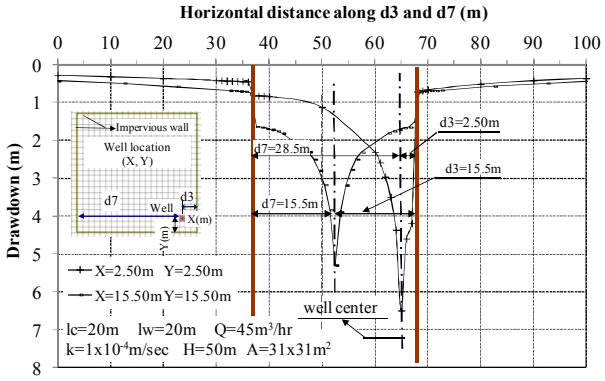


Fig. 17. Drawdown curve for different well location in A 31x31m²

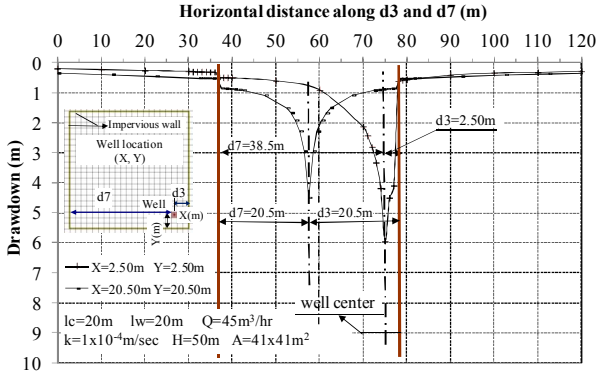


Fig. 18. Drawdown curve for different well location in A 41x41m²

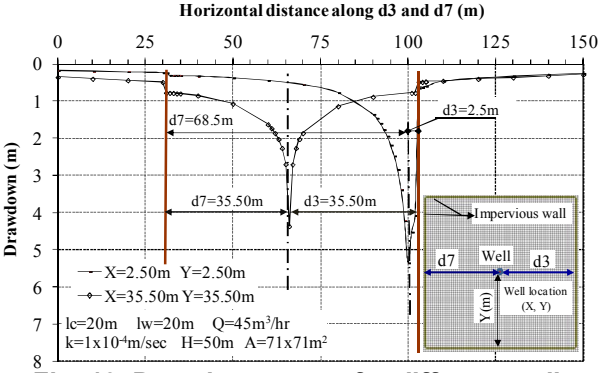


Fig. 19. Drawdown curve for different well location in A 71x71m²

Fig. 20 shows the draw down curves in all directions for a well located near the edge corner of construction pit with a size 21x21 m. Similarly, Fig. 21 shows draw down curves for a well at edge corner of construction pit with the size of 71x71m. It can be observed that drawdown at near distance from wall edge (where D is about 3.53m) is higher than that obtained at the same distance from well center in the long directions (where D is between 18.5m and 68.5m).

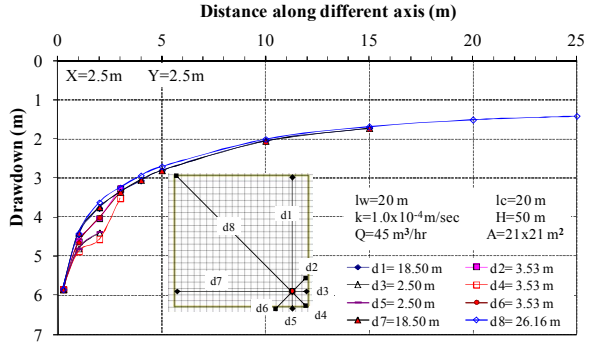


Fig. 20. Drawdown results in different directions for W of 21m (X=2.5m, Y=2.5m)

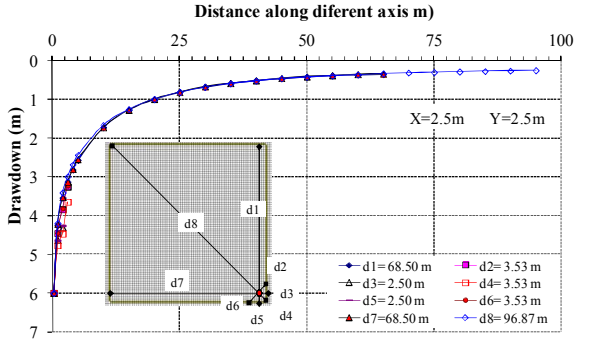


Fig. 21. Drawdown results in different directions for W of 71m (X=2.5m, Y=2.5m)

© 2016 Deep Foundations Institute

Figs. 22 and 23 show the drawdown curves in all directions for a well located at the center of construction pits with sizes 21x21m and 71x71m, respectively. It can be concluded that drawdown at different directions are nearly equal for both cases of areas 21x21m and 71x71m.

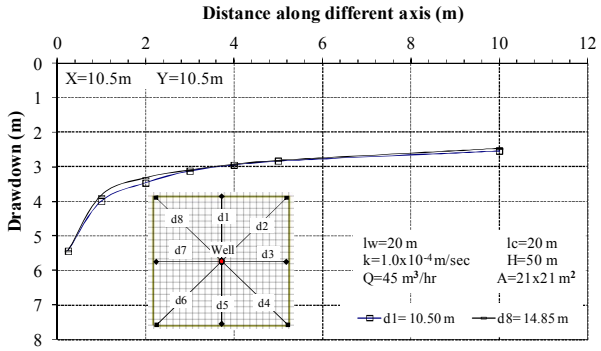


Fig. 22. Drawdown results in different directions for W of 21m (X=10.5m, Y=10.5m)

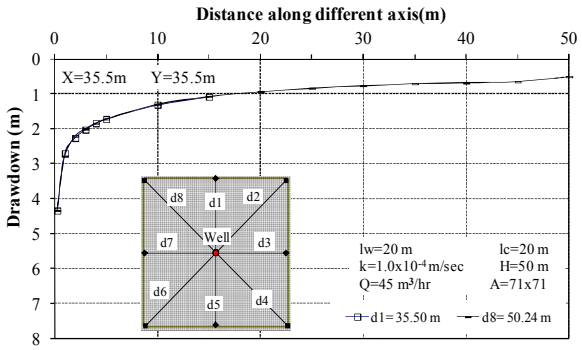


Fig. 23. Drawdown results in different directions for W of 71m (X=35.5m, Y=35.5m)

DRAWDOWN AT WELL CENTER

Fig. 24 shows the relationship between drawdown at or near well center versus distance between well center and wall edge in construction pit with various sizes. In general, as the distance between well center to impervious wall edge increases, the drawdown at or near the center of the well decreases. The values of the drawdown decrease with the increase in construction pit size. Further, the variation of drawdown with distance D; decreases with the increase in size of construction pit.

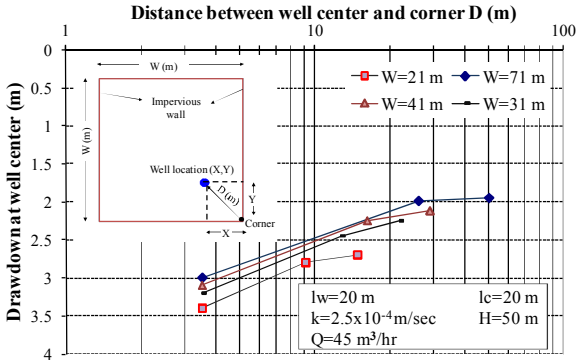


Fig. 24. Relationship between drawdown at well center versus distance between well center and impervious wall

EFFECT OF SIZE OF CONSTRUCTION PIT (W)

In all of the above considered factors, comparisons are given to show the influence of size of the construction pit. In general, the drawdown values decrease with the increase in construction pit size.

SUDDEN CHANGE IN DRAWDOWN ACROSS IMPERVIOUS BOUNDARY

As described earlier in Fig. 4, sudden change in drawdown is the difference between drawdown just before and after the impervious wall along a drawdown curve when crossing the impervious wall. Fig. 25 shows the relationship between sudden change in drawdown across impervious wall versus distance between well center and wall edge in construction pit with various sizes. The closer is the well from the impervious wall; the larger is the sudden change in drawdown across the impervious boundary. The sudden change in drawdown increases as the size of the construction pit decreases.

© 2016 Deep Foundations Institute

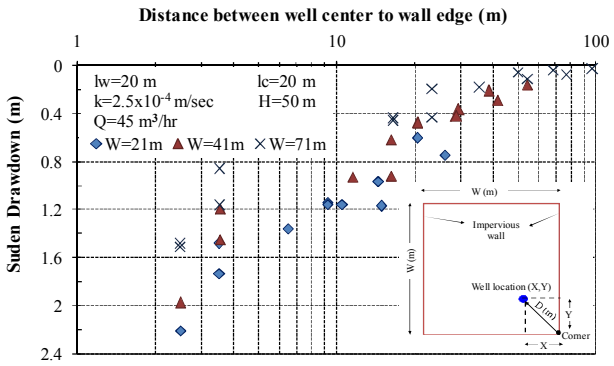


Fig. 25. Relationship between sudden change in drawdown across impervious wall versus distance from well center to wall edge

CONCLUSION AND CONCLUDING REMARKS

This paper focuses on dewatering systems that comprise with partially penetrating deep wells inside square areas enclosed by impervious walls partially penetrating unconfined aquifers. There are many factors that may influence the drawdown of a well in such dewatering system. A comprehensive parametric study of 3D Finite Difference Analysis was carried out to investigate the influence of all the parameters on the drawdown of a deep well in a dewatering system. This paper presents the results of the analysis showing the influence of different parameters on the drawdown of a single well. The following conclusions can be drawn based on the results presented in this paper:

The drawdown results depend on well location as compared to the impervious wall. As well gets closer to the impervious wall edge, drawdown increases.

The construction pit size has significant influence on the amounts of drawdown in general. As the size of the construction pit decreases, the drawdown increases

.

The closer the aquifer to the toe of the wall as reflected in an increase in lc/H, the drawdown increases.

The shallower the well as compared to wall depth, the higher is the drawdown. The increase in well penetration deeper than the wall does not seem to have significant influence on the drawdown.

The drawdown increases with the decrease in coefficient of permeability.

The drawdown increases with the increase in discharge rate of the well.

The sudden change in drawdown across the impervious boundary is influenced by all factors considered in this study in the same manner as the drawdown around a well is influenced.

ACKNOWLEDGMENT

The authors would like to express their deep gratitude to Prof. M. Hamza of Hamza Associates (HA) for providing the version of MODFLOW licensed to HA to carry out this study.

© 2016 Deep Foundations Institute

REFERENCES

Abdel Haleem, A. (2006). *Study of Deep Wells Dewatering System Enclosed by Partially Penetrating Cutoff Walls in Unconfined Aquifer*, M.Sc. Thesis, Civil Engineering, Public Works Department, Cairo University, Egypt.

Harbaugh, A. W., Banta, E. R., Hill, M. C., and McDonald, M. G. (2000), "MODFLOW-2000, The U.S. Geological Survey Modular Groundwater Model User Guide to Modularization Concepts and The Groundwater Flow Process".

Mohamed, D. R. (2012). *Effect of well depth and depth of surrounding impervious wall on drawdown of ground water control system in an unconfined aquifer*. M.Sc. Thesis, Civil Engineering, Public Works Department, Cairo University, Egypt.

Mohamed, D. R., M. Shahien and H. Hafez (2013). "Effect of well depth and depth of surrounding impervious wall on drawdown of ground water control system in an unconfined aquifer," Proceedings of 6th International Perspective on Water Resources and Environment, ASCE, Izmir, Turkey

Mohamed, D. R., Shahien, M. M. and Hafez, H. H. (2015). "Influence of Well Location on Drawdown in Area Surrounded by Vertical Partially Penetrating Cutoff Walls", Proceedings of Int. Conference on Advances in Structural and Geotechnical Engineering, Tanta University, Hurghada.

Driscoll, F. G., (1986). Groundwater and Wells, 2nd ed., Johnson Screens, Wheelabrator Water Technologies Inc., St. Paul, MN.

Freeze, R. A. and Cherry, J. A. (1979), Groundwater, Prentice-Hall, Inc.

McDonald M.G. and Harbaugh A.W. (1988). "A Modular Three-Dimensional Finite Difference Ground-Water Flow Model", U.S. Geological Survey Open-File Report 83-875.

Powers, J.P. "Construction Dewatering", Second Edition, J. Willy and Sons, 1992.

Preene M., Roberts T.O.L, Powrie W., and Dyer M.R., "Groundwater Control - Design and Practice", CIRIA Report No. C515, 2000.

NAVFAC, "Naval Facilities Engineering Command" Dewatering and Groundwater Control (NAVFAC P-418; TM 5-818-5 / AFM 88-5, Chapter 6), November, 1983.

© 2016 Deep Foundations Institute

GROUND FREEZING FOR GROUNDWATER CUTOFF AT THE SEATTLE SEAWALL REPLACEMENT PROJECT

Daniel Mageau, P.E. - dmageau@soilfreeze.com Larry Applegate – lapplegate@soilfreeze.com
Brenton Cook, P.E. – bcook@soilfreeze.com Aaron McCain, P.E. – amccain@soilfreeze.com
SoilFreeze, Inc., Seattle, Washington, USA, 206-420-2759

ABSTRACT

Ground freezing was successfully used to create an impermeable barrier wall along 3,000 feet of the downtown Seattle waterfront as part of the Seattle Seawall Replacement Project. This $360 million re-development involved removing the original 1930-era seawall and replacing it with a new, seismically resistant concrete wall supported on fill soil improved by jet-grout. The replacement wall was one of the largest construction projects in the state of Washington at the time. Ground freezing is typically used as both a structural shoring wall and groundwater cutoff. However, at this project it was used primarily as a cutoff wall to stop groundwater seepage into the 16-foot deep excavation from the landside areas.

The project was divided into ten boxes (sections), each typically 200 to 600 feet in length. Only a few boxes were under construction at one time to reduce the impact on tourist-dependent local businesses. Soil conditions in the freeze zone consisted of sand, gravel and silt fill with abundant wood and debris. Tidally influenced groundwater, much of it contaminated, was difficult and expensive to control using wells during initial stages of construction. For all subsequent stages, a frozen soil wall extending to about 35 feet in depth was used to stop groundwater seepage into the excavation. An additional benefit of ground freezing included elimination of potential settlement beneath nearby buildings, utilities, and the Alaskan Way Viaduct that would likely have occurred with the use of conventional dewatering wells. The frozen soil wall crossed more than 130 buried utility locations (including water, sewer, storm, electrical, fiber optic, phone, gas, steam) with no damage. The frozen soil wall was very successful and limited groundwater influx from the landside to the seawall excavation to a negligible amount so that pumping, storage and treatment of contaminated groundwater was essentially eliminated.

Keywords: ground, freezing, frozen, soil, contaminated, groundwater, cutoff, barrier

INTRODUCTION

The original seawall was built along the busy Seattle waterfront in the 1930's. After the 2001 Nisqually earthquake near Seattle it was discovered that this timber-pile supported concrete structure was severely deteriorated, unstable (particularly under seismic shaking) and needed replacement. A key reason for this instability was the presence of 40 to 80 feet of highly liquefiable sandy fill. Numerous engineering analyses predicted that under even a moderate earthquake, this loose sand fill would spread laterally ten or more feet into Puget Sound, taking the seawall with it. The City of Seattle and their engineering consultants subsequently designed a new, stouter seawall that would replace the 80-year old structure. In 2013 the general contractor for the City, began the large task of replacing the historic seawall structure with a new 3,400-foot long concrete seawall. Figure 1 shows the overall project site plan.

The project site was divided into 10 boxes, each typically ranging from about 200 to 600 feet in length. The excavation in each box was approximately 18 feet deep and 30 to 40 foot wide. Only Boxes 2 through 10 have been constructed – Box 1 is planned for future construction. The entire width of the project site is only about 70 to 90 feet, which includes excavation for the new 40+ foot wide seawall (delineated by a tied-back soldier pile wall along the east), a narrow construction access road, construction equipment and supplies for five subcontractors and the frozen soil wall system.

© 2016 Deep Foundations Institute

The two-level Alaskan Way Viaduct was located along most the west side of the project and the Puget Sound along the east side, as shown in Fig. 1. The frozen soil wall bordered many of the foundations of this aged structure protecting it from construction impacts. Also, a ferry dock and numerous commercial buildings servicing an active tourist area were situated on piles over the water along the entire waterfront. Seawall construction was therefore staged so that only a few boxes were under construction at one time, and construction during the busy summer tourist season was stopped to limit disruption to these commercial enterprises.

An additional complexity for the contractors was the protection, moving or replacement of hundreds of buried utilities within the project site, some of which are primary lines for the City and the state. Buried utilities included water, sewer, storm, electric, gas, fiber option, telecommunications, and steam.

The replacement wall construction project consisted of excavating down about 16 feet to remove the original seawall structure and part of the timber pile caps, stabilizing the liquefiable soil using thousands of jet grout columns, and then building the new seawall structure. A typical cross section of the complex seawall replacement project is shown in Fig. 2. Much of excavation and ground improvement was accomplished below groundwater level – thus groundwater control became a very significant issue for this project. As discussed in the following sections, a frozen soil wall proved to be the perfect solution for preventing groundwater seepage into the excavation during construction of the new seawall.

GROUNDWATER CONTROL ISSUES

The soils in the upper 50 top 80 feet typically consisted of very permeable, loose sand/gravel/silt fill with abundant debris (logs, wood, concrete, rebar, cinder, bricks, etc.). Groundwater level typically ranged from about 8 to 11 feet bgs and a significant portion of the groundwater was contaminated with hydrocarbons. Groundwater levels were controlled by tidal action of Puget Sound on the west and groundwater seepage from downtown Seattle hillside on the east. Soils with higher fines content and lower permeability (sandy silt and silt) were commonly present below a depth of about 25 to 30 feet. These higher silt content soils were fairly effective at limiting groundwater seepage from below. However, because the excavation for the seawall replacement extended 5 to 8 feet below groundwater level, well into the highly permeable fill soil, control of horizontal groundwater seepage into the excavation proved to be a key component of this project. A deep sheet pile wall was driven along the entire length of the project approximately 10 feet west of the replacement seawall face (see Fig. 2) to control/reduce inflow of Puget Sound water into the excavation.

For the other three sides (east, north and south), conventional dewatering wells were used in an attempt to control/reduce landside groundwater seepage for the initial two Boxes (Nos. 9 and 10 with a combined length of 420 feet). These two boxes were completed during the first construction season using soldier pile and lagging shoring for the excavation. Controlling water into the excavation proved to be very difficult for these first two boxes. First, water seepage through the sheet pile wall on the west was higher than desired. Additional pumps were required to keep sea water that seeped through seams in the sheets from flooding the excavation. Fortunately, this water was ‘clean’ and required minimal treatment and no storage before it was pumped back into Puget Sound.

Unfortunately, control of groundwater seepage on the landside using wells was much less effective than expected because of the very permeable fill soils. Groundwater discharge rates were over several hundred gallons per minute (gpm) for these two relatively small boxes. Even with the dewatering wells working at full capacity, it was difficult to maintain dry conditions in the excavation. Of greater concern was the fact that storage and treatment of this volume of contaminated water required a multitude of large temporary storage tanks and extensive treatment time, resulting in very high costs. When extrapolated to the remainder of the 3,400 foot long project site, it became evident that costs and schedule issues related to

© 2016 Deep Foundations Institute

dewatering would be extremely problematic. Therefore, a temporary frozen soil wall was used to control landside groundwater into the excavation for Boxes 2 through 8. The frozen soil wall had a combined total length of over 3,000 feet.

FROZEN SOIL CUTOFF WALL

The frozen soil wall for this project was designed to cut off or significantly reduce groundwater seepage from the east (landside) of the excavation. This would essentially eliminated the need for dewatering wells and storage/treatment of contaminated water. Seawater inflow from the west (Puget Sound) was still restricted by the deep sheet pile wall with a separate pumping/treatment system, as previously mentioned. Groundwater from the east was cut off by the creation of a north-south running frozen soil wall along each box. This frozen wall was located about 40 feet east of the soldier pile and lagging wall that provided structural shoring for the seawall excavation. Groundwater seepage into the excavation along the north and south ends of the boxes was mitigated by installing east-west transverse walls that connected the sheet pile wall to the main frozen soil wall. The transverse walls were a combination of sheet piles (west half within the excavation area – structural and groundwater control) and a frozen soil wall (east half – non-structural, groundwater cutoff only). A plan view of a typical layout for the sheet pile, soldier pile and frozen soil walls (including freeze pipe locations) is shown in Fig. 3.

The frozen soil cutoff wall extended approximately 35 feet below ground surface and was located 30 to 50 feet from the excavation area. Frozen soil walls were formed by circulating calcium chloride brine chilled to -15^{o}F or colder through a series of predominately vertical steel freeze pipes installed at approximately 2.5 feet on center along the freeze pipe alignment (see Fig. 3). Freeze pipes were 3-inch diameter, Schedule 40, welded steel sections with a steel plate welded to the bottom. Over 1,400 steel freeze pipes were required. As chilled brine flowed through each freeze pipe, soil around the pipes froze radially outward until a solid wall of frozen soil was created along the three sides of the excavation area.

Two typical sections through the frozen soil cutoff wall are shown in Fig. 4. In areas absent of buried utilities (over ninety percent of the frozen soil alignment), the freeze pipes were vertical as illustrated in the drawing on the right side of Fig. 4. However, at most of the buried utility crossings it was necessary to add extra freeze pipes, many of them angled, to provide adequate freezing around and below the buried utilities. This is illustrated in the drawing on the left side of Fig. 4. Angled pipes allow a continuous frozen soil wall to be created next to and below the buried utility without interrupting service and without damage to the utility. Frozen soil formed with a combination of vertical and angled pipes results in a water-tight seal around buried utilities. Precautions, such as a detailed utility location program, were taken to ensure that the freeze pipe installation did not damage buried utilities. In addition, special freeze pipe designs near active water lines were required to protect these utilities. These measures are discussed in more detail in the section below entitled “Freeze System Installation.” Frozen soil walls around buried utilities illustrate high flexibility of the system, not available to most other conventional shoring methods.

Numerous vehicle and construction traffic crossed the frozen soil wall. In these areas, a shallow trench was pre-excavated and the freeze pipes were installed at the bottom of the trench. After the freeze system was hooked up and operating, steel plates were placed over the trenches to allow traffic over the freeze system. Freeze pipes in a trench are illustrated in vertical freeze pipe drawing on the right side of Fig. 4.

Frozen soil is essentially impermeable; therefore, groundwater flow through the frozen soil wall was negligible. Groundwater seepage below the frozen soils wall was cut off or significantly restricted by the presence of layers of low permeability silt and/or sandy silt located below about 25 feet in depth. Seepage into the excavation along or below the frozen soil wall was therefore non-existent or minor. Any seepage that did occur could be easily handled with small ditch pumps.

© 2016 Deep Foundations Institute

THERMAL ENGINEERING ANALYSIS

We completed numerous thermal analyses to evaluate certain aspects of freezing at this site. The primary purpose of these analyses was to evaluate the expected growth of the frozen soil wall and the time required to attain a fully formed barrier (termed 'freezedown'). We also evaluated the impact of freezing on certain temperature-sensitive buried utilities (i.e., inactive water lines). For these analyses, we used the 2D computer program TEMP/W developed by Geo-Slope International (Version 8.0 included as part of GeoStudio 2012 software package). TEMP/W models thermal changes in the ground, including frost growth around a chilled freeze pipe resulting from below-freezing temperatures.

We delineated the soils into two basic units for thermal analysis: 1) saturated, silty sand and gravel and 2) wet soft silt. Thermal soil properties, including moisture content, thermal conductivity, heat capacity, unfrozen moisture content, and initial temperature, were evaluated from available geotechnical data on soils.

Tidal action impacts on groundwater level along the frozen soil wall alignment, which was over 70 feet from Puget Sound, was typically less than a foot. This level of tidal action does not appreciably impact the freezedown because a) tidal groundwater movement is slow and b) the direction of flow is back and forth twice a day (not continuous in one direction). This allows time for the ground to freeze solid. Measured values of chloride in four samples recovered from borings near the alignment were less than 900 ppm, which represents a salt content of about 5% of sea water. This amount of salt in the groundwater has negligible impact on freezing.

Figure 5 shows the results from the TEMP/W analyses for the two soil types (silty sandy gravel and soft silt) after 3 weeks of freezing. This figure is a plan view of a horizontal section cut through the frozen soil zone at two different depths, each representing a different soil type. The small red triangles represent 3-inch diameter freeze pipes. Each small square/triangle on the plot represents a soil element, each with specific thermal properties. The blue dashed line represents the 32^{o}F isotherm and the other lines and colors on the figure represent other isotherms. The results in Fig. 5 indicate a solid frozen soil wall with a thickness of 5+ feet in the silty sand and gravel layer and 3 feet in the underlying soft silt layer. Frozen soil typically forms slower in soils with higher silt contents because these soils have lower thermal conductivity properties and higher moisture contents than sand/gravel. These differences translate into more energy needed to freeze silt than sand/gravel. A minimum wall thickness of 3 feet is typically used for groundwater cutoff; therefore 3 weeks was determined to be the minimum freezedown time before excavation below the groundwater table was allowed. The frozen soil wall continued to grow outward, albeit at a slower rate, after the initial freezedown.

FREEZE SYSTEM INSTALLATION

The freeze pipe alignment was almost entirely within a 10 to 12 foot wide corridor paralleling the Viaduct columns. An extensive utility location program was completed prior to the start of freeze pipe installation to prevent damage to buried utilities. As previously mentioned, there were over a hundred locations were the frozen wall crossed a buried utility – all were within 3 to 15 feet of ground surface. First, the required one-call to the state utility service provided marks on the ground for many (but not all) of the utilities. Second, available utility drawings were evaluated and compared with marks on the ground. Many buried utilities were either too new (not sufficient time for utility company to add to plans) or too old to be accurately portrayed either on the ground by the one-call service or on the utility drawings. Therefore, the third and most critical part of the utility location program was potholing every potential crossing. This was accomplished by excavating a small to large hole to visually identify every utility that the frozen wall crossed or bordered. High suction vacuum trucks were used for this purpose. After the utility was identify and the size and depth measured, a PVC pipe was installed on either side of the utility to delineate the

© 2016 Deep Foundations Institute

location and the hole was backfilled. Figure 6 shows photos of potholing and PVC sleeves near utilities. An exception was near water lines where the ground around the line was completely excavated, leaving the water line exposed to the air throughout the freezing process. This prevented the water lines from freezing. Freezing of all other utility lines was, for the most part, not a concern since freezing does not adversely impact utilities that do not convey water or sewer/storm lines that convey warm, moving water that does not fill the pipe.

The steel freeze pipes were installed with two methods. A small pin pile driver, which pounds the pipe into the ground via a hydraulic hammer, was used for most of the freeze pipes. This method works well in loose to medium dense soil and is quite efficient (up to 30 freeze pipes per day). In many locations, however, obstructions such as concrete pieces or large timbers that were not sufficiently cleared with pre-spudding resulted in refusal using the pin-pile method. In these cases a sonic drill rig was used to penetrate a hole in the ground and obstacles so that a freeze pipe could be installed to design depth. The sonic rigs were able to drill through any obstacle encountered at the site, including reinforced concrete sections. Photos of the two drill methods are shown in Fig. 7 (overhead power was turned off during freeze pipe installation). The two systems worked very well and all freeze pipes were successfully installed within schedule.

Refrigeration chillers were turned on and ran full capacity to initially freeze the ground (3 to 4 weeks). A photo of the chiller and freeze system brine lines is shown in Fig. 8. After the frozen wall was fully formed (as indicated by ground temperature measurements – see below), the chillers were adjusted to reduce the electrical usage by about half. This resulted in slightly warmer frozen wall temperatures, however, the frozen wall/cutoff integrity was not impacted.

Numerous temperature pipes were installed within the frozen wall alignment (over 60 total) to monitor the temperature of the ground during freezing. These consisted of 3-inch steel pipes, about 35 feet in depth, with a temperature string inserted inside. Each string had thermocouple sensors attached at 3-foot intervals. All sensors were connected to a data acquisition system that stored the data and transmitted it back to the SoilFreeze office a few miles away. In addition, key performance information for the chillers (brine temperature, pressure, etc.) was transmitted to the office. This allowed the engineers to monitor frozen soil wall performance at a very detailed level. Any problems, such as warming of brine, electricity disruption, etc., were identified in a timely manner and steps were taken to mitigate the problem, such that there was no impact to groundwater barrier integrity.

CONCLUSIONS

Figure 9 shows a typical plot of ground temperature versus time for a point in the frozen wall in between two freeze pipes. This is the warmest point along the frozen soil barrier centerline; soils between this point and the freeze pipes are always colder. As seen, the frozen wall temperatures dropped below freezing rapidly in the first few weeks than slowly decreased with time after that, as predicted in the thermal analysis.

The frozen wall worked very well to cutoff groundwater into the excavation during the new seawall construction. Figure 10 shows overviews of the site during excavation in Box 4. The chiller on the far left of the photo is along the frozen soil wall alignment (i.e., along the west edge of the viaduct). The depth of the excavation in this photo is about 5 to 8 feet below groundwater level. The water at the bottom of the excavation is mostly rain water that was removed via small pump. The general contractor completed the project within a very tight schedule with little to no effort for controlling or treating contaminated groundwater seepage from the landside areas of the site.

© 2016 Deep Foundations Institute

ACKNOWLEDGMENTS

The authors would like to acknowledge Mortensen-Manson JV and the City of Seattle for their vision and flexibility in adopting ground freezing for this complex project. Also, they would like to acknowledge McDowell NW, Inc. (pin-pile driving) and Cascade Drilling (sonic drilling), who installed the 1,400+ steel freeze pipes for the project.

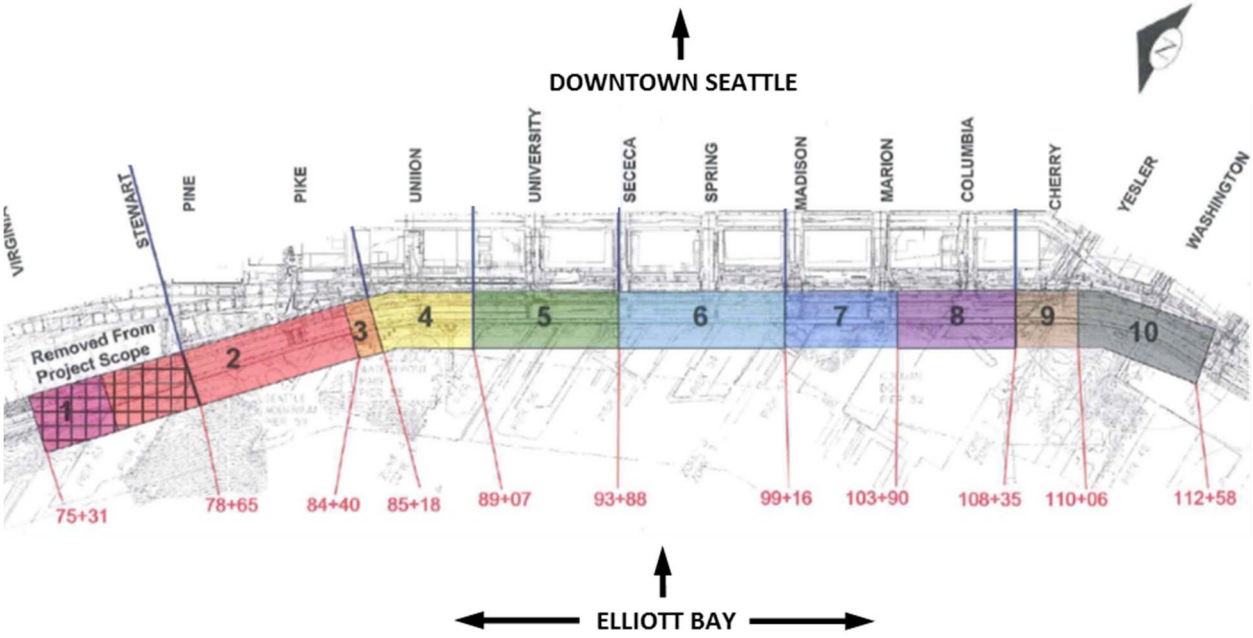


Fig. 1. Plan view of project site showing box locations

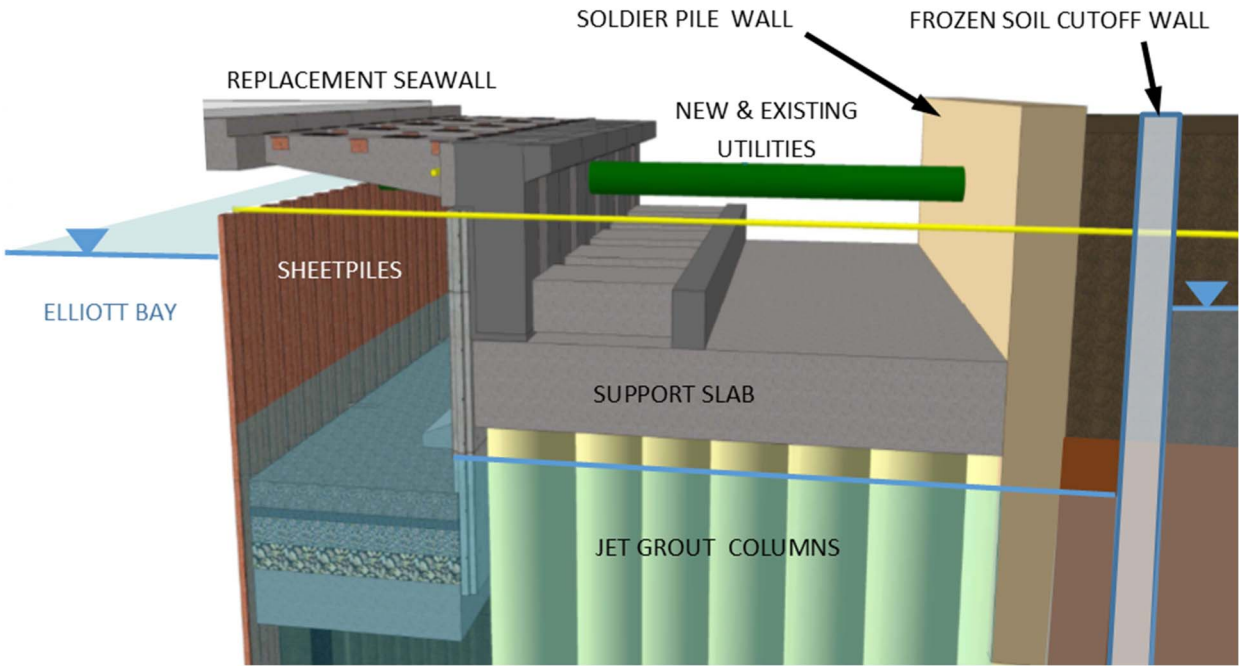


Fig. 2. Cross-section showing construction components of replacement seawall

© 2016 Deep Foundations Institute

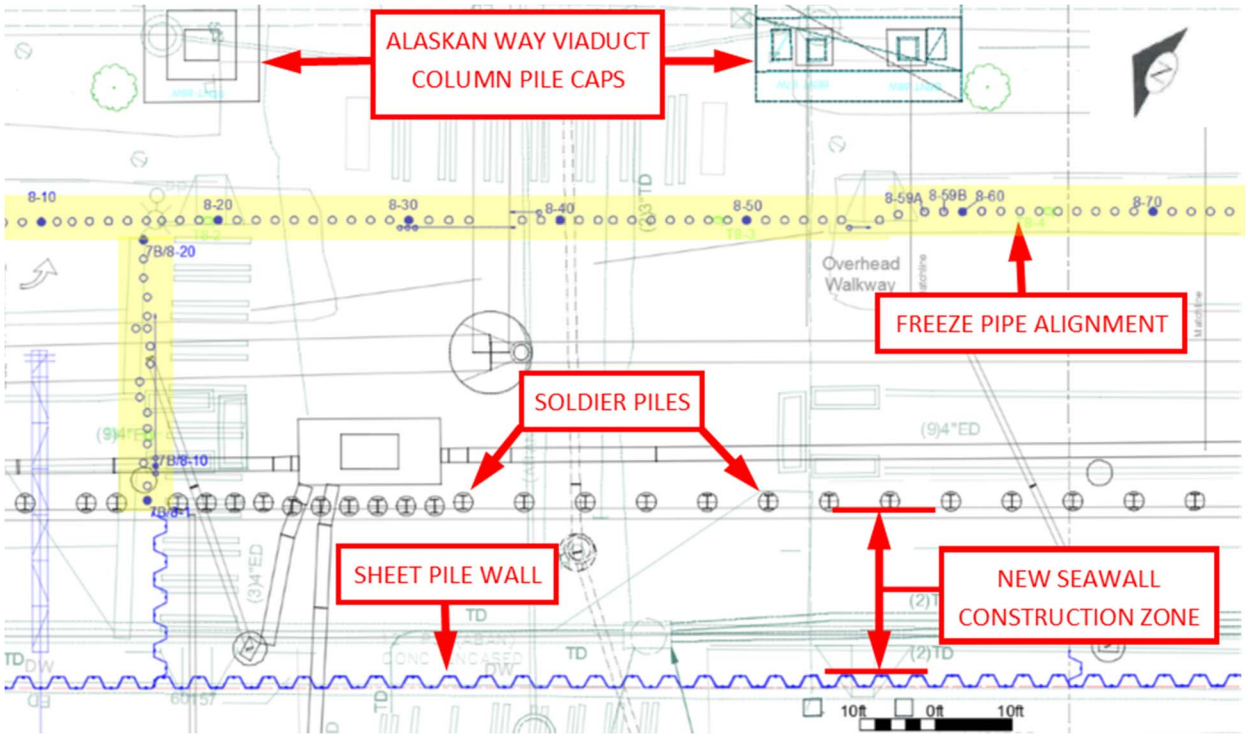


Fig. 3. Plan view showing as-built freeze pipe alignment and new seawall construction zone

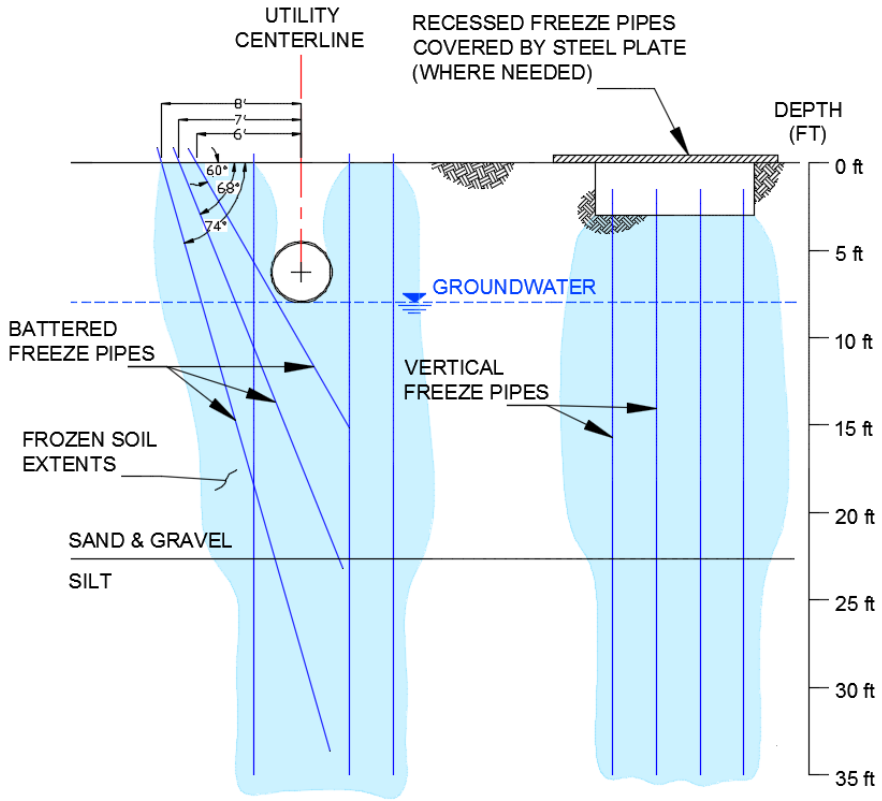


Fig. 4. Typical freeze pipe installation sections

© 2016 Deep Foundations Institute

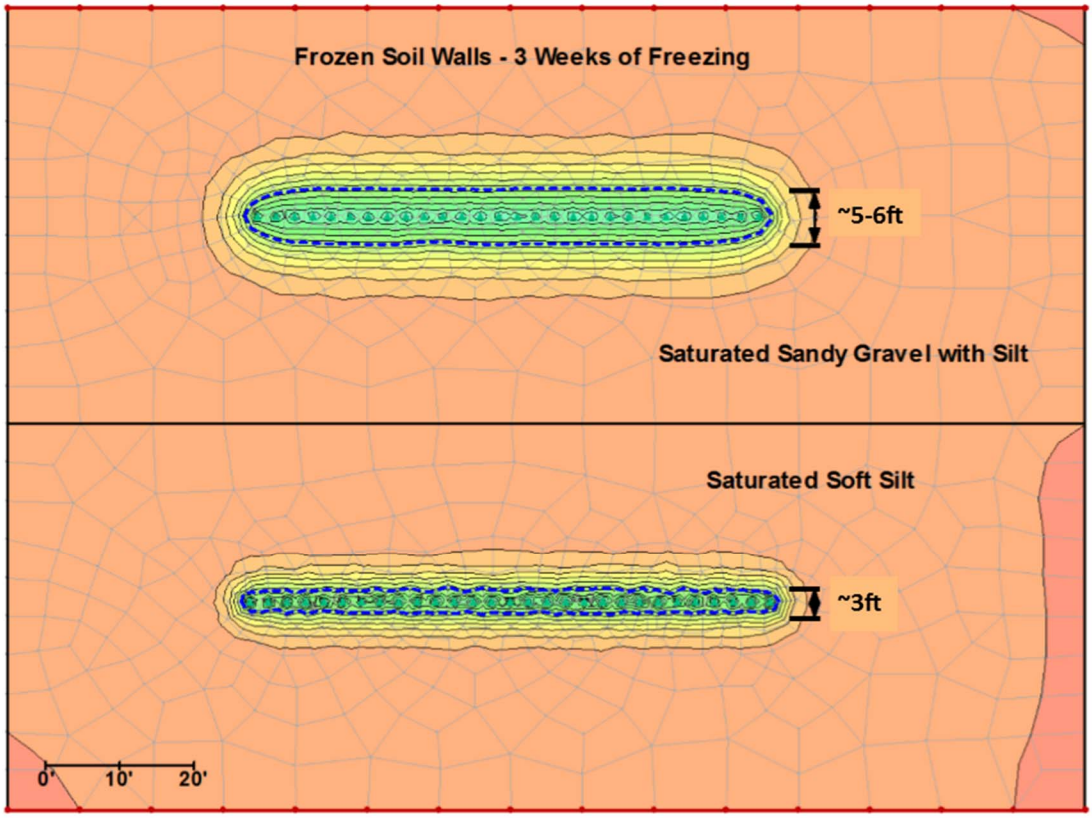


Fig. 5. Plan view of thermal model results showing wall thicknesses



Fig. 6. Potholing and insulating existing utilities

© 2016 Deep Foundations Institute



Fig. 7. Photograph showing equipment used for freeze pipe installations



Fig. 8. Photograph showing activated freeze system

© 2016 Deep Foundations Institute

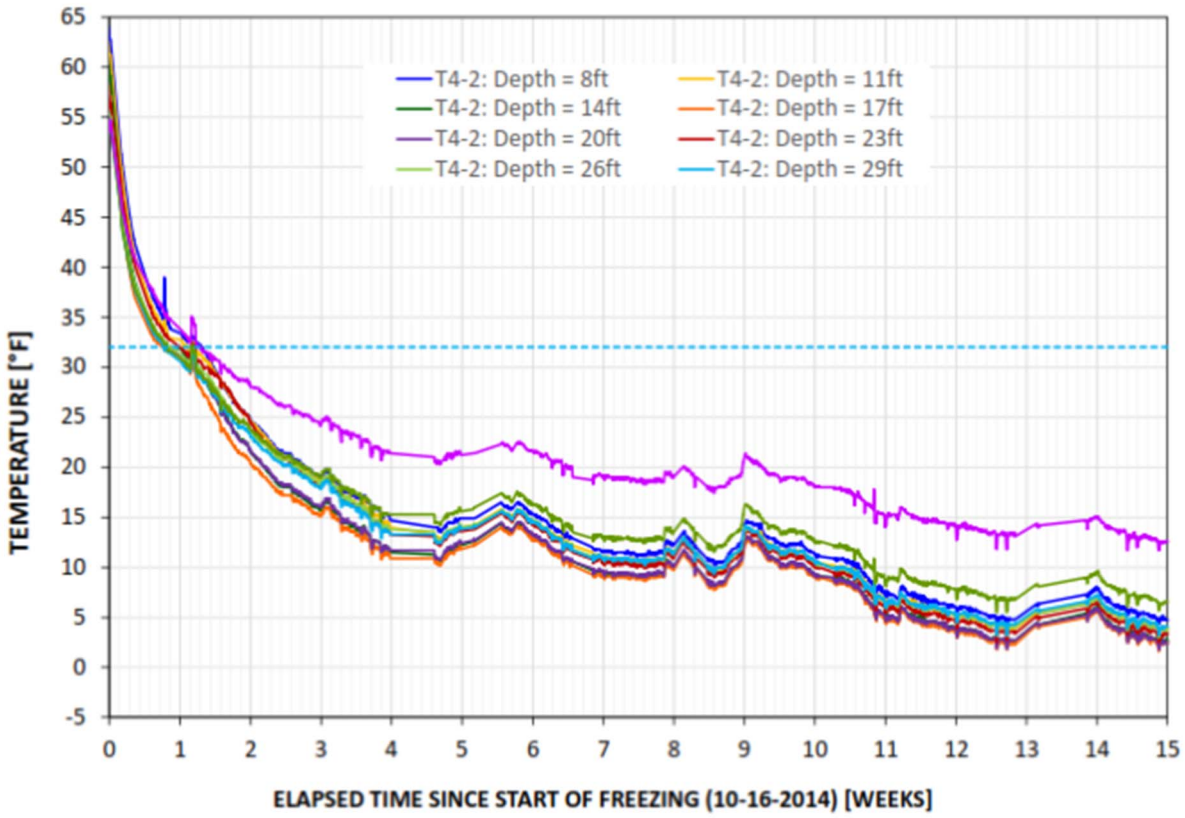


Fig. 9. Plot of measured ground temperatures versus time



Fig. 10. Photograph showing excavation for seawall replacement

© 2016 Deep Foundations Institute

GROUTING OF AN ARTESIAN ZONE IN ADVANCE OF POWERHOUSE EXCAVATION FOR THE RED ROCK HYDROELECTRIC PROJECT

Rachael V. Bisnett, P.E., MWH Global, Chicago, IL, USA, (312) 831-3188, rachael.v.bisnett@mwhglobal.com
Thomas G. Andrews, P.E., MWH Global, Chicago, IL, USA, (312) 831-3022, thomas.g.andrews@mwhglobal.com
Dewey W. Geary, P.E., MWH Global, Pella, IA, USA, (641) 620-2426, dewey.w.geary@mwhglobal.com
R. Michael Bivens, P.E., Rembco Geotechnical Contractors Inc., Knoxville, TN, USA, (865) 671-2925, mike.bivens@rembco.com

ABSTRACT

The powerhouse for the Red Rock Hydroelectric Project is being constructed at the toe of the existing U.S. Army Corps of Engineers' Red Rock Dam immediately adjacent to the existing stilling basin retaining wall. The powerhouse excavation extends though overburden and 40 feet into rock. Prior to rock excavation, a grouting was performed around the excavation for the purpose of increasing the strength of zones of fractured rock. For the majority of the perimeter, the rock was fairly tight and grout takes were relatively low. However, for a 160 foot extent along the landside of the excavation, extensive grouting was required to treat a two to five foot thick zone characterized by artesian flows from grout holes on the order of 20 gallons per minute (gpm) or higher from a voided zone in rock that was partially filled with sand, clay, and rock fragments. This paper discusses the treatment measures used to fill and improve the strength of this zone, which ultimately entailed a combination of thick cement grout and polyurethane grout, holes spaced as close as 18 inches on-center, and two additional rows of grout holes.

Keywords: artesian, excavation, grout, gypsum, seepage, void

INTRODUCTION

The Red Rock Hydroelectric Project (Project) is currently under construction at the existing U.S. Army Corps of Engineers' (USACE) Red Rock Dam on the Des Moines River near Pella, Iowa. At peak generation, the Project will generate up to 55 MW with an average annual energy output of 178 gigawatt-hours.

The powerhouse for the Project will be located at the toe of the existing dam immediately adjacent to the existing stilling basin retaining wall. The powerhouse excavation extends up to a depth of 40 feet into rock through the St. Louis Limestone and terminates in the Warsaw Limestone. Prior to rock excavation, grouting was performed around the excavation for the purpose of improving the strength of zones of fractured rock. An auxiliary benefit of the grouting was reducing seepage into the excavation. Grouting began on July 20, 2015 and ultimately concluded on September 26, 2015.

Drilling and grouting occurred as planned on the riverside (southwest) of the excavation along the cellular cofferdam and stilling basin wall (grout holes P-1 through S-13) and on the upstream (northwest) side of the excavation along the secant pile excavation support wall (grout holes P-14 through S-19) (refer to Fig. 1). The rock in these areas was generally tight with relatively low grout takes. However, on the landside (northeast) of the excavation (grout holes P-20 through P-28), partially infilled voids and artesian flows were encountered during drilling generally between depths of about 28 to 40 feet.

© 2016 Deep Foundations Institute



Fig. 1. Downstream excavation (looking northwest, landside is to the right)

Ultimately, fifth order (quintenary) split-spaced grout holes along the original row and a second row of grout holes, in combination with modified drilling and grouting methods, were required to achieve closure in this area. Refer to Fig. 2 for the layout of additional grout holes along the landside of the excavation (right side of the excavation as shown in Fig. 1).

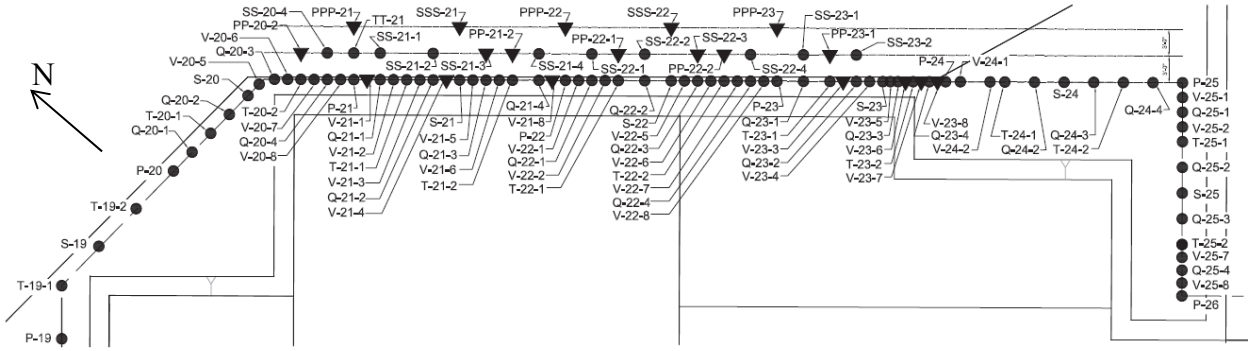


Fig. 2. Plan view of additional grout holes along the landside of the excavation

GEOLOGY

Bedrock at the site is comprised of the Warsaw Limestone and St. Louis Limestone. The St. Louis Limestone, Mississippian in age, consists of alternating beds of limestone, sandstone, dolomite and discontinuous basal evaporite beds. The St. Louis Limestone is present from the top of rock at about El. 672 down to about El. 637. Below El. 637, the Warsaw is part of the Augusta Group of Mississippian age, which consists of dolomitic and argillaceous shales.

The existence of discontinuous, water-soluble evaporite (gypsum and anhydrite) deposits at the base of the St. Louis Limestone has been an issue at the dam since its design and construction in the 1960s. As a result

© 2016 Deep Foundations Institute

of known gypsum and anhydrite deposits, the USACE has performed remedial grouting from the left abutment along an approximately 1,000 ft length of the dam's embankment. USACE drilling records from areas of known evaporite deposits typically show brecciated rock, lost circulation, core loss, apparent voids, or zones of very rapid drilling. Fig. 3 shows the approximate distribution and thicknesses of known evaporite bodies at Red Rock Dam relative to the location of the new hydroelectric project.

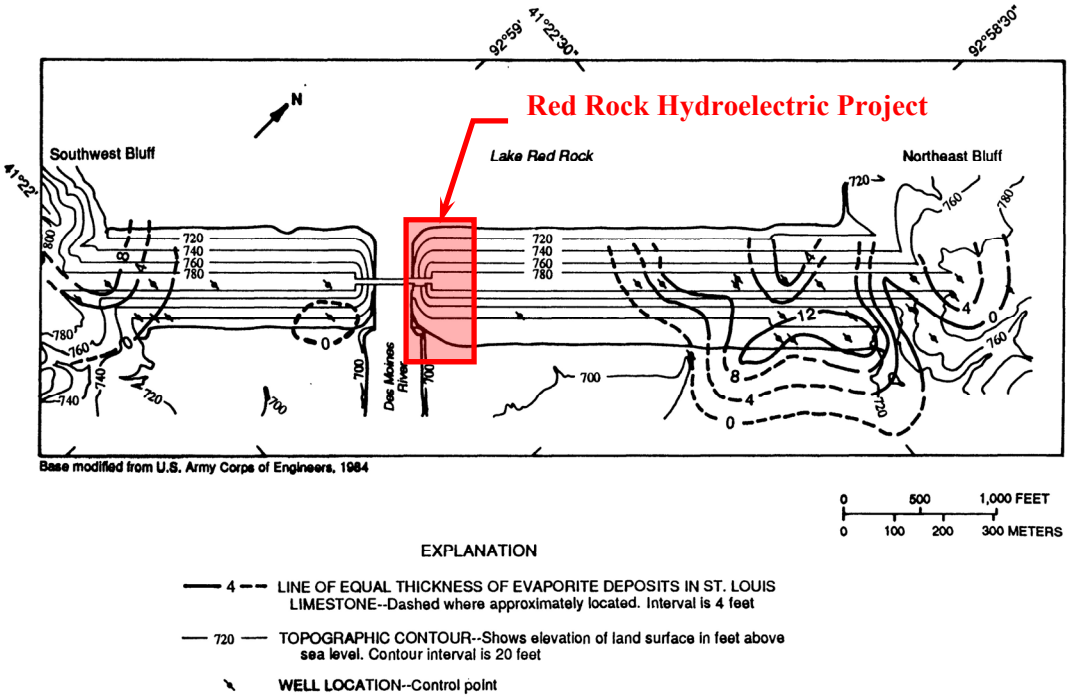


Fig. 3. Inferred evaporite deposits at Red Rock Dam (Lucey 1991)

USACE drilling records from borings in the vicinity of the hydroelectric project did not suggest the presence of evaporite deposits based on the lithologic description, excellent recovery, and good water return during drilling. No direct evidence of evaporites was observed in the supplemental borings performed prior to construction, but one boring performed in the area did experience loss of drill water return, rapid coring, and approximately 1 foot of core loss around El. 640. This was considered to be an anomalous occurrence when viewed as part of the larger data set including the USACE's extensive drilling records. However, five exploratory boreholes performed in the area during the initial stages of construction experienced lost circulation, zones of rapid coring, and core loss in localized intervals within a horizon between about El. 645 and El. 635 along the bottom of the St. Louis Limestone (circulation was typically lost above the zone of no core recovery).

DRILLING AND GROUTING METHODS

Drilling

A track-mounted drill rig and a 4 inch down-the-hole-hammer was used for all of the drilling along the river and upstream sides of the excavation and was initially also used along the landside. However, since air communication was observed between open holes across distances of up to fifty feet following commencement of drilling with the down-the-hole-hammer along the landside of the excavation, subsequent drilling along this side was performed using rotary methods with a tri-cone roller bit. Rotary drilling limited the potential influence of drilling on nearby grouting activities, such that they could occur

© 2016 Deep Foundations Institute

concurrently in the limited space available, and minimized disturbance of the partially in-filled voids. The holes along the landside generally produced artesian flows; there were no instances of lost circulation, unlike what was seen in some of the exploration borings. No signs of material transport were observed in the flows emerging from the drilled holes.

Primary and secondary holes were generally drilled at a 20 degree angle from vertical, inclined away from the excavation. Higher order holes along the landside of the excavation were all drilled vertically.

The grouting alignment was located approximately five feet outside of the planned excavation face. Primary holes were located at a spacing of 24'-0" with required secondary holes located split-spaced between the primary holes. Higher order holes were split-spaced as shown in Fig. 4.

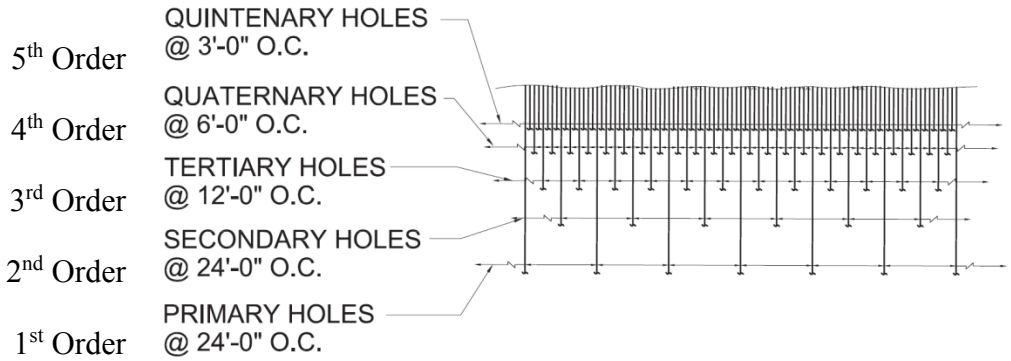


Fig. 4. Typical grout hole arrangement detail

Cement Grouting

Grouting was generally performed using the ascending stage method, which consists of drilling the hole to the full depth and using a packer set at progressively shallower depths to isolate and grout consecutive stages of the hole. Where large voids or artesian flows were encountered, that stage was grouted prior to drilling the hole to its planned depth and commencing with ascending stage grouting. Four stable cement grout mixes, which included bentonite and diutan gum and varying amounts of superplasticizer to modify the viscosity, were initially developed, as shown in Table 1.

A suite of supplemental low mobility grout mixes, as shown in Table 2, were also developed in the field to fill open features under flowing groundwater conditions after a sand-cement mix was tremied into one hole with limited success. The thickest low mobility mix was modified to add the anti-washout admixture V-Mar for the last grout mix. Trial samples of these supplemental low mobility mixes were placed in flowing surface water to evaluate washout resistance and mobility. Aggregate and bentonite were not used in any of the supplemental mixes.

Table 1. Standard grout mixes

Mix Designation	Unit Weight (pcf)	Marsh Funnel Viscosity (sec)
A	92.6	35 – 40
B	92.7	40 – 55
C	92.7	55 – 70
D	92.3	> 150

© 2016 Deep Foundations Institute

Table 2. Supplemental low mobility grout mixes

Water (gal)	Cement (bags)	Diutan Gum (oz)	V-Mar (oz)
35	4	0 – 4	-
25	4	0 – 3	-
25	5	0 - 2	-
25	5	0 - 2	32

Rembco (grouting subcontractor) used a custom-built data collection and recording device to display and record real-time data including grouting Lugeon values and at-depth grouting pressures for up to four channels of simultaneous grouting. Field trials were performed to verify the properties of the grout mixes and perform calibration of the data acquisition system. The field calibration accounted for grout viscosity, hose losses, elevation head from the manifold to the packer, and groundwater head.

Threshold (limit) injection pressures were defined in the technical specifications and are shown in Table 3. Grouting of a stage was generally considered complete when the flow rate reduced to 0.5 gpm or less when pumping at the threshold pressure. In general, split spaced holes were required adjacent to holes with a grout take exceeding 25 lb/ft of cement solids. Maximum grout take limits were established on a hole-by-hole basis in the areas where the voided feature was encountered to localize treatment.

Table 3. Threshold grout injection pressures

Depth of Stage (ft)	Threshold Injection Pressure (psi)
0 – 15	5
15 – 30	30
> 30	60

Chemical Grouting

High grout takes and artesian flows continued during cement grouting of fourth and fifth order holes on the landside of the excavation, so additional grouting was performed with a hydrophilic polyurethane grout. In ideal laboratory conditions, the grout is capable of expanding up to 30 times the original volume upon contact with water. The expansive pressure of the grout itself ranges from about 3 to 5 psi.

The Mountain Grout Flexible System polyurethane grout used was a hydro active, or a moisture cured system. Water is the catalyst that initiates the chemical reaction, so the polyurethane reacts and expands following the path of least resistance. In water control applications, this is typically the seepage path itself.

Like cement grout, the chemical grout was pumped into the hole through a packer. The following limits were used for injection of the chemical grout:

- Threshold injection pressure of 1 psi/ft.
- Maximum injection volume of 50 gallons (one drum) per hole.

Prior to injection, the grout was pumped through the hose and packer to determine the pump pressure required to overcome the internal friction of the system. The effective injection pressure was taken as the differential between the maximum pump pressure during injection and the maximum pressure measured during calibration.

© 2016 Deep Foundations Institute

GROUTING RESULTS

P-1 through S-19

Primary holes were grouted in three stages with packer depths generally corresponding to the specified allowable threshold grout injection pressures shown in Table 3. Grouting began in each stage with Mix A. Where pressure could be built to the threshold injection pressure, this mix was used until refusal. In general, only Mixes A and B were used along the river and upstream sides of the excavation. Secondary holes were not required based on primary hole grout takes.

Grout takes in the Warsaw Limestone indicated that the unit is very tight. Along the secant pile wall face shallower secondary holes were drilled and grouted to increase the strength of the St. Louis Limestone below the wall. Average grout takes between P-1 and S-19 are presented in Table 4.

Table 4. Average grout takes between P-1 and S-19

Bedrock Unit	Approx. Elevation	Average Pounds of Cement per Foot
Weathered Sandstone	> El. 663	23.0
St. Louis Limestone	El. 663 – El. 635	10.9
Warsaw Limestone	< El. 635	4.9

P-20 through P-28

During drilling of P-24, artesian flow was encountered around El. 644. Drilling continued until the top of a void was encountered at approximately El. 641. The drill rod was slowly lowered until resistance was encountered again around El. 636. Steady-state artesian flow out of the hole was on the order of 20 gpm, and the water was observed to be clear. Approximately 2000 gallons (270 cubic feet) of progressively thicker grout was pumped into the hole; grouting pressures were observed to build and release throughout. Down-hole probing the next morning indicted that the grout did not successfully fill the feature. A total of 32 cubic yards of sand-cement was then tremied into P-24 which reduced artesian flows. During drilling of P-20, artesian flow was encountered around El. 646 and a hydraulic connection was observed between P-20 and P-24. P-20 was grouted with about 2600 gallons (350 cubic feet) of Mixes A, B, C, and D.

The drilling logs prepared by Rembco indicated that apparent voids or zones with minimal resistance to drill advancement were encountered in all of the primary grout holes drilled from P-20 through P-24. The elevations of the apparent voids vary, but generally were encountered between depths of 28 to 40 feet below the top of rock, which correlates to about El. 645 to El. 635. Initially, many of the drilled grout holes produced upwards of 30 gpm of artisan flows and grout takes in nearly all of the primary and secondary grout holes in this area exceeded 800 gallons (100 cubic feet).

Due to the voided zone encountered and large grout mixes, the grouting was extended along the landside of the excavation to P-28. Evidence of the voided horizon was not encountered during drilling of P-27 or P-28 and grout takes were relatively low, so grouting was not extended further.

Grouting along the landside of the excavation initially began in each stage with standard grout Mix A, progressively moving up to Mix D as grout takes increased with only limited pressure build up. Since only minimal pressure increases could be achieved using Mix D, it became apparent that a more viscous grout

© 2016 Deep Foundations Institute

with greater cohesion was required to fill potentially open spaces in rock and to be resistant to washout by potentially flowing water. Therefore, the supplemental low mobility grout mixes were primarily utilized beginning with the tertiary holes.

As the grout program proceeded, most of the drilled holes no longer produced water and those that did mostly produced less than 1 gpm. Steady-state seepage from the drilled holes remained clear throughout the drilling and grouting program. Closure was generally achieved in the upper stages (0 to 30 feet depth) of the primary and secondary holes from holes P-20 through P-28. Consequently, most of the tertiary holes were pressure grouted only within the voided zone (below about 30 ft), and the upper portion of the holes were tremie grouted as the packer was removed from the hole. Due to the large takes below a depth of about 30 feet, up to fifth-order (quintenary) holes were performed along the original grouting alignment.

Average grout takes between P-20 and P-28 are presented in Table 5. Actual grout takes for P-20 through P-28 are presented on Fig. 5.

Table 5. Average grout takes between P-20 and P-28

Bedrock Unit	Approx. Elevation	Average Pounds of Cement per Foot
Weathered Sandstone	> El. 663	159.4
St. Louis Limestone	El. 663 – El. 635	196.5
Warsaw Limestone	< El. 635	162.1

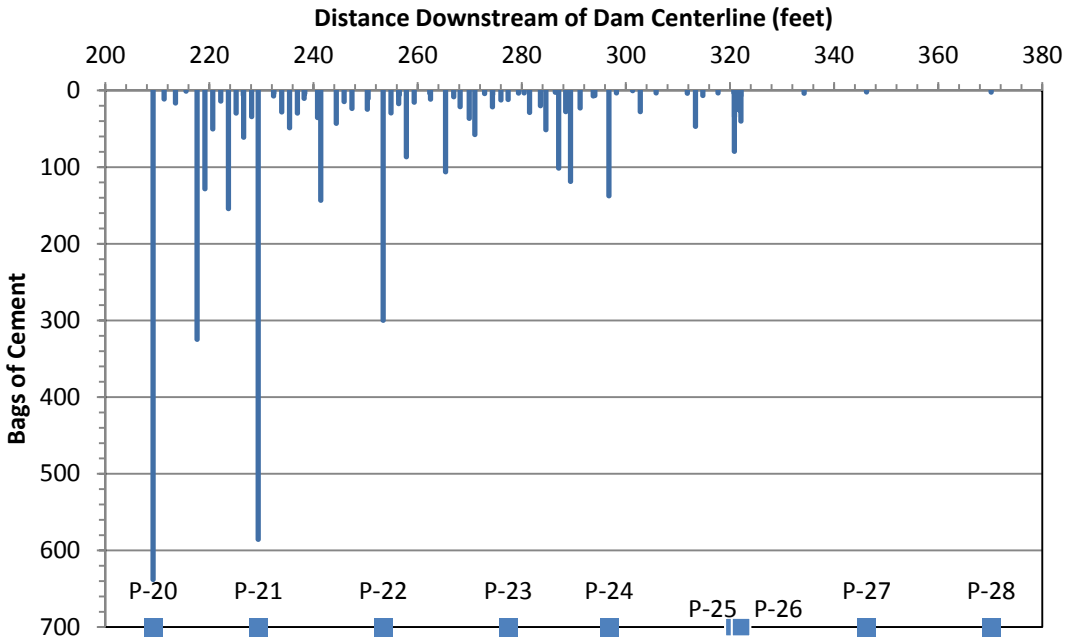


Fig. 5. Grout takes between P-20 and P-28

For the last week of grouting, chemical grouting was alternated every-other-day with cement grouting. While alternating chemical and cement grout, the cement grout holes generally had grout takes

© 2016 Deep Foundations Institute

on the order of 10 to 20 gallons (1 to 3 cubic feet), with a couple of exceptions where grout takes were about 150 to 225 gallons (20 to 30 cubic feet).

Closure was generally obtained along a second row that was established at a three foot offset to the landside of the original alignment. Some chemical grouting was performed on a third line of grout holes that was offset an additional three feet towards the landside to try to provide additional coverage and confinement to minimize the possibly of large concentrated flows into the excavation. The grouting procedures ultimately resulted in takes meeting the closure criteria, so the grouting program was completed.

SUMMARY AND CONCLUSIONS

On the landside of the excavation, shown on Fig. 6, the artesian flows and apparent voids were encountered in a horizon between about El. 645 to El. 635, roughly corresponding to the contact between the Warsaw Limestone (lower unit) and St. Louis Limestone (upper unit). Based on observations during drilling and grouting, it appears water traveled through serpentine pathways within this horizon, which drilling returns indicated were partially filled with clay, sands, silts, and rock fragments.

Communication was observed in adjoining grout holes during drilling, indicating that there is at a minimum indirect hydraulic connections in rock across distances of 50 feet or more. However, despite the large grout takes, return was seldom seen through neighboring grout holes, even closely spaced holes. Observed sequences of pressure build and release indicated that the grout was moving and/or compacting soil in the horizon. Upon excavation, it appeared that the grout increased the density of the soil with the limestone acting as a confining roof above the zone.

During excavation, angled drain holes were drilled from the excavation face down into the voided horizon to relieve water pressures prior to excavating through the zone. Many of the drain holes produced little to no flow, however some had outflows on the order of 20 gpm or higher which consistently flowed throughout excavation. Some of the drain hole locations are shown on Fig. 6 around El. 655 (green circles).

A six to nine inch seam of grout was observed in the excavation wall at the top of an approximately 18 to 24 inch layer of soil and below about 18 inches of unjointed limestone, as shown in Fig. 7. The soil primarily consisted of coarse-grained poorly cemented sand, as shown in Fig. 8, with medium to high plasticity clay present in pockets/seams of up to 12 inch thickness, and angular rock fragments.

The grouting program and drains installed during excavation have been effective in controlling the artesian flows, as little to no seepage was observed through the soil zone during excavation.

© 2016 Deep Foundations Institute

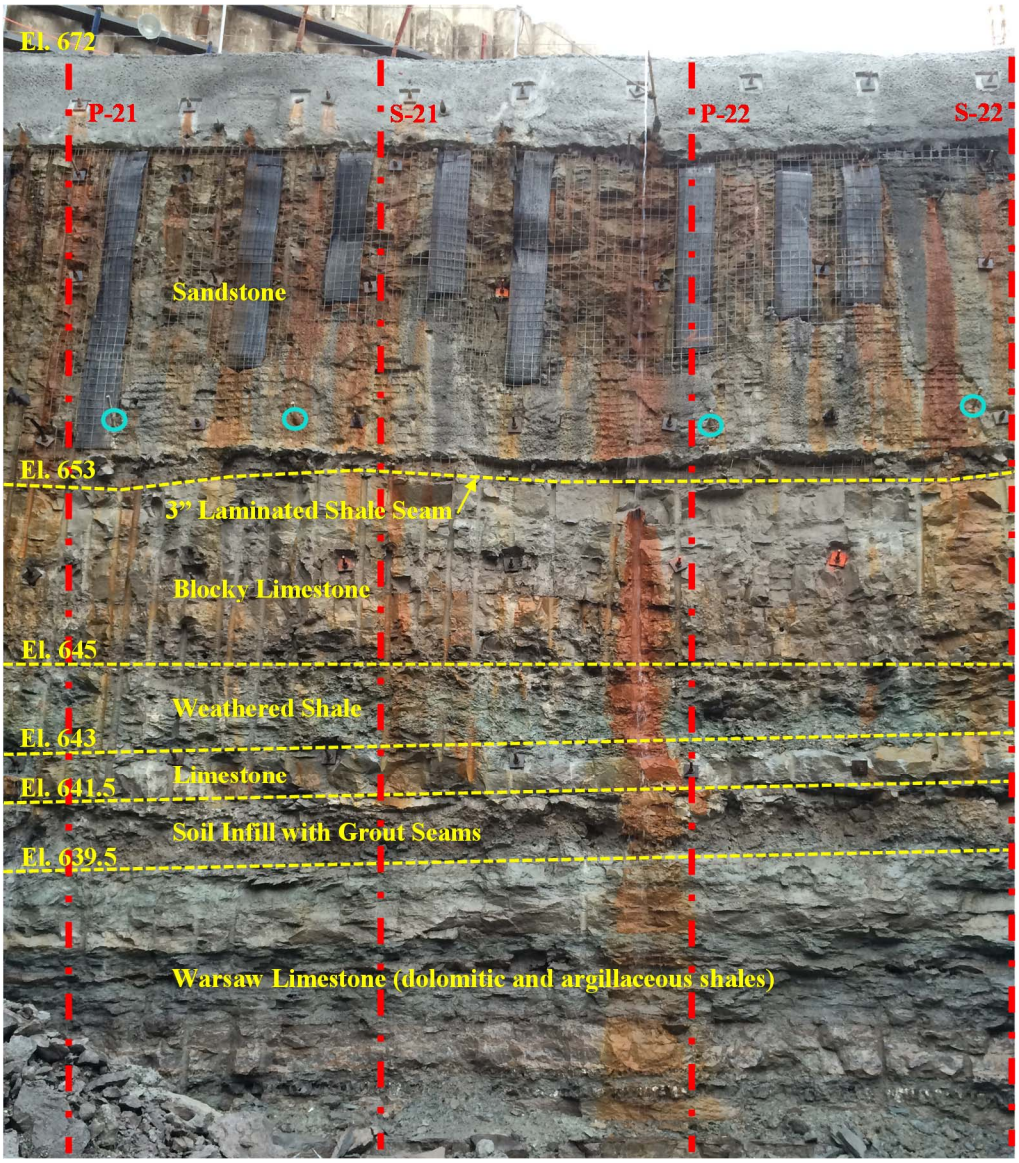


Fig. 6. Landside wall of excavation

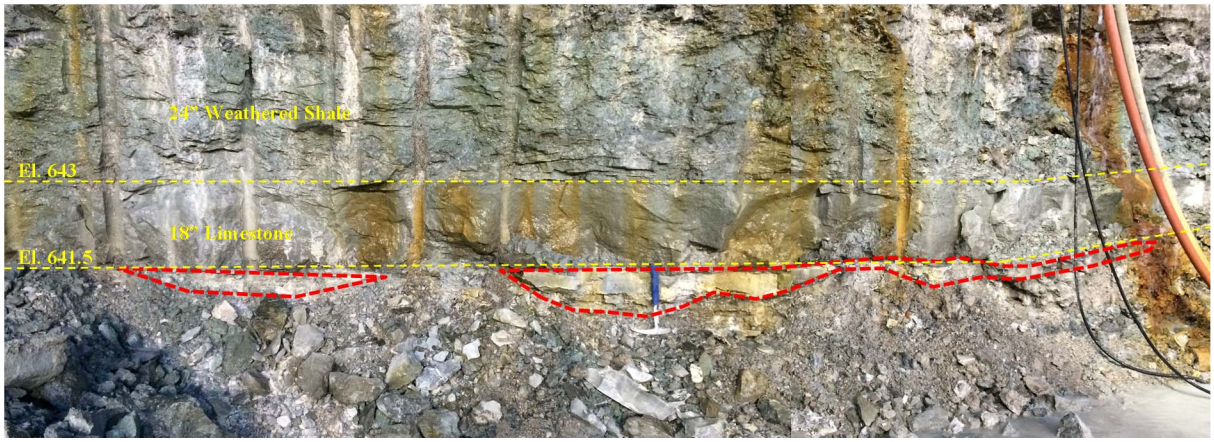


Fig. 7. Grout seams within soil horizon (outlined in red)

© 2016 Deep Foundations Institute

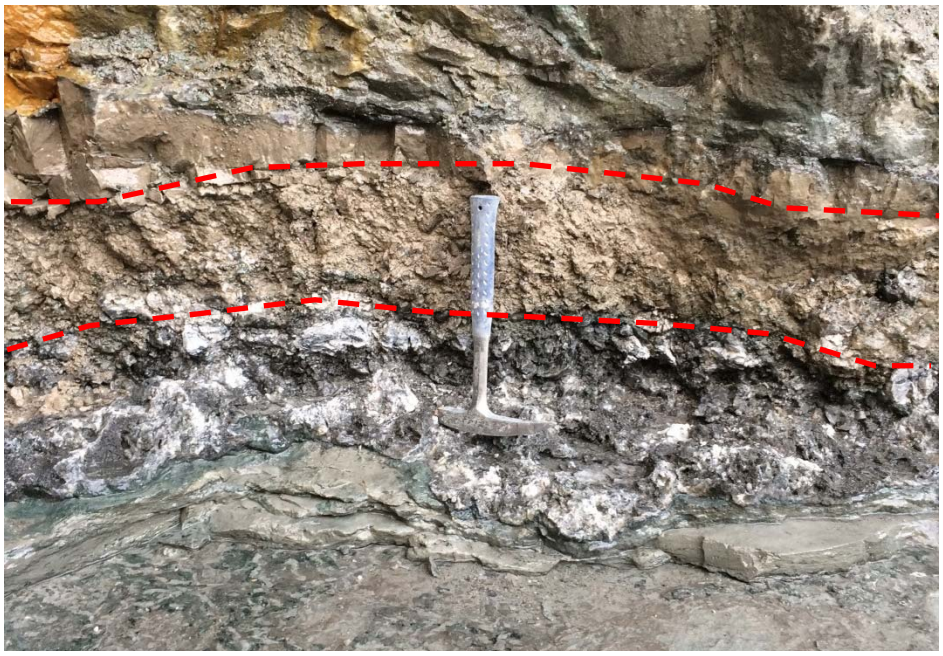


Fig. 8. Soil horizon along landside of rock excavation

ACKNOWLEDGMENTS

The Project is being developed by Western Minnesota Municipal Power Agency (WMMPA), a joint-action agency of municipalities that own their own electric systems, which is represented by Missouri River Energy Services (MRES).

MWH Americas, Inc. (MWH) (now part of Stantec) is the engineer-of-record for the project and is providing resident engineering services during construction, including field direction of the grouting program.

Ames Construction Co. (Ames) is the general construction Contractor and subcontracted Rembco Geotechnical Contractors, Inc. (Rembco) to perform the rock grouting program.

The authors wish to thank MRES for supporting the publication of this paper.

REFERENCES

Lucey, K.J. (1991). *Analysis of the Ground-Water Flow System, Geochemistry, and Underseepage in the Vicinity of the Red Rock Dam Near Pella, Iowa*. U.S. Geological Survey, Water-Resources Investigation Report 91-4092.

MWH (2014a). *Red Rock Hydroelectric Project – Geotechnical Data Report*. Prepared for Western Minnesota Municipal Power Agency.

MWH (2014b). *Red Rock Hydroelectric Project – Geotechnical Interpretative Report*. Prepared for Western Minnesota Municipal Power Agency.

MWH (2015). *Red Rock Hydroelectric Project – Grouting Report*. Prepared for Western Minnesota Municipal Power Agency.

© 2016 Deep Foundations Institute

JET GROUTING FOR GROUNDWATER CUTOFF AT TIE-IN BETWEEN EXISTING TUNNEL AND NEW CONSTRUCTION

Chu E. Ho, Arup, New York, New York, USA, chu.ho@arup.com
Aaron Evans, Nicholson Construction Company, Chicago, Illinois, USA,
aaron.evans@nicholsonconstruction.com
Tom Richards, Nicholson Construction Company, Canonsburg, Pennsylvania, USA,
tom.richards@nicholsonconstruction.com

ABSTRACT

This paper describes an innovative case history of using jet grouting to form a groundwater cutoff wall at the interface of a cut-and-cover construction and an existing tunnel. The design of the jet grouting layout was complicated by the presence of a left-in-place bulkhead of unknown dimensions abandoned by previous construction of the existing tunnel. Jet grouting was performed through predrilled holes within the narrow space between the existing bulkhead sheeting and tunnel structure. A systematic sequence of primary and secondary jet grout elements of varying diameters and panel lengths was executed to achieve an effective groundwater barrier. A key challenge in the jet grout design was the impact of potential deviations due to shadowing effects of the left-in-place bulkhead sheeting and pre-formed jet grout columns, as well as overbreak concrete in adjacent secant pile walls. The flexibility of the jet grouting methodology enabled a wide range of operational parameter combinations to be performed, allowing column diameters to be adjusted based on actual field installation measurements. Special precaution was exercised when grouting at critical locations to avoid over-pressurization and damage to the existing tunnel structure.

Keywords: tunnel, jet grouting, groundwater cutoff, tunnel, grout panel, grout column

INTRODUCTION

The 96th Street Station box for the new Second Avenue Subway Line in Manhattan, New York included a transition tunnel, which connected the station to an existing tunnel built in the early 1970s (Figure 1). The new station and transition tunnel were both constructed using the cut-and-cover technique with bottom-up sequence (Aksman et al. 2012, Grigson et al. 2016). The main station structure incorporated permanent diaphragm walls for most of its length, except for the southern segment where high rock levels were encountered and the station box was constructed within temporary secant pile walls socketed into the rock. The permanent diaphragm walls were also utilized for temporary excavation support (Ho and Hu 2014). At the northern end of the site, the new transition tunnel was constructed within temporary secant pile walls that extended from the station diaphragm walls to the existing tunnel at the tie-in location. A temporary traffic deck spanning 60ft across the secant pile and diaphragm walls along the full excavated length of the site allowed traffic to continue operating at street level while construction of the station and transition tunnel progressed below grade.

The structural framing of the existing tunnel consisted of a series of concrete encased steel frames spaced at 5 feet intervals along the length of the tunnel with unreinforced concrete arched roofs between them. Because of the large difference between the structural stiffness of the new transition tunnel and the existing tunnel, any displacement arising from the new structure may cause potential damage to the existing tunnel if not properly controlled. To minimize any impact on the existing tunnel, the last 10 feet of the invert slab of the transition tunnel was left uncast to allow differential displacements to stabilize before the tunnels were linked. Therefore the excavation adjacent to the existing tunnel was required to remain open for a

© 2016 Deep Foundations Institute

significant amount of time. The following sections present the development and implementation of the groundwater cutoff scheme using jet grouting to enable the tie-in construction to be completed in the dry.

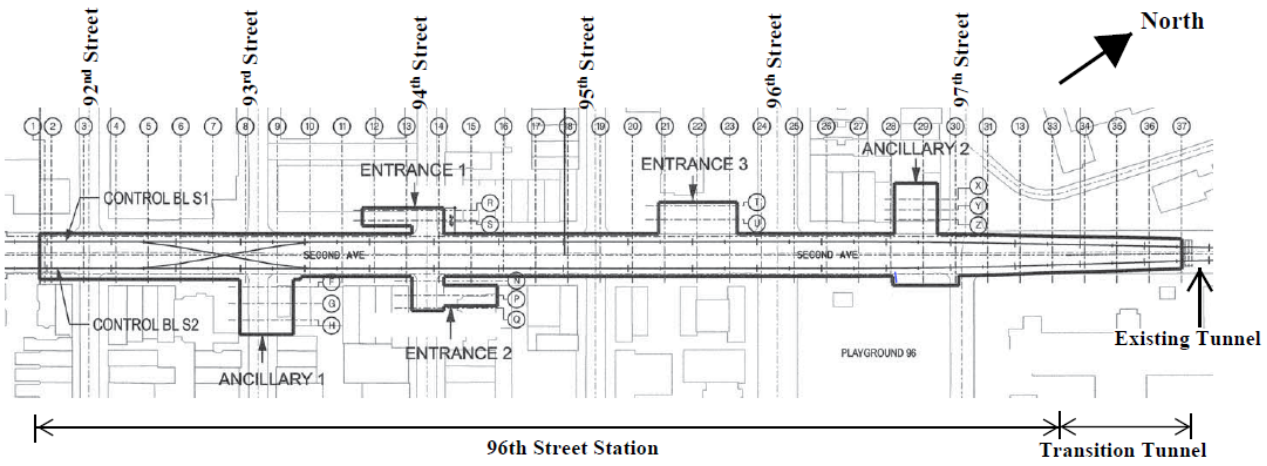


Fig. 1. Plan of 96th Street Station

GROUNDWATER CUTOFF SCHEME

The ground surface at the existing tunnel was generally flat at elevation EL+111ft. The subsoil profile consisted of fill overlying organics, silty sand and a deep layer of glacial silt. The existing tunnel invert slab was about 3 feet thick with the top of slab level at EL+75ft. The total excavation depth to the base of the existing tunnel invert was about 40ft feet with the final subgrade located within the glacial silt. The existing water table varied between 10 to 13 feet below ground surface. Full-scale pumping tests indicated that hydraulic conductivity of glacial silts in the horizontal and vertical directions were in the order of $5x10^{-4}$ cm/s and $5x10^{-5}$ cm/s respectively.

Historical information indicated that the face of the existing tunnel was covered by a concrete wall cast against a left-in-place bulkhead from the earlier construction. The bulkhead would have to be demolished to expose the existing tunnel for the tie-in connection to take place. The old bulkhead, which consisted of a steel sheet pile wall at the tunnel face, extended to an unknown depth below the tunnel and could not be relied upon for proper groundwater cutoff. Ground modification using geotechnical grouting was therefore selected to form a barrier wall below the existing tunnel to curtail seepage from beneath.

The design of the groundwater cutoff scheme was complicated by the uncertainty associated with the lack of details about the extent and configuration of the abandoned bulkhead. It was not possible to carry out field survey and test pit excavation in the roadway to ascertain the details of the bulkhead at the design stage due to the significant impact of partial road closures to traffic flow along Second Avenue. In addition, there were several utilities running over the tunnel roof, including an existing 48 inches diameter water main located close to the centerline of Second Avenue in the north-south direction that precluded early investigations.

Figure 2 depicts the preliminary concept adopted for the cutoff wall at the time of bid. The toe level of the temporary braced secant pile wall supporting the excavation was EL+28ft. The design called for a jet grout wall that extended below the excavation to an elevation matching the adjacent secant pile toes. Due to uncertainty regarding the position of the bulkhead sheeting, jet grouting was planned to be executed from the existing ground surface offset from the external face of the bulkhead. Additional jet grouting would be

© 2016 Deep Foundations Institute

performed to seal the gaps between the temporary secant pile walls and the sides of the existing tunnel, as well as between the temporary secant pile walls and the permanent station diaphragm walls on each side. The backfilled soil above the existing tunnel roof would be temporarily stabilized during the construction period. Prior to breaking the existing tunnel invert slab for the tie-in connection, a base seal beneath the tunnel would be required using permeation grouting. This would be implemented from within the excavation by drilling through the bulkhead after it has been exposed.

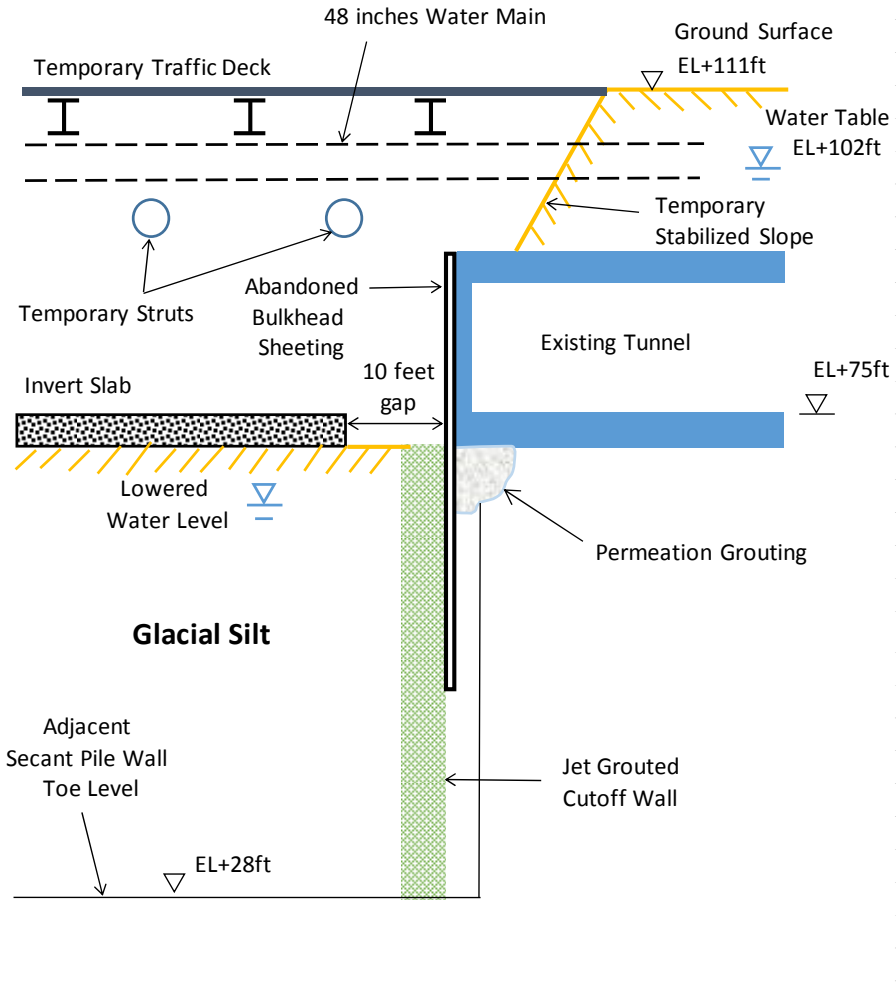


Fig. 2. Preliminary groundwater cutoff concept at tie-in to existing tunnel

During the course of construction work, further test pit investigations and survey work revealed the actual location and configuration of the left-in-place bulkhead sheeting. However, the termination depth of the sheeting remained unknown. To improve construction flow, an alternate grouting approach was subsequently adopted to avoid the two-phase operation involving jet grouting and permeation grouting. It was decided to perform the majority of the jet grouting from behind the bulkhead, in the narrow space between the abandoned sheeting and the tunnel structure, to form the cutoff wall in one continuous operation (Figure 3). The objective was to install fully formed jet grout columns below the toe of the bulkhead sheeting in order to extend the cutoff depth of the barrier wall. Jet grouting was then continued behind the sheeting above the toe level up to the bottom of the tunnel invert to provide the base seal. The jet grout columns were installed in a primary-secondary sequence, with smaller diameter primary columns being completed prior to the larger diameter secondary columns to mitigate potential shadowing effects. A

© 2016 Deep Foundations Institute

temporary sheet pile retaining wall was installed from ground surface up to the top of the tunnel roof to retain the soil above it. The retaining wall was extended across the roof to form a "T" junction with the existing bulkhead sheeting on both sides of the tunnel. However, due to obstruction of the water main at the tunnel roof, additional jet grout columns were required to fill the gap in the retaining wall at this location.

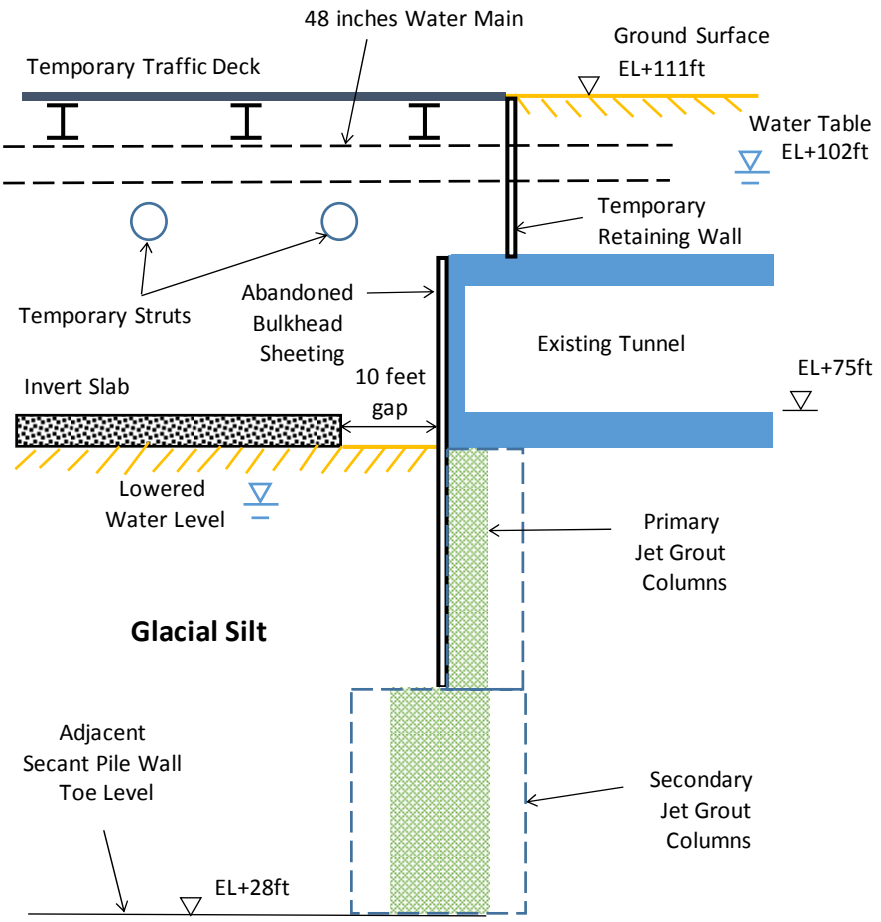


Fig. 3. Alternate grouting approach

PRODUCTION JET GROUTING

The layout of the jet grouting positions was constrained by the geometry of the abandoned sheeting uncovered at the bulkhead. A total of 20 jet grout columns and 2 jet grout panels were implemented during production grouting with target column diameters ranging from 4 to 8.5 feet. The jetting parameters were derived from an earlier trial program using a double fluid system carried out at a nearby empty plot of land in similar ground conditions (Ho, 2012). Grouting was performed using a 3.5ins (89mm) diameter monitor attached to a 4.5ins (114mm) diameter drill rod. A single 0.25ins (6.5mm) diameter nozzle was used in conjunction with a lift step of 1.57ins (4 cm). The trial confirmed that jet grout columns of 7 feet diameter and 5 feet diameter were reasonably achievable in the glacial silt deposits with and without compressed air respectively. The trial column diameters were determined using the Cyljet method based on electrical resistivity

© 2016 Deep Foundations Institute

measurements (Frappin and Morey, 2001). Table 1 lists the jetting parameters used in the trials (TC01, TC09 and TC10) and those selected for production grouting (Combination 1 to 7). Figure 4 shows the corresponding specific energies based on the selected jetting parameters for the target column diameters in comparison to the trial results.

Table 1. Jetting parameters for production grouting

Parameter Combination	Column Diameter (feet)	Grout Pressure (bar)	Grout Flowrate (l/min)	Air Pressure (bar)	Rotation Speed (rpm)	Lift Speed (sec/step)	Water-cement Ratio
1	4.0	405	296	0	9.2	13	1.43
2	5.0	405	296	0	5.2	23	1.43
3	5.0	365	258	10 to 13	15.0	8	1.58
4	7.0	365	258	10 to 13	6.3	19	1.53
5	8.0	395	302	10 to 13	5.7	21	1.53
6	8.5	395	302	10 to 13	4.6	26	1.53
7		520	258	10 to 13		1.5	1.34
TC01	6.7	315	290	10	4.4	15	0.68
TC09	8.0	325	305	10	5.8	22	0.89
TC10	4.9	390	320	0	5.5	22	1.2

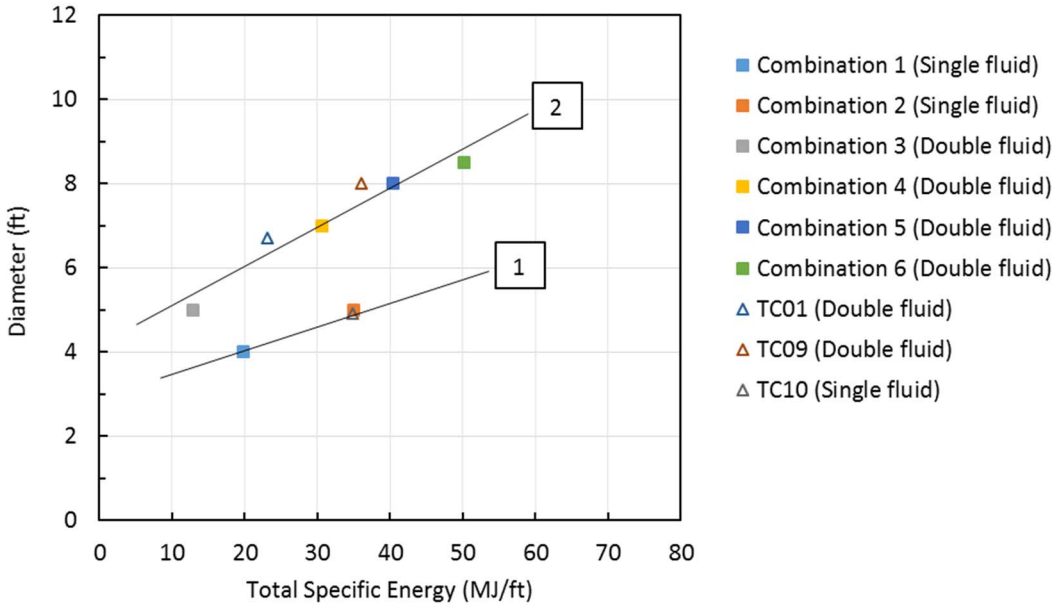


Fig. 4. Target column diameter and jetting energy

Production grouting was implemented in two stages following the sequence of partial closure of the street. Figure 5 shows the planned layout of the jet grout columns. The columns on the western half of the tunnel (U01 to U12 and V01) were completed first, followed by the eastern half (U13 to U19 and V02). In each case, the 4 feet diameter primary columns U03, U08, U14 and U16 were formed prior to the adjacent 7 to 8ft diameter secondary columns. The adoption of smaller

© 2016 Deep Foundations Institute

diameters for the primary columns was to avoid potential issues associated with deviation and shadowing during jetting of the larger adjacent columns. For columns U03 to U08 and U13 to U17, grouting was executed from approximately EL+28ft (matching the toe of the adjacent secant piles) to EL+75ft to form a continuous cutoff wall using overlapping columns beneath the tunnel invert. A minimum contact width of 2 feet 8 inches with the underside of the tunnel invert was achieved using this arrangement.

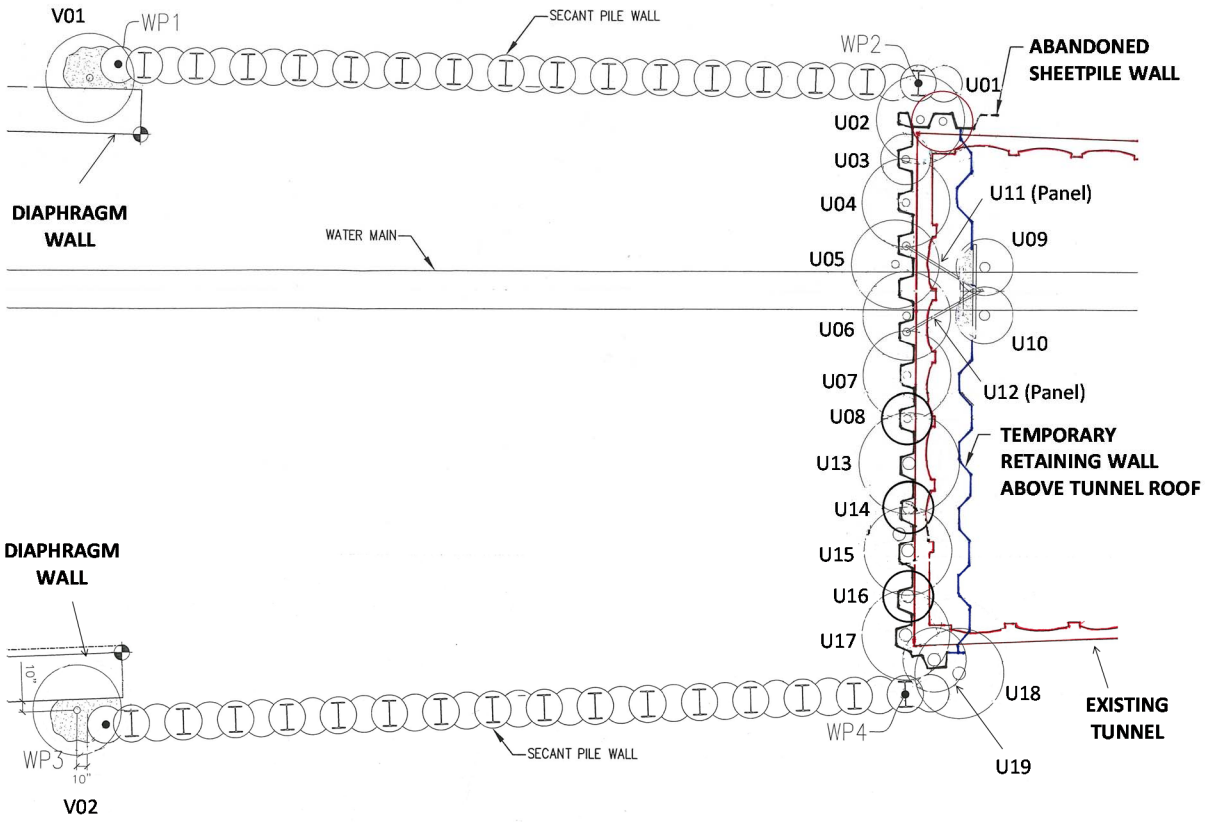


Fig. 5. Planned layout of production jet grout columns

Column U02 was installed after completion of U03, but prior to U01 with grouting taking place from EL+28ft to EL+105ft (i.e. 3ft above the normal high groundwater level at EL+102ft) to seal the sides of the tunnel. The presence of the abandoned sheet pile wall presented a problem with shadowing for grouting at U02, as the soil behind the sheeting could not be grouted directly. To provide proper closure, U01 was directed at the back of the sheeting to complete the seal with the tunnel. An 8ft diameter column was initially formed in the lower portion of U01, but jetting energy was reduced once grouting commenced above the tunnel invert and a smaller 5ft diameter column was formed. This was a compromise between ensuring good contact and avoiding potential damage to the tunnel side wall as a result of over-pressurization in the tight space between the sheeting and the tunnel structure above the invert level. Similarly, U19 was installed after completion of U17 and U18 to seal the tunnel side using a reduced 5 feet diameter column. Columns U09 and U10, were installed above the tunnel roof from positions located on either side of the water main to provide continuation of the temporary retaining wall beneath the water main

© 2016 Deep Foundations Institute

where the sheet piles could not be placed. Table 2 summarizes the grouting elevations and parameters adopted.

Table 2. Details of production jet grout columns

Jet Grout Element	Elevation (feet)		Parameter Combination	Column Diameter (feet)	Remarks
	From	To			
U01	+56.7	+75.0	5	8.0	Double fluid
	+75.0	+105.2	3	5.0	Double fluid
U02	+27.9	+105.0	4	7.0	Double fluid
U03	+54.2	+75.6	1	4.0	Single fluid
U04	+27.1	+75.7	4	7.0	Double fluid
U05	+27.8	+66.5	5	8.0	Double fluid
	+66.5	+75.5	4	7.0	Double fluid
U06	+27.1	+75.6	4	7.0	Double fluid
U07	+27.2	+65.2	5	8.0	Double fluid
	+65.2	+75.8	4	7.0	Double fluid
U08	+27.9	+75.7	1	4.0	Single fluid
U09	+93.8	+102.6	1	4.0	Single fluid
U10	+94.0	+102.5	2	5.0	Single fluid
U11	+33.4	+75.8	7		Panel
U12	+55.0	+77.1	7		Panel
U13	+28.0	+75.0	4	7.0	Double fluid
U14	+28.0	+75.0	1	4.0	Single fluid
U15	+28.0	+75.0	4	7.0	Double fluid
U16	+28.0	+75.0	1	4.0	Single fluid
U17	+28.0	+75.0	4	7.0	Double fluid
U18	+28.0	+75.0	4	7.0	Double fluid
U19	+28.0	+75.0	3	5.0	Double fluid
U20	+28.0	+75.0	3	5.0	Double fluid
V01	+27.8	+47.9	6	8.5	Double fluid
	+47.9	+105.1	4	7.0	Double fluid
V02	+28.0	+105.0	4	7.0	Double fluid

Figure 6 shows the as-built layout of the jet grout columns at the toe levels. The measured drilling deviations ranged from 1.8 to 3.5% over the depth of installation. Several situations arose during grouting where the deviation was significant due to difficulty in drilling within the tight space between the bulkhead sheeting and the tunnel face or where jetting alignment was affected by shadowing of a pre-installed jet grout column. U20 was a compensation column which was introduced after drilling measurements revealed U14 had deviated significantly, creating a lack of interlock between U14 and U15. Grout panels U11 and U12 provided additional groundwater seepage containment beneath the water main location. During predrilling of V01, hardened concrete was encountered between depths of 2.5 and 27 feet, resulting in a large deviation of 3.13% in the direction away from the secant pile wall. As a result, the column diameter for V01 was increased from the nominal 7 feet to 8.5 feet below EL+47.9ft to ensure good contact between the secant pile wall and diaphragm wall at the deeper levels.

© 2016 Deep Foundations Institute

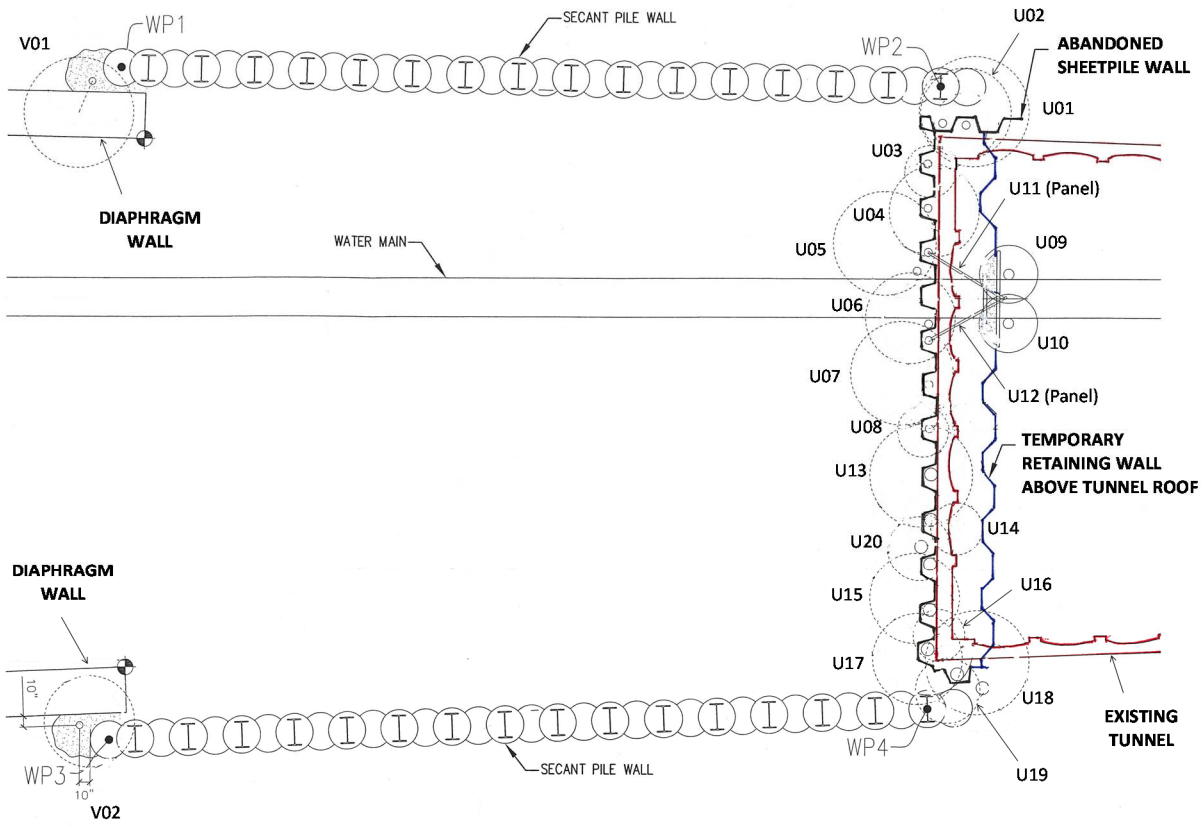


Fig. 6. As-built positions of jet grout columns at toe level

The jet grouting works were completed prior to the excavation of the new station and transition tunnel. Upon exposure of the ground in front of the existing tunnel and subsequent breakout of the bulkhead, it was observed that seepage flows were reasonably controlled and the excavation was maintained dry by pumping from ejector wells. Figure 7 shows the water-tight seal at the contact between the new construction and the existing tunnel. The 10ft gap between the existing tunnel invert and the new invert slab of the transition tunnel was temporarily bridged over with steel plates to allow access between the two tunnels during construction.

Observation well OSP 97-5, located at the final subgrade next to the pour strip, indicated that the lowered water level within the excavation subgrade fluctuated between EL+58.2ft and EL+67.4ft, which was well below the existing tunnel invert at about EL+71ft. Vibrating wire piezometer VP 97-6, installed in the retained ground adjacent to the temporary secant pile wall, indicated piezometric heads varying between EL+99.0ft and EL+101.6ft, suggesting that the drawdown in the external water table was less than 3 feet. The gap in the new invert slab of the transition tunnel was poured once backfilling above the new tunnel roof has been completed for at least one month and the movement rate was less than 0.1 inch per week. Figure 8 shows the completed link up between the new construction and the existing tunnel.

© 2016 Deep Foundations Institute



Fig. 7. Watertight seal at tie-in with existing tunnel

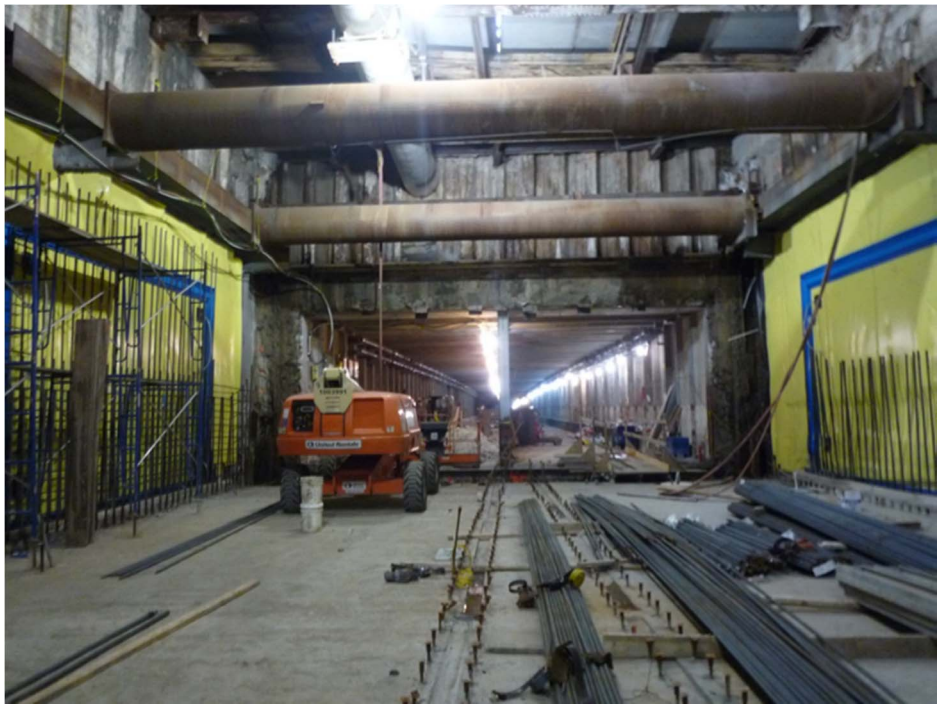


Fig. 8. Completed link up between existing tunnel and new construction

© 2016 Deep Foundations Institute

CONCLUSIONS

An innovative case history of groundwater cutoff using jet grouting is presented in this paper. The design of the jet grouting layout was complicated by the presence of abandoned bulkhead sheeting left behind from previous construction of the existing tunnel. The majority of jet grout elements were installed through predrilled holes within the narrow space between the bulkhead sheeting and the tunnel structure in one continuous operation. Various configurations, including columns and panels, were used to form the groundwater barrier. A combination of column diameters was executed, ranging from 4 to 8.5 feet, to achieve adequate overlap between adjacent jet grout columns. The larger columns (7 to 8.5ft diameter) were formed using double fluid parameters with compressed air, while the smaller columns (4 and 5ft diameter) were formed using single fluid parameters without compressed air. Jetting parameters were based on field trials carried out at an adjacent plot of land in similar ground conditions. Lower jetting energy was used when performing grouting at critical locations to avoid over-pressurization and damage to the existing tunnel side walls.

A key challenge in the jet grout design was the impact of potential deviations due to shadowing effects caused by existing bulkhead sheeting and pre-formed jet grout columns, as well as obstructions in the ground due to overbreak concrete in adjacent secant pile wall. Systematic monitoring of drilling deviations enabled gaps in the jet grouted wall to be identified and successfully treated. Compensation of identified gaps was achieved by varying the column sizes during production grouting or installation of additional jet grout columns. Flexibility in performing jet grouting using a combination of parameters was critical to the success of the project.

REFERENCES

Aksman, B., Grigson, R., and Giffen, R., 2012. Second Avenue Subway Project, New York: design of new underground stations. Proceedings of the Structures Congress 2012, Chicago, IL, ASCE.

Frappin, P. and Morey, J. 2001. Jet grout column diameter measurement using the electric cylinder method. Travaux No.775, France.

Grigson, R., Ho, C. and Lemus, P., 2016. Second Avenue Subway Project: deep excavation support of a cut-and-cover station. Proceedings of the Geotechnical and Structural Engineering Congress 2016, Phoenix, AZ, ASCE, 402-415.

Ho, C. E., 2012. Jet grouting field trial in Manhattan soil deposits, New York. Grouting and Deep Mixing 2012, Geotechnical Special Publication 228, Vol.2, ASCE, 2122-2131.

Ho, C. E., and Hu, S., 2014. Design optimization of underground subway station diaphragm walls using numerical modeling, Geotechnical Special Publication No.234, ASCE, 3122-3132.

© 2016 Deep Foundations Institute

METRO ROME LINE "C" - ARTIFICIAL GROUND FREEZING APPLICATION FOR THE UNDERPASS OF THE EXISTING METRO STATION SAN GIOVANNI - LINE "A"

Alessandro Bertero, Trevi SpA, Cesena,Italy, abertero@trevispa.com
Ferruccio Cribari, Trevi SpA, Cesena,Italy, fcribari@trevispa.com
Roberto Sorge, Astaldi SpA, Roma, Italy, r.sorge@astaldi.com
Eliano Romani, Metro C ScpA, Roma, Italy, e.romani@metrocspa.com
Giulia M.B. Viggiani, DICII, Università di Roma Tor Vergata,Roma,Italy, viggiani@uniroma2.it
Francesca Casini, DICII, Università di Roma Tor Vergata,Roma,Italy, francesca.casini@uniroma2.it

ABSTRACT

The use of Artificial Ground Freezing (AGF) to form earth support and seepage control barrier systems has had applications worldwide. These cover a variety of construction problems, including the formation of frozen earth walls to support deep excavations, structural underpinning for foundation improvement, and temporary control of ground water in construction processes. An application of AGF during construction of Line C of Rome underground is described in detail.
The new line "C" of San Giovanni metro station was constructed adjacent to the existing line "A" of San Giovanni station; the new metro line "C" is deeper than line "A" and runs below the existing station. The underpass project envisages an Artificial Ground Freezing in order to create a structural and waterproof temporary cut-off suitable to the two new tunnels excavation, while keeping trains running in the safest possible condition. The excavation of tunnels under the water table, with a hydraulic head of over 10 m in not homogeneous soils having different mechanical properties and permeability and below existing sensitive structures, involves special ground freezing sequences and methods, as well as peculiar drilling systems and operations, such as TDDT (Trevi Directional Drilling Technology).

Keywords: Artificial Ground Freezing, Directional Drilling, Metro Stations, Tunnels excavation

ARTIFICAL GROUND FREEZING INTRODUCTION

Natural freezing occurs seasonally in many areas of the globe and can affect negatively the engineering performance of geotechnical structures; on the other hand, man-made freezing is a controllable process that can be used by civil engineers to stabilise temporarily the ground in order to support and/or exclude groundwater from an excavation until the final retaining and lining structures are constructed. Freezing and thawing of the pore fluid within soils involve complex thermal, hydraulic and mechanical processes that have significant mutual interactions. For example, phase changes of the pore fluid caused by temperature variations can modify the pore water pressure in the soil, which, in turn, can induce mechanical deformation. At the same time, any changes in the hydraulic and mechanical conditions affect the thermal processes by advection and changes of ice and water contents.
An application of AGF which will be presented in the paper, refers to the construction works of Line C of Rome underground. In this case AGF was carried out to provide temporary support during excavation of the two Line C running tunnels directly underneath the existing S. Giovanni station of Line A, in the safest possible condition. In order to keep the trains of Line A operational during excavation, a temporary structure was installed and a real-time monitoring system was conceived in order to control the temperatures of the soil during freezing and the response of the structures of the existing station.

© 2016 Deep Foundations Institute

AGF IN UNDERGROUND CONSTRUCTION

The last decade has seen an increased use of artificial ground freezing in underground construction. Due to its comparatively high costs, the technique is typically applied for relatively short structures, such as shafts, cross passageways, and platform tunnels; many recent examples are reported in the literature of structures of this type that have been successfully completed using AGF under very diverse geotechnical conditions all over the world (see e.g., Pimentel et al. 2011, Haß & Schäfers, 2013).

Underground frozen soil structures are created by installing freeze tubes into the ground and then circulating a refrigerating medium until the temperature of the ground around the tubes is below the freezing point of water. The refrigerating medium, flowing down an inner pipe and returning within the space between the two pipes, can be nitrogen, liquid at about -196°C, and then vented as gas to the atmosphere at temperatures around -100°C to -60°C ("direct" or "open circuit" method), or brine (calcium chloride) entered at about -35°C, and then re-circulated through a refrigeration plant ("indirect" or "closed circuit" method). Choosing either one of the two methods depends on economic and logistic factors, as well as on the time required to freeze the soil. Heat extraction from the soil results in transformation of free water into ice, and additional cooling of the frozen soil. Initially, a frozen soil cylinder will form around each freeze pipe. With continued heat extraction, the frozen cylinders increase in diameter until they merge and form a frozen wall, serving both mechanical (support) and hydraulic (water tightness) purposes.

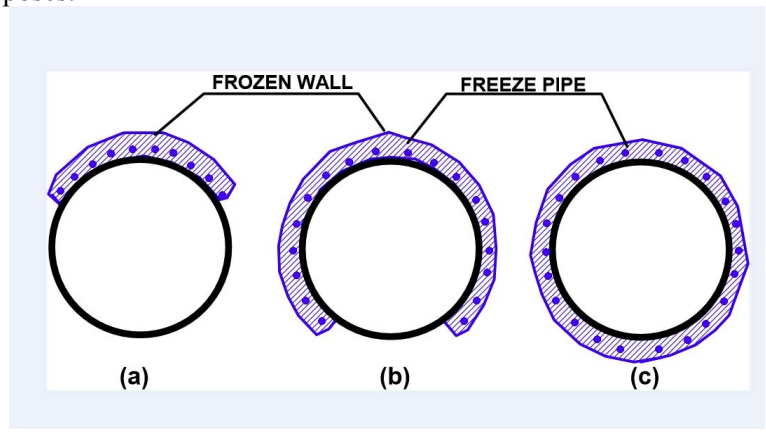


Figure 1: Tunnel supported by frozen soil: (a) roof, (b) roof and sides, (c) closed ring

In tunnelling applications a relatively thin frozen wall is formed around the outer contour of the tunnel, to provide support to its roof portion, to its roof and sides, or forming of a full closed ring around the excavation (Figure 1). The freeze tubes are typically placed sub-horizontally starting from a shaft or an open excavation. The relative freeze tube spacing controls the time needed to complete satisfactory freezing, which is exponentially proportional to the relative pipe spacing: an empirical criterion derived from data by Shuster (1982) is that the spacing of refrigeration pipes should not be larger than about 13 times their diameter. To avoid gaps in the frozen wall, the drilling tube direction must be controlled during installation and the alignment of all freeze pipes verified after their installation.

Soil type, density, and water contents are needed to estimate soil thermal properties. In situ tests can provide information on the order of magnitude and variability of the permeability.

In the absence of flowing water, the extent of the frozen soil zone depends primarily on the frozen thermal conductivity. This parameter may vary by a factor of 4 or 5, with lower values for silts and clays and higher values for rock. Groundwater flowing at velocities larger than 1 to 2 m/day may cause problems to the formation of a continuous frozen wall. If the groundwater seepage is less than 3.0 m/day, to ensure the formation of a continuous wall it may be sufficient to adopt reduced freeze pipe spacing; if the groundwater flow exceeds 3 m/day, either the permeability of the formation or the groundwater gradient must be reduced. This can be accomplished by grouting prior to or during installation of the freeze pipes

© 2016 Deep Foundations Institute

or intercepting the water flow with wells. In addition to seepage concerns, the location of the water table is significant because of the importance of water content and degree of saturation in determining the strength of frozen soil. Also, the salinity of groundwater should be determined as, when the pore water contains a significant quantity of solutes the freezing point is depressed.
Monitoring requirements for ground freezing are normally limited to freeze hole deviation, and temperatures within the coolant distribution system and in the ground, thus providing information of frozen front advance, and ground displacements at surface and at depth, during freezing and thawing.

LINE C OF ROMA UNDERGROUND

Project layout

At present the existing underground network in Rome consists of only two lines, Line A and Line B, intersecting at Termini Central Railway Station. Line B was built during the 1930s by cut-and-cover whereas the bored tunnels of Line A date back only a few decades, as the line was constructed in the 1970s. The new Line C of Rome underground will run northwest to southeast across the city centre, for a total length of more than 25 km and 30 stations (Figure 2). Metro C SpA is the General Contractor, consisting of Astaldi (representative), Vianini Lavori, Ansaldo STS and CMB, which deals with the construction of Line C in all its phases (design, archaeological surveys, building the tunnels, the stations and trains, and start-up). The project is divided into seven contracts; the stretch between Monte Compatri-Pantano, at the easternmost point of the line, and Giardinetti (contract T7) is on surface, while the remaining part of the line is excavated underground using two EPB shields of a diameter of 6.7 m. The stretch between Giardinetti and Parco di Centocelle was completed in November 2014 while the following six stations, from Mirti to Lodi, were opened to the public in June 2015, thus completing contracts T6 to T4. The part of the line between S. Giovanni and Fori Imperiali (contract T3) is currently under construction, at different stages of advancement: S. Giovanni station should be ready this year, while it is expected that the stretch between San Giovanni and Fori Imperiali will take another five years to be finished.

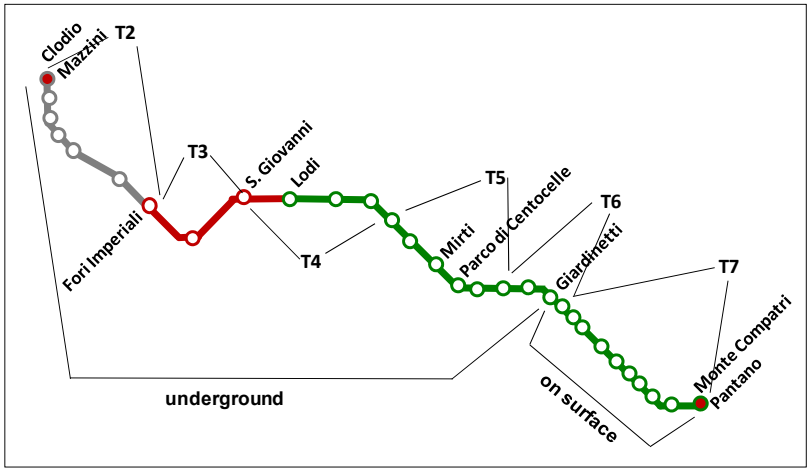


Figure 2: Line C of Rome underground

Construction of contracts T3 and T2 is more problematic than the previous eastern stretches of the line as the running tunnels and the stations will have to be built in the historical centre of the city, with significant problems connected to buried archaeological remnants and to the need of minimising the effects of construction activities on the historical and monumental heritage.

© 2016 Deep Foundations Institute

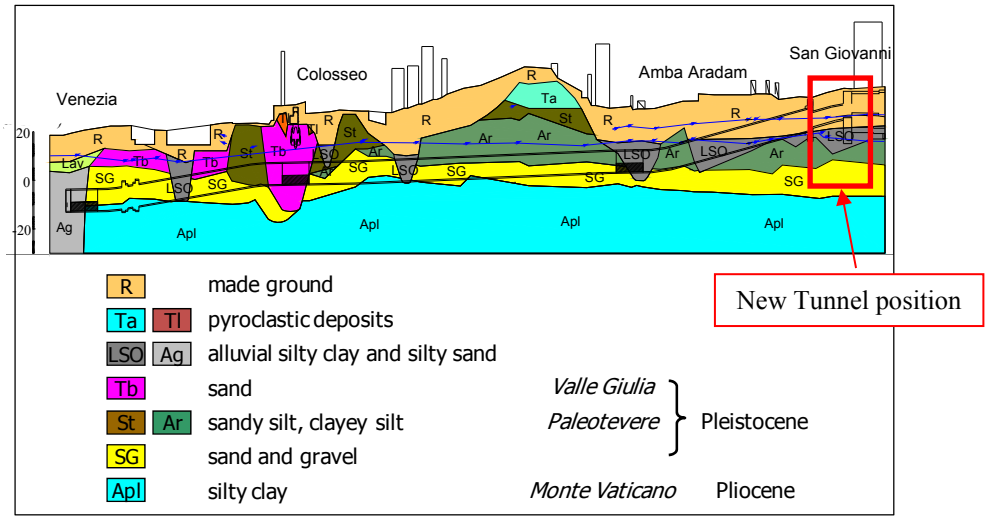


Figure 3: Soil profile and groundwater conditions along contract T3 of Line C of Rome underground

The soil profile and groundwater conditions along the central part of the route of Line C (contract T3) are illustrated in Figure 3. Starting from the bottom, the soil profile consists of a deep layer of stiff overconsolidated silty clay from the Pliocene (Apl) followed by gravel (SG), clayey silts and sandy silts (St, Ar), and sandy and silty soils (Tb), all from the Pleistocene. These are overlain by pyroclastic deposits (Tl and Ta) in the area of Colosseo. Frequent and relatively deep ancient ditches filled with alluvial silty–clayey and sandy deposits (LSO) cut through the Pleistocene deposits. A layer of made ground (R) of varying thickness covers the whole area. The ground water table is almost everywhere nearly hydrostatic with a hydraulic head at about 10 m a.s.l. However, in the area of S. Giovanni/Amba Aradam, the hydraulic head in the made ground is higher than in the sands and gravel (SG) and there is a downward steady sate seepage through the finer grained soils (LSO/Ar).

SAN GIOVAANI STATION - LINE C OF ROMA UNDERGROUND

Design issues and use of AGF

In the original design, to avoid direct interferences with buried archaeological remnants and limit surface settlements, the running tunnels were placed at depths between 20 and 35 m below ground level, with the exception of the stretch between Largo dell'Amba-Aradam and S. Giovanni, where the running tunnels were shallower to allow interchange with the existing Line A. However, some important archaeological remnants, brought to light during the construction of the new San Giovanni station, made it necessary to change the original design and lower the tunnels also in this area. The new S. Giovanni station reaches a depth of 30 m below ground level and is supported by 55 m deep diaphragm walls reaching the base layer of stiff overconsolidated clay (Figure 4). To underpass the existing Line A, two tunnels were excavated by conventional mining directly underneath the existing S. Giovanni station using a variety of consolidation techniques including chemical and cement injections and AGF. The project presented several technical challenges connected to the restrictions imposed by the existing structures of Line A, which included two alignments of large diameter piles at a distance less than the track section width, that had to be re-profiled, and the need to maintain the trains of Line A fully operational during the works. Further element of complexity derived from the local stratigraphy and hydraulic conditions.

© 2016 Deep Foundations Institute

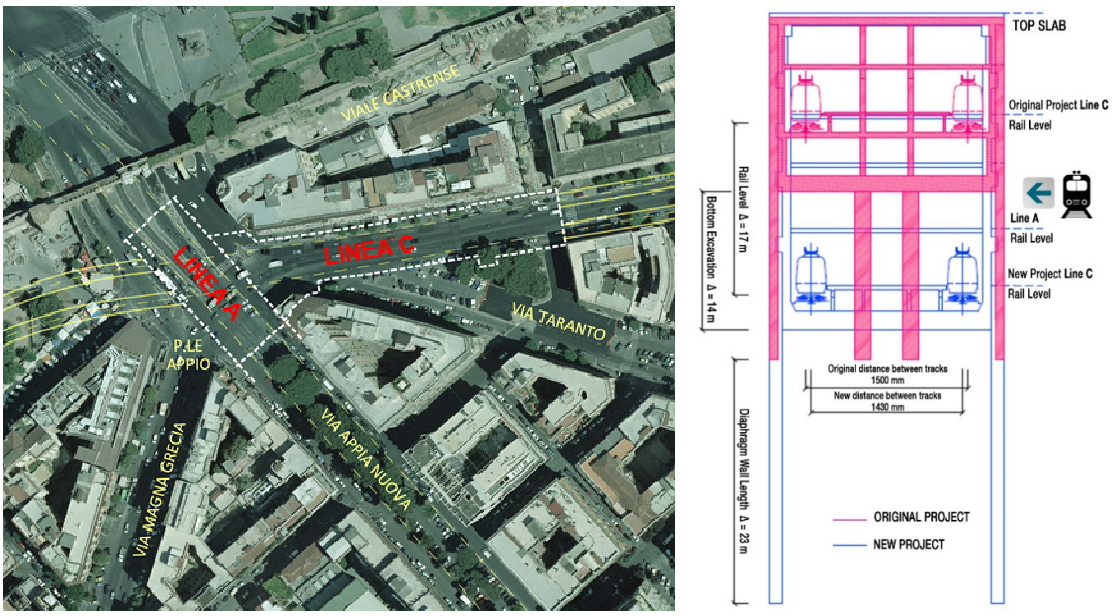
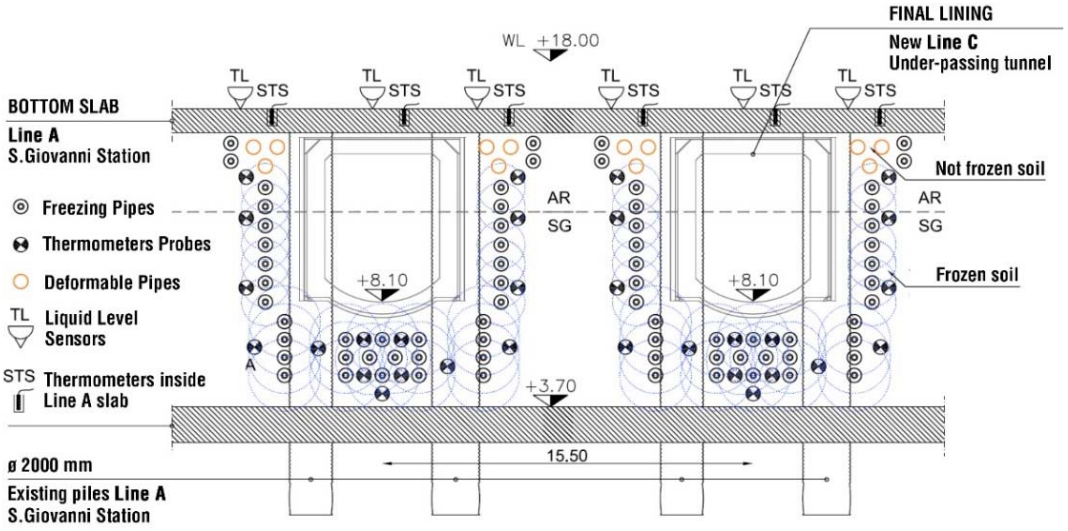


Figure 4: San Giovanni Station – intersection of Line C and existing Line A – original and new design

The soils directly affected by AGF and subsequent tunnel excavation are the clayey silts and sandy silts (Ar) of the Pleistocene and the sands and gravels (SG), resting on the base stiff overconsolidated Pliocene clay (Apl).

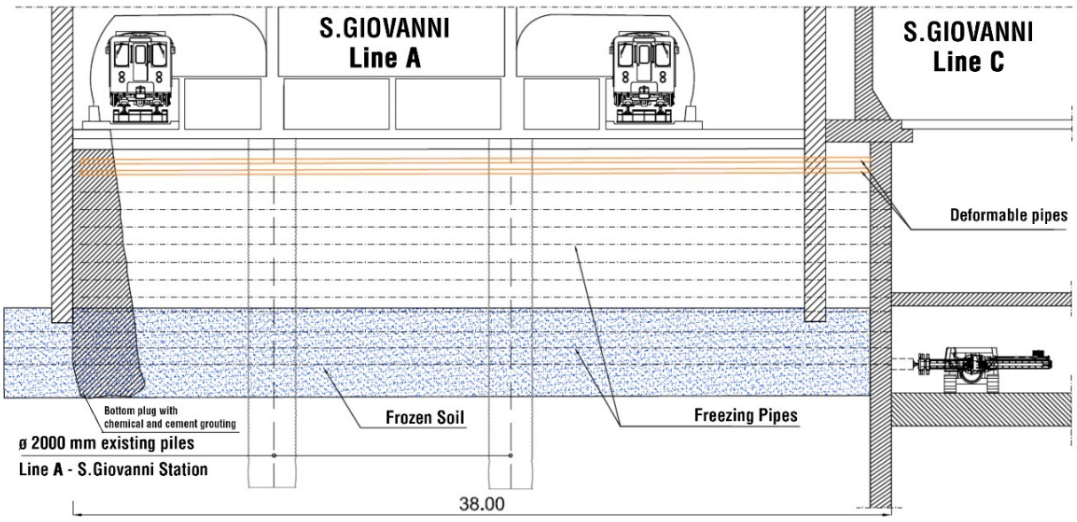
Chemical and cement injections were carried out first to consolidate the excavation front and homogenise the soil before freezing. Freeze tubes and secondary holes to measure the temperature in the ground, with an average length of about 40 m, were installed sub-horizontally from the shaft of the new S. Giovanni station, immediately below the base slab of the existing station and around the perimeter of the tunnels, in order to create a frozen wall providing structural support and ensuring water tightness during excavation (Figure 5).



(a)

Figure 5: Layout of freezing and secondary holes (a) cross and (b) longitudinal section

© 2016 Deep Foundations Institute



(b)

Figure 5: Layout of freezing and secondary holes (a) cross and (b) longitudinal section

Moreover, due to the presence of lenses of permeable material coming into contact with Line A station, some additional freezing tubes were installed on the sides of the frozen body. If necessary, these can be used to carry out further non-structural, waterproofing freezing ("soft ice"). The pipes were arranged in such a way as not to freeze a thickness of soil of about 1 m immediately below the existing slab, so that this layer could undergo deformations and absorb any movements or heave due to the creation of the ice wall. Moreover, to reduce further deformations caused by the formation of the ice wall, deformable PVC pipes were installed in the area between the slab and the frozen body.

Freezing was activated with nitrogen, to reduce the time required to form the ice wall, particularly in the area of the piles, where the spacing between the freezing tubes reached values larger than 3.5 m, and maintained with brine, to control the growth of the ice wall with time, thus reducing the deformations induced on the existing slab of Line A, and save on costs.

Once the ice wall was formed, the diaphragm walls of the new and existing S. Giovanni stations were cut by diamond wire saw (Figure 6). The tunnels were excavated with the temporary support of sprayed concrete and steel ribs at the roof and sides and, finally, the invert was excavated, a waterproof membrane installed and the permanent lining cast (Figure 7). After completing the permanent lining of the tunnels, re-profiling of the foundation piles, interfering with the track sections of Line C, was carried out (Figure 8).

In order to allow the trains of Line A to operate regularly to and from the existing station, a temporary structure, consisting of steel bridges founded on micro-piles, was installed at platform level to support the Line A rails.

© 2016 Deep Foundations Institute


(a)

(b)

Figure 6: (a) cutting of the diaphragm wall and block removal, and (b) tunnel excavation


(a)

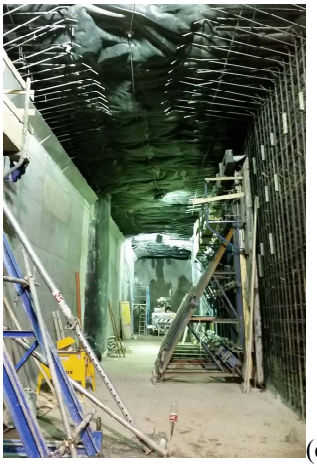
(b)

(c)

Figure 7: (a) temporary lining, (b) excavation of invert and installation of water proofing, and (c) final lining

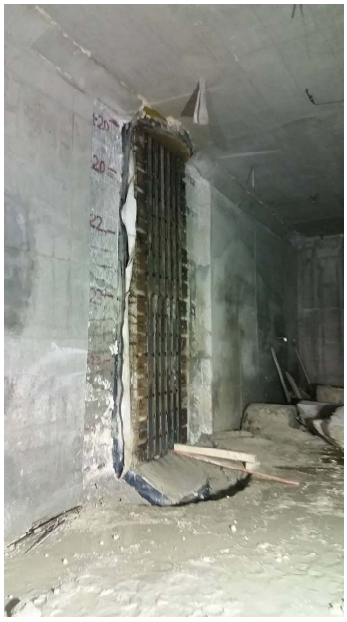

(a)

(b)

Figure 8: Re-profiling of piles: (a) initial phase, and (b) completed re-profiling

© 2016 Deep Foundations Institute

Observed phenomena

Due to the complexity and sensitivity of the works, it was necessary to use different monitoring systems, aimed at guaranteeing the safety of construction, checking the response of existing structures and controlling the freezing process. An integrated monitoring system was installed on the structures of the existing S. Giovanni station (diaphragm walls, pillars, slabs and rail tracks) and on the new tunnels of Line C (Bertero et al., 2016).
The monitoring system included: (i) liquid level sensors along three alignments of the bottom slab of Line A, (ii) a set of thermometers inserted inside the bottom slab, (iii) mini prisms and topographic benchmarks on the bearing structures of the existing S. Giovanni station, (iv) mini prisms and topographic benchmarks checking on the bearing structures of the new tunnel, and (v) electric level on the temporary support steel bridges. Figure 9 shows a plan of the instrumentation installed at the level of the base slab.

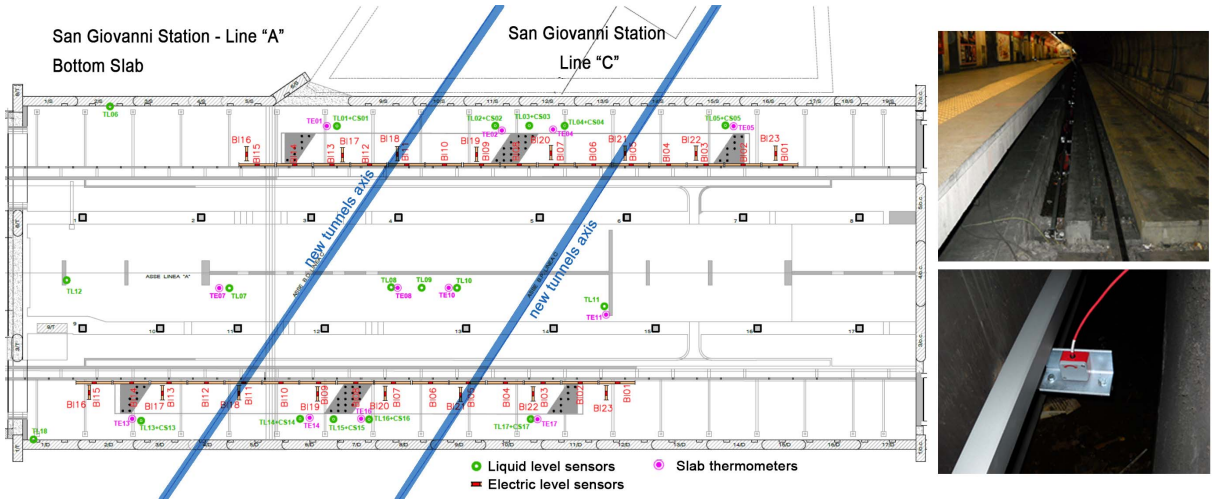


Figure 9: S. Giovanni station, plan of the instrumentation installed on the base slab

In addition to this instrumentation, thermometers were installed in secondary tubes to monitor the temperature of the ground around the freezing tubes. Particular care was taken to reach the areas between the piles, where the spacing between the freezing tubes was larger than 3.5 m, due to the presence of the piles. If needed, these holes could be changed into additional freeze holes to repair any lack of continuity in the formation of the ice wall. The work, however, was completed without the need to activate freezing at these positions.
Figure 10 shows the heave recorded along two alignments of liquid level gauges on the floor slab of the existing S. Giovanni station at different stages of the freezing process. The initial vertical displacements of some of the points are not zero, as some heave was caused by the initial the preliminary chemical and cement injections. The maximum heave recorded close to the diaphragm wall on the side of the new S. Giovanni Station (alignment 1) is of the order of 20 mm, while in the central alignment (alignment 2) the maximum heave is as high as 50 mm. Also, the recorded heave has clear correspondence to the freezing activities, with the liquid level gauges corresponding to the left tunnel (TL1 and 2, and TL7 and 8), which was treated first, responding before those closer to the right tunnel (TL4 and 5, and TL9 and 10).

© 2016 Deep Foundations Institute

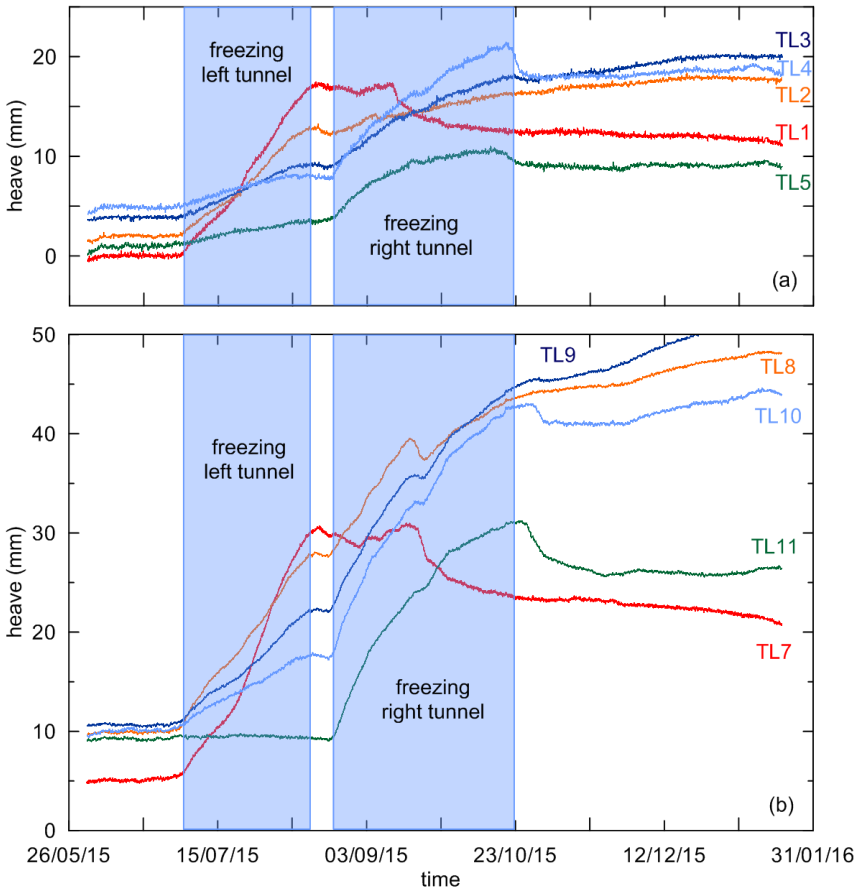


Figure 10: Heave of base slab: (a) alignment 1, and (b) alignment 2

This is also very clear in Figure 11, showing the contours of base slab heave at the: (i) end of chemical and cement injections, (ii) end of nitrogen activation of the left tunnel, and (iii) end of nitrogen activation of the right tunnel while the left tunnel is being excavated under brine maintenance.

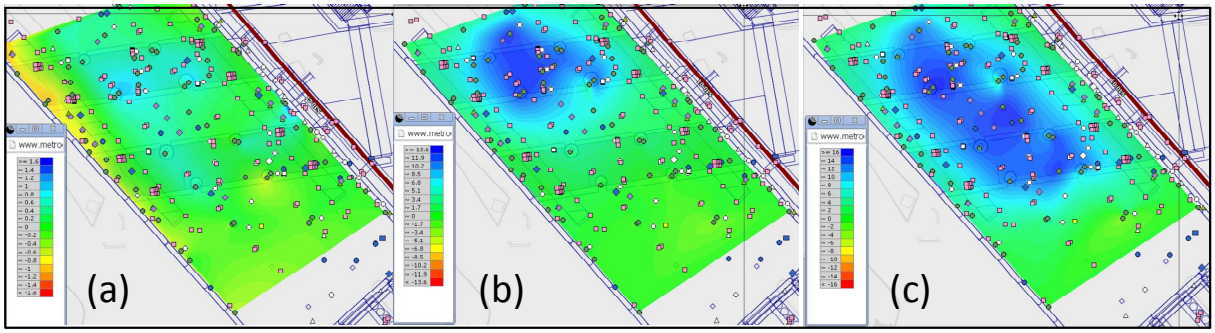


Figure 11: Heave of base slab: (a) end of injections, (b) end of nitrogen activation left tunnel, and (c) end of nitrogen activation right tunnel and excavation under brine maintenance left tunnel

At S. Giovanni, due to the electrification of Line A and the passage of large masses of metal (trains) the magnetic guidance system did not work properly, and the freeze hole tubes were bored using a down-the hole-hammer, with a very rigid casing, and its position checked via an optical system. This meant that a slightly larger diameter should be used for the tubes (140-150 mm). The extreme heterogeneity and the

© 2016 Deep Foundations Institute

large particle size of the soil, however, made it extremely difficult to control the direction of the freeze tubes, with substantial deviations from their intended positions. Moreover, as the freeze holes were designed to be very close to the existing piles of Line A station, which in some cases were larger than their nominal diameter of 2 m and in a different position than expected, it was necessary to abandon some of the freeze holes as the second row of piles was getting in their way. This made it necessary to integrate the freezing mask with additional or alternative freeze holes.
Figure 12 shows the actual position of the freeze holes belonging to the two vertical alignments on either side of the left tunnel, projected on a vertical plane (Figure 12a) and along the length of the tunnel (Figure 12b). The shaded areas represent problem areas, i.e., areas where there may be problems in the formation of the ice wall due to excessive deviations of two contiguous freeze holes.

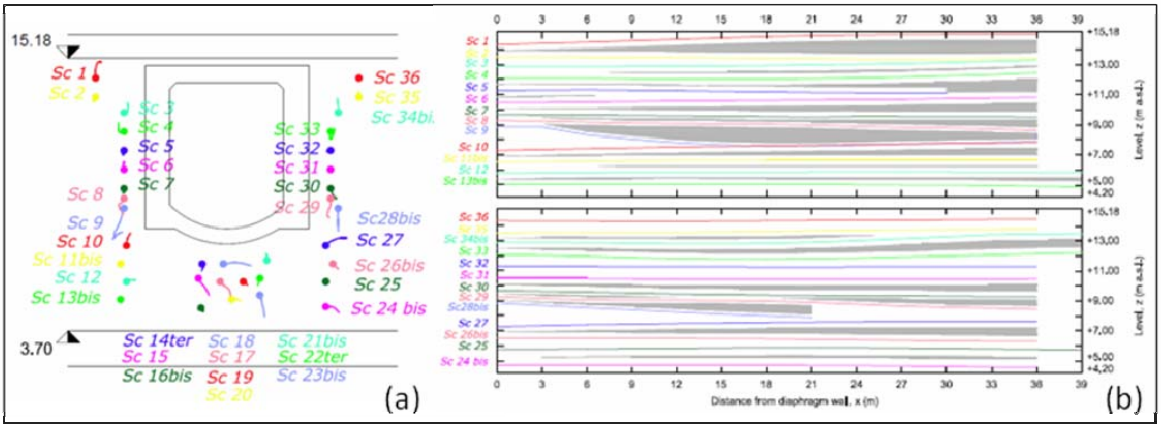


Figure 12: S. Giovanni: recorded positions of freeze holes around left tunnel: (a) vertical projection, and (b) profile along length of tunnel

To control freezing activation and maintenance, the soil temperature variations over time and distance from the freezing pipes were continuously monitored. Soil temperatures are measured by thermometers placed inside secondary boreholes of the same length as the freezing pipes. The thermometers consist of a string of thermocouples set at intervals of 3 to 5 m and placed at known and measured positions.

Directional drilling

As illustrated by the example reported in this paper, one of the main requirements for successful application of AGF in tunnelling is the ability to control the position of sub-horizontal freeze holes over significant lengths. The Trevi Directional Drilling Technology (TDDT) stems from the Horizontal Directional Drilling (HDD) technology used to lay utilities, such as, e.g., cables and pipes, underneath obstacles, such as water streams, roads, railways, structures, buildings, etc.. In turn, much of the technological equipment adopted in this sector comes from the oil industry. The system includes a number of devices that enable to drill small-diameter boreholes (50-200 mm) of considerable length, from a few tens to a few hundred metres. Drilling can be either rectilinear or curvilinear, with a radius of curvature depending on the diameter and thickness of the rods and casing being used. Directional drilling requires real time measurement of the position of the bore path and, if deviations from the intended position are recorded, correction of drilling.
In the magnetic guidance system (Figure 13a), the drill bit position is monitored using a sensor of magnetic field assembled on the drill string. Accuracy ranges from 50 to 200 mm, depending on the distance between the drilling bit and the reference electric field. In some cases, however, such as at S. Giovanni station, due to interferences with the electrification of Line A and the passage of trains, it is not possible to use with success the magnetic guidance system. An alternative option is the use of the optical guidance system, in which the position of the borehole is detected by an optical device, matching a target

© 2016 Deep Foundations Institute

within the drill string. The accuracy is of the order of 10 to 30 mm, but the system can only be used in rectilinear boreholes with a minimum diameter of 140-150 mm and a maximum length of about 40 m. The drilling control technique is based on two types of drilling tools: asymmetric or bevelled drill bits or fixed-bent subs. Both types are suitable for non-cohesive soft soils or stony soils and rocks. When driving the asymmetric drill bit into the soil with no rotation, its flat side is pushed away from the soil (Figure 13b and c).

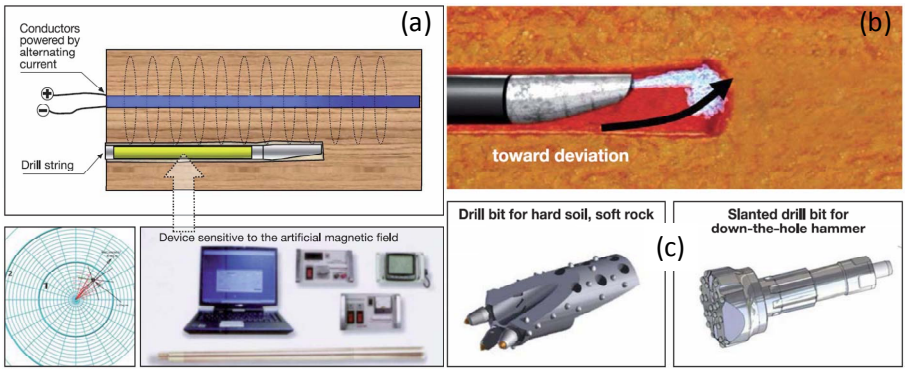


Figure 13: TDDT components: (a) magnetic guidance system, (b) asymmetric drill bit, (c) and (d) different types of drill bits

SUMMARY AND CONCLUSIONS

The Underpass Project of San Giovanni Station - Line A presented many technical challenges related to the restrictions imposed by the existing structures and by the complex stratigraphic and hydraulic conditions at the jobsite.
This experience will surely be useful for any other future applications of the Artificial Ground Freezing Method in complex stratigraphic, geotechnical and environmental contexts.

REFERENCES

Bertero A., Cribari F., Tanzi A., Fancello S., Gondolini T., Giannelli, F, Lodico M., D'Angelo M., Romani E. (2016). Metro Rome line "C" - Artificial Ground Freezing application for the underpass of the existing metro station San Giovanni - line "A". ITA-AITES WTC 2016, April 22-28, S. Francisco, USA.

Viggiani G.M.B., Casini F., Bertero A., Cribari F., Romani E., (2016). Artificial Ground Freezing: case studies, technological development, and fundamental research. XXIV Edition Turin Geotechnical Conference, February 25-26, Turin, Italy.

Haß H., Schäfers P. (2013). Application of ground freezing for underground construction in soft ground. In Geotechnical Aspects of Underground Construction in Soft Ground. Proceedings 5th Int. Symp. TC28. Amsterdam, the Netherlands, 15-17 June 2005, CRC Press, 405-413

Pimentel E., Papakonstantinou S., Anagnostou G. (2012). Numerical interpretation of temperature distributions from three ground freezing applications in urban tunneling. Tunnelling and Underground Space Technology, 28: 57–69.

Shuster, J.A. (1982). Engineering quality assurance for construction ground freezing. In Ground Freezing 1980: Developments in Geotechnical Engineering, ed. P. E. Frivik, N. Janbu, R. Saetersdal, and L. I. Finborud. Amsterdam, Elsevier, vol. 28, pp. 333–47.

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

WILLOW ISLAND HYDROELECTRIC PROJECT: TAMING THE OHIO RIVER TO CREATE GREEN ENERGY

James M. Tantalla, Ph.D., P.E., Mueser Rutledge Consulting Engineers, New York, New York, USA, 917-339-9300, jtantalla@mrce.com
Roderic A. Ellman, Jr., P.E., Mueser Rutledge Consulting Engineers, New York, New York, USA, 917-339-9300, rellman@mrce.com

ABSTRACT

Construction of the powerhouse for the Willow Island Hydroelectric Project, located at the Willow Island Lock and Dam, required a 120-ft deep temporary excavation inside the limits of a cellular cofferdam on the Ohio River side of the excavation and a soil-bentonite seepage cut-off wall on the landward side of the excavation. Stability analyses were performed for critical sections to evaluate the stability of the soil slopes and rock cuts using limit equilibrium, tension crack, and buckling analysis techniques. The stability analyses indicated adequate factors of safety through the landside embankment sections, and for the weighting berm soil slopes in front of the riverside cellular cofferdam. However, the results of deep-seated stability analyses through the rock supporting the riverside cellular cofferdam indicated marginal factors of safety for the 100-year flood condition. Consequently, the riverside portion of the excavation required enhanced stability measures, namely the installation of long, high-capacity rock anchors. Also, the dewatering system was modified to lower the site piezometric head to near the top of rock to improve the stability of the rock cuts.

Thirty high-capacity rock anchors were installed on the upstream, riverside, and downstream sides of the cofferdam rock cut. The rock anchors consisted of multiple, seven-wire strands designed to provide the full-required design load at 60% of the strand capacity. Rock anchor loads varied from 1,968 to 2,074 kips. The maximum number of strands per anchor was 59. Bond lengths assumed an allowable bond stress of 100 psi based on the rock bond from pull-out tests.

Keywords: excavation, river, cellular cofferdam, soft rock geology, stability, deep rock cut, dewatering system

PROJECT DESCRIPTION

American Municipal Power, a nonprofit organization that owns and operates electric generation facilities for its member communities, has been in the process of constructing hydroelectric power plants adjacent to existing navigation locks and dams along the Ohio River. The Willow Island Hydroelectric Plant was recently completed on the West Virginia bank of the river adjacent to the US Army Corp of Engineers (USACE) Willow Island Lock and Dam. The plant is on the opposite river bank to the locks, and includes two turbine generating units with FERC rated capacity of 44MW at a 20-foot head differential. The Ruhlin Company of Sharon Center, Ohio was awarded contracts to construct both the cofferdam and the powerhouse. Construction began in mid-2011, and was completed in early 2016.

COFFERDAM

The cofferdam for the project was designed to allow excavation and construction of the powerhouse and portions of the intake and tailrace channels in the dry while maintaining upstream to downstream pool separation before being flooded and decommissioned to allow final site preparation for plant operation. It was designed to provide protection for up to 100-year flood levels, which raise the river 20 feet (upstream) to 40 feet (downstream) over normal pool levels. The cofferdam has been outfitted with a needle beam floodgate and grouted rip-rap spillway to allow it to be filled if a more severe flood is predicted. Deep dewatering wells were used to draw down groundwater during overburden excavation, for stabilization of the cellular cofferdam, and for depressurization of bedrock during rock excavation. Primary components of

© 2016 Deep Foundations Institute

the cofferdam are shown in Fig. 1. The cofferdam was designed by Mueser Rutledge Consulting Engineers (cellular cofferdam, support of excavation, dewatering system, and rock anchors) and Geocomp/E.L. Robinson of Ohio (landside cut-off, stability analyses and instrumentation). The entire design was reviewed by the Owner's Engineer, MWH Americas, Inc., and was reviewed and approved by the Federal Energy Regulatory Commission (FERC) and the USACE. An aerial photo of the cofferdam during construction is shown in Fig. 2.

The cofferdam consists of a 50-foot deep, 3-foot thick soil bentonite cut-off wall constructed from the original site grade into bedrock covered by an impermeable clay cap up to 25 feet tall around the landside of the cofferdam. On the riverside, a cellular cofferdam consisting of sixteen 62-foot diameter, 67-foot tall cells was installed to the top of rock. The cofferdam allowed dry excavation for the powerhouse, including about 60 feet of rock excavation, while water levels on the Ohio River are more than 120 feet above the deepest excavation.

An extensive instrumentation system was installed on site in order to confirm the performance of the cofferdam during construction. The instrumentation system had three goals: to confirm the safety and stability of the cofferdam by monitoring movements throughout the site, to confirm global slope stability and dewatering system performance by monitoring water levels, and to confirm the performance of the rock anchors by monitoring load within them. Instruments used included an Automated Motorized Total Station (AMTS), in-place and manual inclinometers, tiltmeters, vibrating wire piezometers, flow meters, seismographs, and non-contact elasto-magnetic load sensors. The majority of the instruments were automatically monitored by data loggers with data uploaded to a web server for remote access to data on a near real-time basis throughout the construction.

SITE GEOLOGY

The top of bedrock was generally found at about El. 558 according to the Ohio River Datum. The bedrock consists of the Pennsylvanian Conemaugh Group, and consists of a variety of interbedded sedimentary rock types, including sandstones, siltstones, shales, claystones, and indurated clays with occasional limestone encountered during the project subsurface investigations. Rock stratigraphy was highly variable both vertically and horizontally throughout the site as illustrated in a sample cross section in Figure 3. Rock jointing was found to be generally low angle with varying dip directions. Evidence of slickensides was encountered in some of the borings, particularly in the finer grained sedimentary rocks.

The overburden at the site consists of alluvial sands and gravels overlain by finer grained silts and clays. The deeper, pervious sands and gravels allowed the use of a widely spaced deep dewatering well system for managing groundwater on the site. Thickness of the overburden varied from 0 feet in the river to as much as 60 feet thick on land, still several feet below the 100 year flood level the cofferdam was designed to protect against.

During construction of the existing dam, the USACE identified a fault beneath the planned overflow weir and Pier 9, both of which are within the footprint of the cofferdam for the construction of the powerhouse. A further investigation identified the fault with a strike of N(75-80)°W and dip of 30° south. The fault is up to several feet thick, and rock within the fault is highly fractured. Design of the cellular cofferdam, in particular deep seated sliding stability, carefully considered the presence of this fault on site. The fault also had the potential to be a conduit for groundwater flow into the excavation.

DESIGN SOIL AND ROCK PARAMETERS

Design soil and rock properties were established based on an evaluation of recent and historical subsurface investigations and laboratory testing from as far back as 1959, when preliminary design work for the

© 2016 Deep Foundations Institute

original lock and dam began. A pump test was also conducted to better define the hydraulic conductivity to help understand the performance of the dewatering system for rock depressurization. Laboratory testing of soil and rock samples included standard index tests, consolidation tests and strength testing of the soil and rock. Special laboratory strength testing of the rock included direct shear testing of slickensided rock surfaces, anchor bond pullout tests, and petrographic analyses of slickensided rock samples to understand the degree of faulting and orientation of clay surfaces.

The design rock properties for stability analyses, anchor block and support of excavation design are:

Table 1 – Design Rock Properties

Material	Total Unit Weight	Horizontal Surfaces		Buckling Cross-Bed Shear		Deep-Seated Sliding Cross-Bed Shear		Limit Equilibrium Cross-Bed Shear	
	γ_t, pcf	c', psf	φ', deg	c', psf	φ', deg	c', psf	φ', deg	c', psf	φ', deg
Sandstone	160	0	35	0	50	0	50	7,200	50
Siltstone	160	0	14	0	45	0	45	12,960	45
Claystone	160	0	14	0	24	0	24	1.296	24
Shale	160	0	14	0	45	0	45	2,160	45

CELLULAR COFFERDAM DESIGN

The design for the Cofferdam and Excavation Contract of the Willow Island Hydroelectric Project includes the construction of sheet pile cellular structures as part of the cofferdam to prevent river and groundwater from infiltrating into the excavation for the construction of the powerhouse. The cellular cofferdams have been designed such that the cells remain stable during installation, dewatering, excavation, and throughout powerhouse construction.

Both internal and external stability checks have been performed for cells on the upstream and downstream sides of the Willow Island dam. Additionally, deep seated stability of the cofferdam and underlying rock mass has been considered post-excavation. The cellular cofferdams have been checked for sliding, overturning, pullout, bursting, and shear following methodologies prescribed by the U.S. Army Corps of Engineers (USACE). Minimum factors of safety for each condition have been established in accordance with USACE and Federal Energy Regulatory Commission (FERC) guidelines and were met for a variety of different load cases.

Stability checks were performed in accordance with the recommendations found in the Army Corps' Engineering Manual EM 1110-2- 2503 "Design of Sheet Pile Cellular Structures." Several checks were performed to confirm the external stability of the cell as a whole, including sliding, overturning, and deep seated sliding. Internal stability checks confirmed the performance of the individual components of the cellular structure, and included pullout, interlock tension, and vertical and horizontal shear. Maximum water levels within both the cells and support berms were determined based on expected drainage performance of the cells, and overall stability.

The following configuration was used for the construction of the cellular cofferdam:

- Cell diameter: 62.68 feet.
- Cell sheet count: 120 sheets per cell.
- Cell height: approximately 67 feet.
- Arc radius: 15.48 feet.
- Sheets used to construct cells and arcs: PS 27.5 straight sheet. Wyes: PS 31 straight sheet.

© 2016 Deep Foundations Institute

- Minimum interlock strength = 24 kips/inch.
- Minimum soil overburden during placement and initial filling: 8 feet.
- Final berm: fill to EL 600 upstream and downstream after dewatering to top of underwater berm.

SLOPE STABILITY ANALYSIS

The slope stability analysis of the excavation was analyzed using 2D limit equilibrium techniques in accordance with USACE EM 1110-2-1902 "Slope Stability" and by the tension crack "wedge" method in accordance with EM 1110-2-2503 for Cellular Cofferdam analysis sections. A total of 13 design sections were analyzed. Due to the presence of slickensides and the potential for weak planes in the indurated clay, claystone, and siltstone, a low design friction angle of 14 degrees was selected for horizontal failure surfaces in rock. Results of the analyses showed inadequate deep seated stability factors of safety through the rock for the steeply cut (1H:5V in rock) powerhouse excavation on the river side and the flatter intake and tailrace design sections under 100 year flood conditions.

Further complicating stability is the presence of a fault passing through the site. The fault had previously been encountered during the construction of the original dam. That structure underwent a redesign during construction after it was exposed to address stability. Based on the location of the fault relative to the excavation for powerhouse construction, the potential for stability issues existed in significant rock blocks up to 200 feet wide by more than 40 feet deep. The computer program SWEDGE was utilized to define multiple intersecting planes to model a rock wedge configuration formed by the fault, the rock cut, and a release plane. Driving forces and resisting forces on the fault and planes were calculated to derive the factor of safety on sliding. The analyses determined that a stabilizing load of 5,926 kips was needed to meet minimum safety factors for the fault block.

The evaluation of the results of deep seated stability analyses established that supplemental support was needed to allow the necessary excavation on site. The primary means of support selected were rock anchors. Additional steps were taken to minimize the required anchor loads. Required anchor line loads under 100 year flood levels were quite high, because the tension crack "wedge" analysis required to determine deep seated stability is highly sensitive to the depth of the excavation, as hydrostatic driving pressures increase more rapidly than the frictional resistance at the base of the potential wedge. Reduction in hydrostatic pressure and resulting rock anchor loads was accomplished by deep depressurization wells based upon the results of pump tests performed on site during preliminary engineering.

The construction of the powerhouse included the installation of a more than 11-foot thick base slab throughout the powerhouse footprint. This slab, once sufficiently built up, braced the deep excavation to the rock on the other side of the excavation. The design was modified to utilize a lower design flood level while the deepest portion of the excavation was exposed. This "intermediate" flood level, equivalent to about a 10 year flood, lowered design water levels by more than 10 feet relative to the 100 year flood. This reduction in design water level reduced the load on the deepest failure surfaces, and the resulting rock anchor loads. The cofferdam was returned to 100 year flood protection as soon as the powerhouse base slab construction (which provided the remaining support) was complete.

ROCK ANCHOR DESIGN

With the reduced design flood level during the deepest excavation and depressurization wells installed to improve stability, the final required anchor demand varied from 42 to 86 kips per linear foot to improve deep-seated stability factors of safety above required minimums. In addition, almost 6,000 additional kips of support was required to stabilize potential fault blocks.

© 2016 Deep Foundations Institute

The rock anchors were designed in accordance with USACE EM 1110-1-2908 "Rock Foundations" and Post-Tensioning Institute (PTI) recommendations. The design utilized thirty high-capacity anchors that utilized fifty-nine 0.6 in. diameter, seven wire strands designed to provide 2,074 kips of support at 60 percent of the strand capacity.

The rock anchors were placed as close to the outside of the cofferdam as practical where they would provide the most benefit to the stabilization of the rock mass and allow additional intake and tail race channel excavation while the anchors were still engaged. Soldier pile and lagging support of excavation was installed in some areas to further increase the amount of excavation that could be performed inside the anchors.

Two standardized anchor block designs that could accommodate two or three anchors were used around the site. Spacing between blocks was varied in the different design sections to provide the required resistance within the tributary length of the design sections, and site geometry and construction constraints.

The anchors were nominally 12-inches in diameter, at an inclination of 45 degrees. Bond zones were all 50 feet long. Free lengths were determined as the minimum length that would a) prevent potential rock mass pull out (even though the anchors were installed into the mass and not an external structure), b) keep the anchor bond zone below all potential failure surfaces requiring stabilization, c) keep the anchor bond zone 5 feet below the most critical failure surface identified by limit equilibrium analysis, and d) meet the requirements of USACE EM 1110-1-2908. Based on these requirements, free lengths ranged from 77 to 88 ft.

Anchor bond stresses

A supplemental subsurface investigation was performed to collect large diameter rock cores for testing. Pullout tests, in which a threaded steel bar is grouted into a hole drilled in the core and then stressed to failure to determine the grout to rock bond strength, were performed on two different samples of each of the rock lithologies found on site (limestone, sandstone, siltstone, and claystone). Test results averaged from 214 psi (claystone) to 579 psi (siltstone) for the different rock types.

While the test results for claystone showed a reasonable average pullout stress, one of the tests showed a low result of just 78 psi. Additional evaluation of the claystone was undertaken to better understand the potential bond strength in this rock type.

Unconfined compressive strength (UCS) test data was available in this area of the site from both the original USACE design of the dam, the investigation performed for the design of the hydroelectric plant, and the supplemental subsurface investigation performed for rock anchor design. Bond stress was related to the UCS test results based on the following correlation:

$$\sigma_{b\,ult} = 0.1 \cdot \mathrm{UCS}_{peak}$$

The equivalent ultimate bond stresses from the UCS tests were then plotted with the results of the pullout tests in histogram form and a cumulative lognormal distribution (μ=5.55, σ=0.755) was fit to the data, as shown in Fig. 4. A lower third pullout value for the claystone of 190 psi was selected. Combined with the other rock lithologies in the bond zones, a conservative ultimate design bond stress of 200 psi was selected.

During review of the anchor design, there were concerns about the performance of the bond zone, both in the short term and long term due to creep. Several measures were taken to prove out the performance of the anchors. All anchors were water tested in the bond zone to confirm the quality of the rock present prior to installation, and a two-stage grouting procedure, where the bond zone was grouted, tested, and stressed

© 2016 Deep Foundations Institute

prior to grouting the anchor free length, was utilized. The first anchors that were installed were successfully performance tested to 133% of the design load to confirm the design capacity. An extended creep test was carried out on one of the performance tested anchors. This test showed very good creep performance even at the full design load. All remaining anchors were proof-tested, and locked off at 105% of the design load.

The design included several contingencies in case the target bond was not reached. Secondary grout tubes were included in the anchors, and would have been utilized for post grouting had any of the anchors not passed testing. Had the post-grouting not provided sufficient capacity, the anchor blocks, which were designed for two or three anchors, were designed with spare sleeves to accommodate the installation of additional supplemental anchors, and would have allowed for a 33 percent reduction in bond stress. Fortunately, the system worked as designed, and these features did not need to be utilized.

GROUNDWATER AND SEEPAGE CONTROL

The seepage control system needed to meet certain performance requirements both to meet specification and to maintain stability of the cofferdam in the excavated condition. As part of the seepage control system, the soil bentonite cut-off wall was required by specification to be at least 85 percent effective. What this meant is that for a given level of drawdown, groundwater levels 50 feet inside the cut-off wall need to be no higher than 15 percent of the total drawdown above the deepest point in the excavation. In other words, 85 percent of the groundwater drawdown within the cofferdam needed to occur within 50 feet of the cut-off wall.

In addition, results of analyses in some areas required more stringent drawdown to maintain excavated slope stability. Excavation within the cofferdam required sufficient dewatering to allow for excavation of soil in the dry and depressurization of bedrock to maintain excavation stability throughout powerhouse construction. The initial plan for dewatering included a ring of twenty deep dewatering wells around the site as shown in Fig No. 5. A total of forty piezometers installed around the site, within cells, excavated slopes, cell support berm, and outside the cofferdam were used to monitor groundwater conditions and ensure that water levels were sufficiently drawn down to maintain stability and meet cut-off wall effectiveness requirements on a near-real time basis throughout the construction.

Dewatering wells were installed in 12-inch diameter holes with a 6-inch diameter PVC riser. A typical well on the land side of the cofferdam where rock cut stability was not a concern extended seven feet into rock. Wells on the river, intake and tailrace sides of the cofferdam extended approximately sixty feet into rock to allow for deep depressurization in order to improve excavation stability. The overall system capacity was 3,000 gallons per minute, and was discharged into an on-site treatment plant.

In addition to the dewatering wells, a highly permeable toe drain was installed at the toe of the soil excavation at the top of rock. The toe berm acted as a collector for groundwater that could flow through higher permeability soils that were found directly above the rock at the site. In addition, finger drains were installed in the cell support berm at the ordinary downstream pool elevation to further increase the drainage capabilities of these berms. These two systems in concert were intended to manage groundwater on site after excavation to the top of rock was complete with minimal pumping from the dewatering wells.

Additional flow in upstream cell support berm

After the cofferdam was constructed and excavation to the top of rock was completed, further excavation into the cellular cofferdam support berms was needed to install support of excavation and high capacity rock anchors. During this excavation, substantial flow developed on the upstream side of the cofferdam. Groundwater flow caused significant erosion of the clean cell support berm fill. The source of water could not be determined, but was thought to be river water that was entering the cofferdam either through

© 2016 Deep Foundations Institute

communication through the fault or through potential windows at the bottom of multiple cells on the upstream side of the cofferdam.

A total of three additional, supplemental wells shown in Fig. No. 5 were added in areas where piezometers indicated limited drawdown due to existing dewatering. Between these wells, and careful, deliberate continuation of the excavation, flow through the support of excavation in this area eventually slowed down to minimal quantities.

Cellular cofferdam decommissioning stability

Design calculations for cellular cofferdam stability included assumptions about the expected drainage of the fill within the cells in accordance with USACE Engineering Guidance. Due to the high permeability of the cellular cofferdam fill, a 1H:1V phreatic surface was assumed in each cell for global sliding and stability calculations. Instrumentation on site included piezometers in each of the cells capable of confirming these assumptions. Due to difficulties getting a good seal on the existing dam spillway, the fault that passes below several cells, and potential windows at the base of the cells due to sheet pile refusal during cofferdam construction, several cells exhibited higher than assumed water levels under normal pool conditions.

During cofferdam decommissioning, the cellular cofferdam support berm was removed prior to watering up the cofferdam. If high water conditions had occurred during this time, the higher water levels in the cells could have compromised sliding stability without the berm in place. In order to address this potential stability issue, dewatering wells were installed within nine of the cells. Since the cells were stable at normal pool conditions, the wells were not continuously pumped. After installation, pump tests were performed to confirm that adequate drawdown could be obtained by the wells. The wells would have been turned on to lower cell water levels if there was a prediction for high water on the Ohio River. Fortunately, there were no high water events during decommissioning, and the wells were removed with the cells.

CONCLUSIONS

The combination of groundwater control systems (dewatering wells, toe drains, cellular cofferdam, and soil bentonite cut-off wall) worked together as designed to allow the cofferdam to maintain stability during the entire hydroelectric plant construction. Effective use of deep depressurization within bedrock was utilized to help stabilize a substantial rock cut and reduce supplemental rock anchor demand. Dewatering was used to improve stability of cells during cofferdam decommissioning, which was completed in summer, 2015. The cofferdam performed well during several high water events, but did not experience a 100-year flood or greater that would have required emergency flooding. Final commissioning work on the plant was completed in early 2016, and the plant (shown in Fig. 6) is now online and producing power.

REFERENCES

- FERC, 2010. Engineering guidelines for the evaluation of hydropower projects. Federal Energy Regulatory Commission.
- PTI Committee, 2014. Recommendations for prestressed rock and soil anchors. Post-Tensioning Institute, 98 p.
- Terzaghi, K. and Peck, R.B., 1948. Soil mechanics in engineering practice. New York, John Wiley and Sons, 566 p.
- U.S. Army Corps of Engineers, 1989. Design sheet pile cellular structures, cofferdams and retaining structures. Engineering Manual EM 1110-2-2503, 186 p.
- U.S. Army Corps of Engineers, 1994. Rock foundations. Engineer Manual EM 1110-1-2908, 120 p.
- U.S. Army Corps of Engineers, 2003. Slope stability. Engineer Manual EM 1110-2-1902, 205 p.

© 2016 Deep Foundations Institute

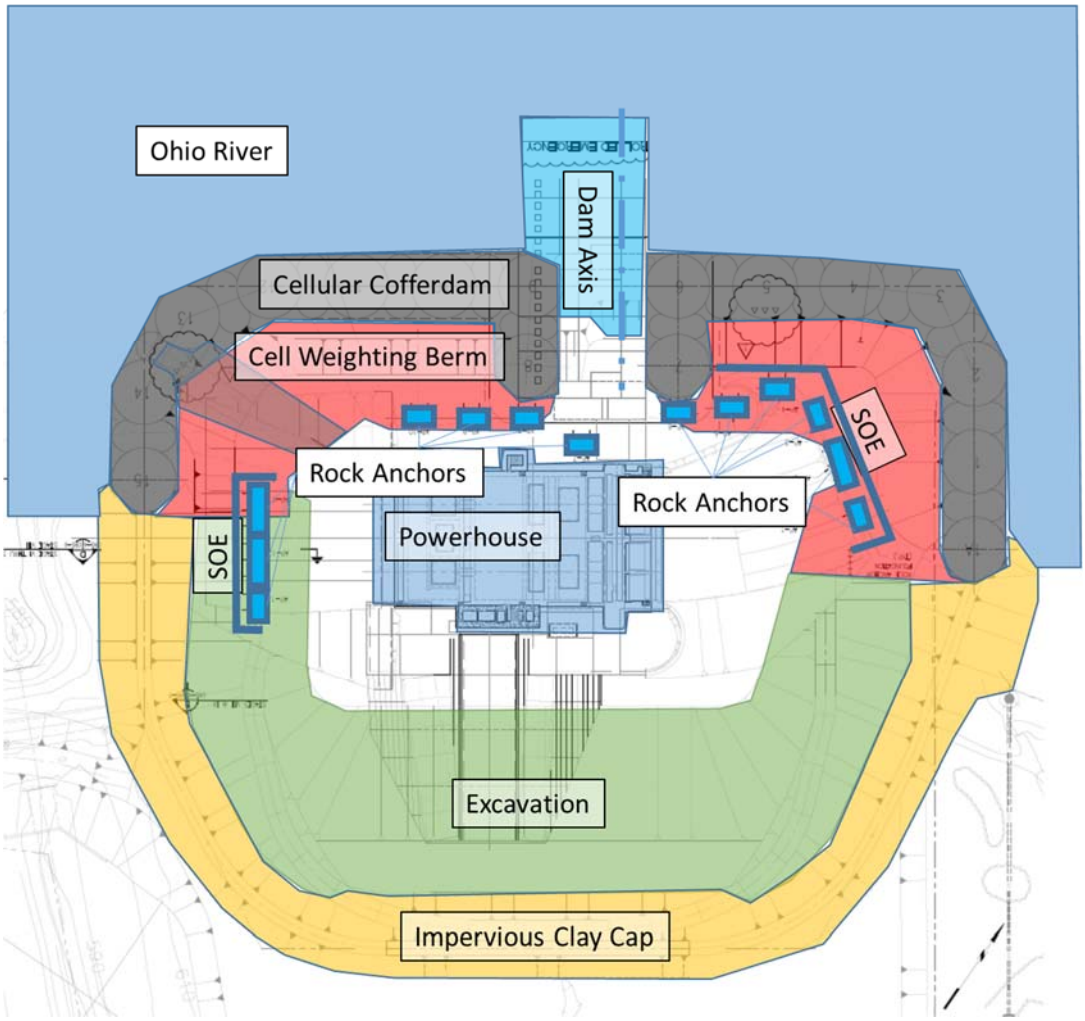
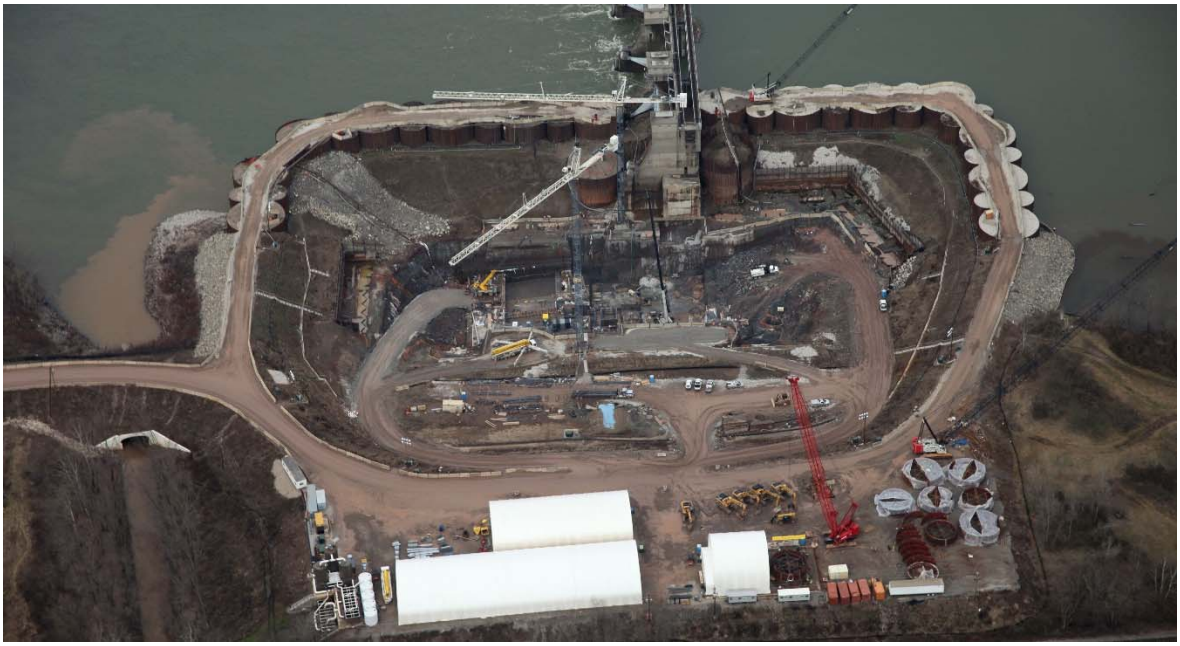


Figure 1 – General Cofferdam Configuration


Figure 2 – Aerial Photo of Cofferdam

© 2016 Deep Foundations Institute

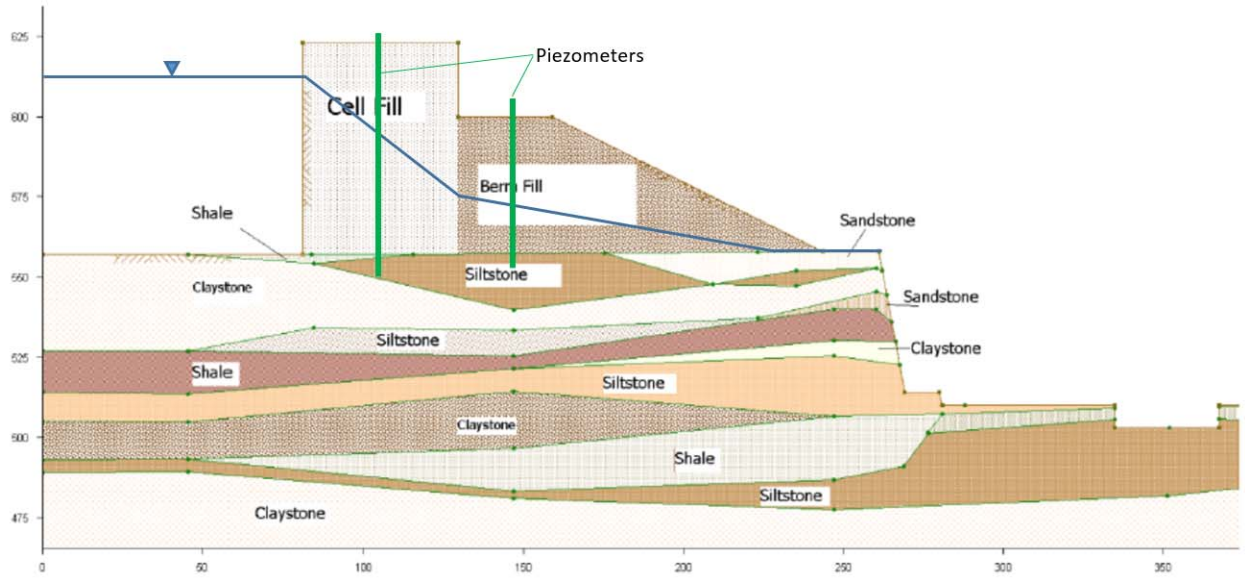


Figure 3 – Sample Rock Stratigraphy

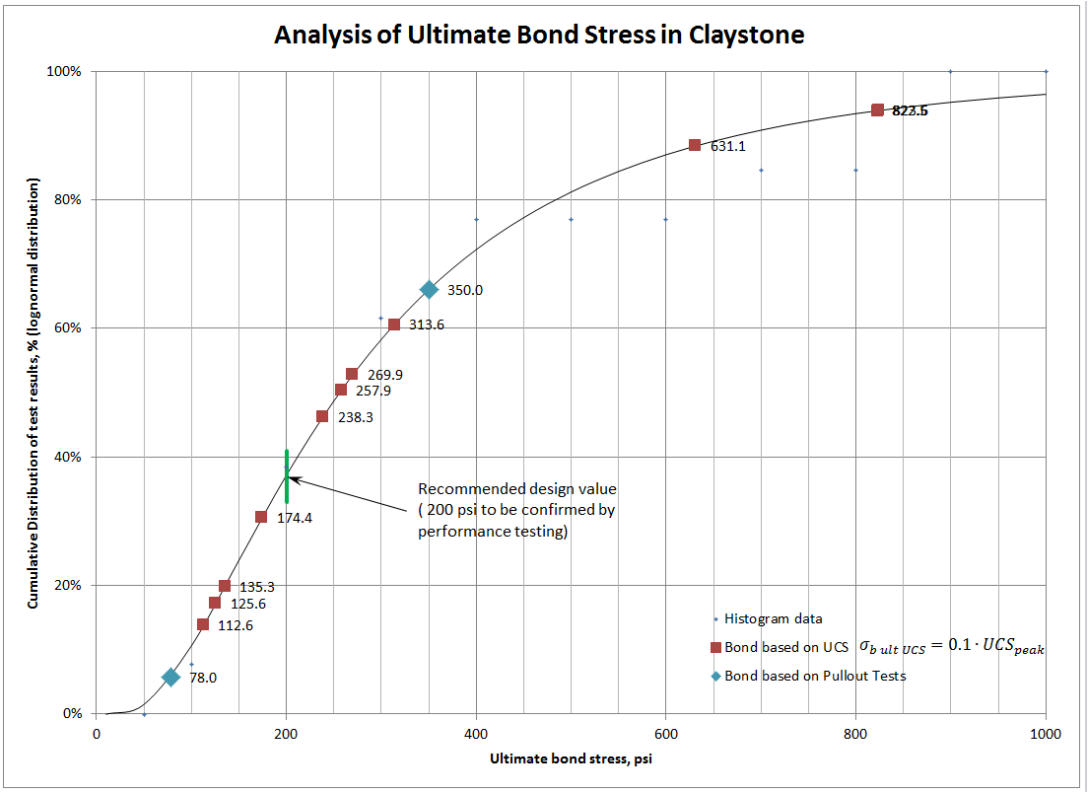


Figure 4, Distribution of Ultimate Bond Stress in Claystone.

© 2016 Deep Foundations Institute

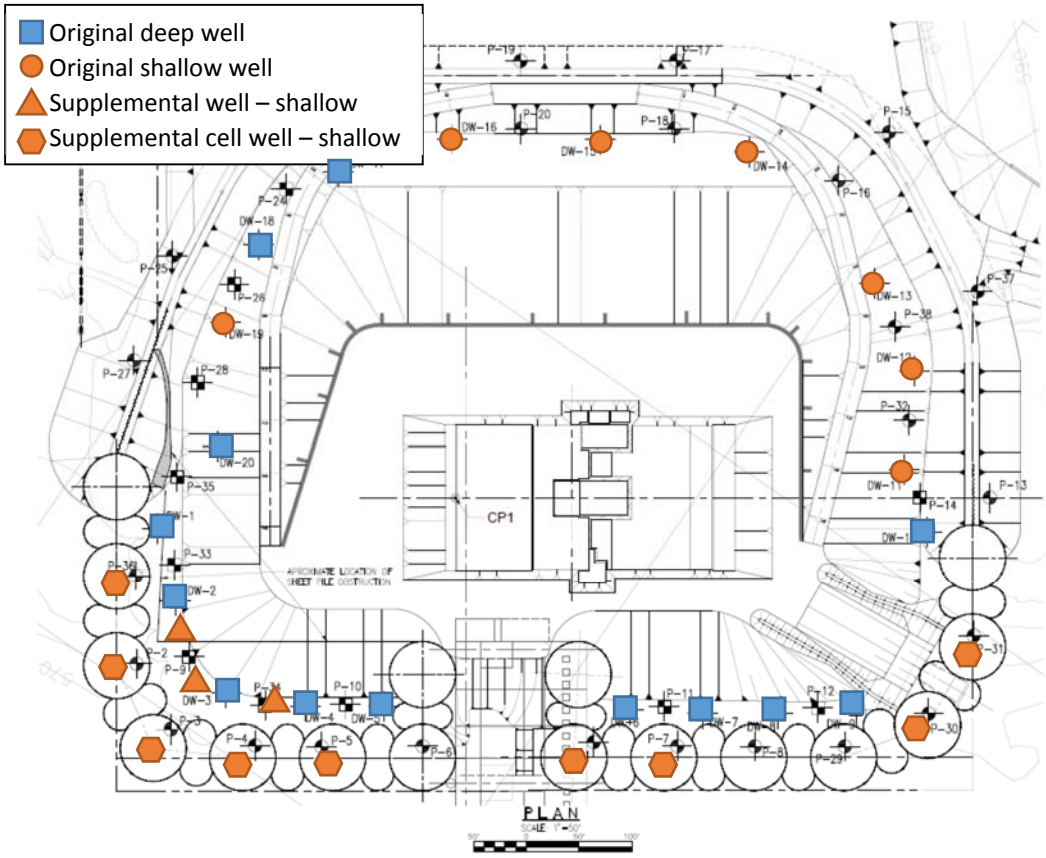


Figure 5 – Dewatering / Depressurization well locations



Figure 6 – Willow Island Hydroelectric Plant in operation

© 2016 Deep Foundations Institute

INTERACTIVE 3-DIMENSIONAL MODELING OF ANCHORS AT BLUESTONE DAM

Jamey Rosen, Geosyntec Consultants. jrosen@geosyntec.com
Georgette Hlepas, Dam Safety Modification and Mandatory Center of Expertise, United States Army Corps of Engineers. Georgette.Hlepas@usace.army.mil

ABSTRACT

Bluestone Dam is a concrete gravity structure with an overall length of 2,060 feet and a maximum height of 165 feet above the streambed. Several major project modification designs have been implemented and additional modifications are currently in construction and design phases. Due to the various modifications since original construction was completed in 1948, detailed documentation with visualization of the project features is needed. A three-dimensional model of the internal and external features of Bluestone Dam was generated from LiDAR data, survey information, and other data sources. One of the key features of interest in this model is an accurate visualization of over 200 dam anchors based on downhole anchor surveys. The model is accessible in Microstation and Geographic Information Systems (GIS), allowing users to rotate the anchors in three dimensions; view them in the context of geology, internal dam structural features, historical construction records, site photographs, bathymetric data, and instrumentation data. Users can "click" on individual anchors to view associated construction details and open a comprehensive installation packages. The model will be used by the United States Army Corps of Engineers (USACE) as part of a larger Geospatial Data Management System as a historical accounting of the project construction data, and be a long-term living document for use by both design engineers and operations staff to assess dam safety risk and to design future project modifications.

Keywords: Database, GIS, Dam Safety, Anchor

INTRODUCTION

During the life of any project, critical data concerning the construction and performance history are required to be collected and maintained. The data collected prior, during, and post-project construction help with the design process, ensure that the project is built to specifications and is performing as per design intent. During the Post-Implementation Evaluation (an evaluation performed after a major project modification), these data are used to visualize the repairs, verify the construction was performed per design intent, and document the history of the construction phases, associated test results, and overall effect on the project. The project described in this paper is the construction of a system of tools to effectively use these data.

Data collected and maintained during the operations phase helps to ensure that the project is continuing to perform as per design and aids in advance warning of any potential failure or required maintenance. Dam Safety, Emergency Management (EM), and Operations and Maintenance personnel can use the system to verify the status of the project in preparation for a high flood event. For example, if there is repair work being performed on a critical feature of the project that would affect the operation of the dam structure during a flood event, Operations personnel would be responsible to update the status of the system such that Dam Safety and EM would be able to identify the feature that is out of service, its date of expected repair completion, and the implications during a flood. In addition, risk assessments typically require a full history of the project and its performance. Any concerns that arise concerning the safety or performance of a project will require a full documented history of the project and its components. Proper documentation and data management are key in this assessment process.

USACE project engineers identified the need for a Bluestone Dam Data Management System (BlueDMS) to address these needs. The intent of the BlueDMS is to provide a complete 3D geospatial data

© 2016 Deep Foundations Institute

management system which uses readily available software technology in order to better organize, manage, analyze and present project site geography, features, stratigraphy, geology, construction details, and performance history. As an end product, the 3D model along with project data will be readily available to USACE team members.

PROJECT BACKGROUND

Bluestone Dam is located in southern West Virginia in Summers County within the New River Basin, which is a sub-basin of the Kanawha River Basin. Bluestone Dam is located approximately one and a half miles upstream of the City of Hinton and a half mile upstream of the confluence of the New and Greenbrier Rivers. Bluestone Dam is a straight, concrete gravity structure with an overall length of 2,060 feet and a maximum height of 165 feet above the streambed. Discharge capacity of the existing structure consists of 21 spillway crest gates and 16 gated sluices in the 790-foot wide main spillway and 6 gated penstocks that pass flows through the Penstock Intake Section into the penstock stilling basin (auxiliary spillway). Bluestone Dam began operation in 1949.

In 2008, a Dam Safety Action Classification (DSAC) II (urgent) was assigned in 2008. As a result of the DSAC, modifications to the original design have been implemented while others are undergoing construction and/or are in design phases. This work includes extension of the penstocks, construction of thrust blocks, training walls, and gravity walls, installation of over 200 high capacity anchors in critical monoliths, over 250 high strength steel strand anchors in the spillway and non-overflow monoliths, decommissioning of gallery drains, installation of new gallery drains and uplift cells, and construction of a scour pad with baffle blocks downstream of the penstock extension.

Geosyntec Consultants was selected as the contractor to develop the BlueDMS, with the mandate to document the original construction, track the various project modifications, verify construction specifications were met, and ensure the resultant construction meets the design intent necessary to reduce the overall risk associated with the project. This 3D geospatial data management system will ultimately serve as a historical record of the project and will be utilized by both operations and engineering personnel to monitor and evaluate project performance.

USACE identified several work products to comprise the BlueDMS and meet the project needs. These include:

- 3D Microstation model;
- 3D GIS model;
- 2D GIS projects (web and desktop);
- Database constructed from project documents; and
- Tools for data import and export

These work products were envisioned to be built from the following data sources:

- Light Detection and Ranging (LiDAR) data collected from the dam exterior and interior;
- Bathymetric survey data;
- Spreadsheets with tabulated details and attributes from dam structures, anchors, instruments, historical grouting, Monolith factor of safety simulation results, and more;
- A gINT Borehole database;
- Historical construction and as-built drawings; and
- Anchor documents

© 2016 Deep Foundations Institute

DAM MODELING WITH MICROSTATION AND GIS

Key work products for this project included models intended for USACE to have as an archive of project data and as tools to guide future construction activities. These models included a three-dimensional model developed in Microstation, and models developed using GIS technology in 2D (plan, profile, and cross-sectional) and 3D. The general workflow used to construct these models included:

1. Build a relational database from digital and digitized project documents (historical records, construction and as-built drawings, spreadsheet archives, etc.)
2. Create GIS features (vector and raster) from geospatial information in the database (e.g., coordinates of instrument locations, downhole anchor survey data, station/offset of borehole locations, etc.). Add these features to GIS and Microstation models
3. Import LiDAR cloud data into Microstation, and use it to derive 3D features (internal and external dam structures, concrete lifts, etc.). Convert these features into GIS data types and add them to the GIS models
4. Join (link) and relate GIS features to tables and views in the database to allow access to these attribute data from the models
5. Compile scanned documents that relate to dam features. Build a database table that relates these documents to the GIS feature names, allowing them to be opened as spatial hyperlinks from the GIS models
6. Symbolize (e.g., color-code, etc.) GIS features by linked attribute data
7. Publish the GIS files to desktop and web software accessible by USACE

At each of the above stages, various database, Microstation, and GIS tools were developed or customized to facilitate data assembly, geospatial feature construction and projection (for example, data were available in two different state plane coordinate systems [NAD27 and NAD83 with associated vertical datums] as well as in station/offset), software file type conversions, and more. Some specific data sources and modeling activities are detailed below.

Key Data Sources

LiDAR Data Interpretation

The LiDAR surveys conducted on the exterior and interior of the dam represent the primary data source used to render the dam structures accurately in three-dimensions. The largely manual task of interpreting point cloud into solid features was performed using Microstation tools:

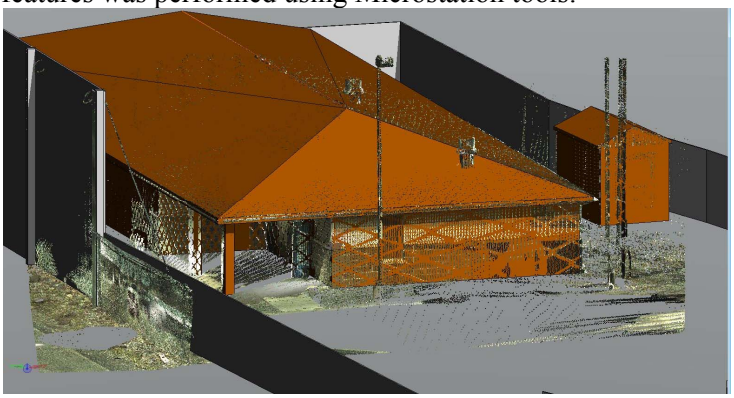


Fig. 1. Generation of structures from LiDAR point cloud in Microstation

© 2016 Deep Foundations Institute

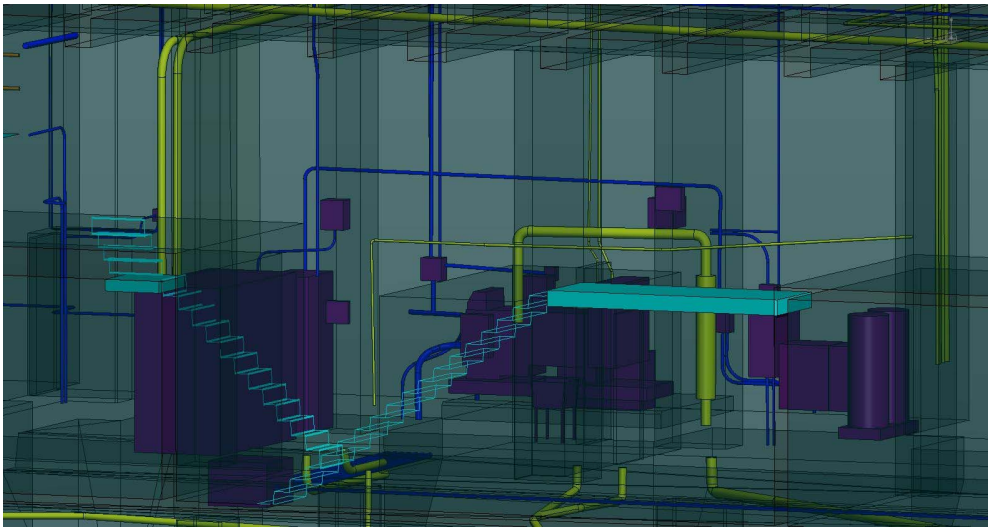


Fig. 2. Dam internal features digitized from LiDAR points

gINT Database Usage

Borehole lithological and geotechnical data at Bluestone Dam are stored in gINT software. In order to use this resource in the Microstation and GIS models, we imported the gINT Database tables into the database, keeping the gINT table structure intact. This allowed the gINT data to be converted to GIS features by using the gINT locations and depths to calculate the starting and ending 3D coordinates (accounting for inclined boreholes) of boreholes and each sample, or lithological or soil unit within the borehole. We then developed formulas to symbolize and label the boreholes to show the lithology, RQD, etc.

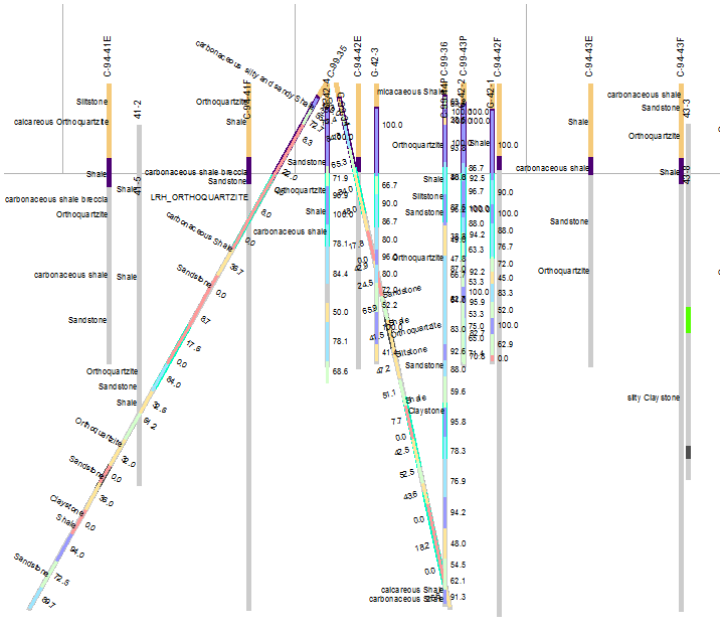


Fig. 3. GIS Visualization of gINT data showing lithology, core loss, and RQDs

© 2016 Deep Foundations Institute

Anchor Data Assembly

The high-capacity rock anchors installed at Bluestone Dam are a significant component of the project's rehabilitation and required special handling in the BlueDMS. The database was populated with several related tables designed to store the anchor as-built construction details, the results of downhole surveys, and data associated with the anchor composition, pre-grouting methods and laboratory analyses, the results of watertightness and stress tests, and more. These data were compiled mostly manually from Anchor Closeout Packages and as-built drawings.

To accurately visualize this information, the downhole survey data were interpreted into 3D GIS features that show the subtle deviations in the anchor shape along its length. Various components of the anchor composition (stressing length and bond length) were mapped to these features and color-coded by several attributes, allowing (for example) to view the components of each anchor in the context of surrounding geology, dam structures, etc.

Blue DMS Components

Database

All data compiled, imported and inputted into the BlueDMS from primary sources are stored in the Enterprise Database (EDB). The EDB was developed in Microsoft SQL Server, and is accessible via web and GIS tools described below. Most of the GIS features are either created within the EDB, or are linked to tables in the EDB (i.e., to assign attributes to their geometry). The raw data tables in the EDB can be accessed directly over the internet through a Microsoft Access interface.

Microstation Model

The Microstation model is intended to show the accurate detailed positions and shapes of the modeled features. While it contains most of the same information as the 3D GIS model described below, it's purpose is to inform future construction and as-built drawings, and for compatibility with USACE tools designed to work with Microstation.

GIS Model

The 2D and 3D GIS models are intended to relate the spatial features created in Microstation and created in the EDB (or from other sources) to attribute data stored in the EDB. This allows features to be queried and spatially analyzed, color-coded by attributes, and used to generate maps and reports.

© 2016 Deep Foundations Institute

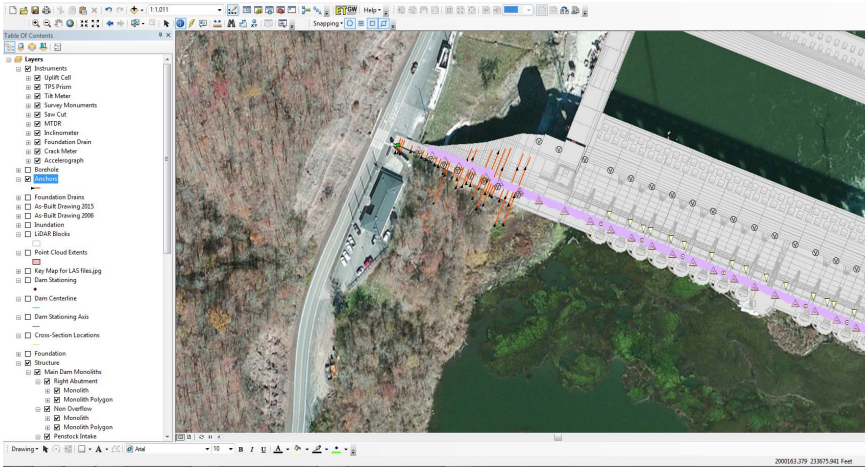


Fig. 4. Plan View Desktop GIS showing Structures, Anchors, and Instruments

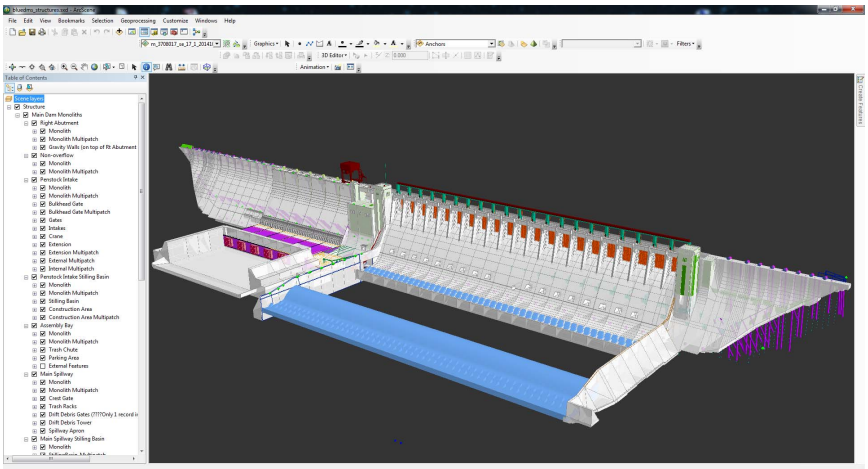


Fig. 5. 3D GIS Model showing upstream view of external dam features and GIS Table of Contents

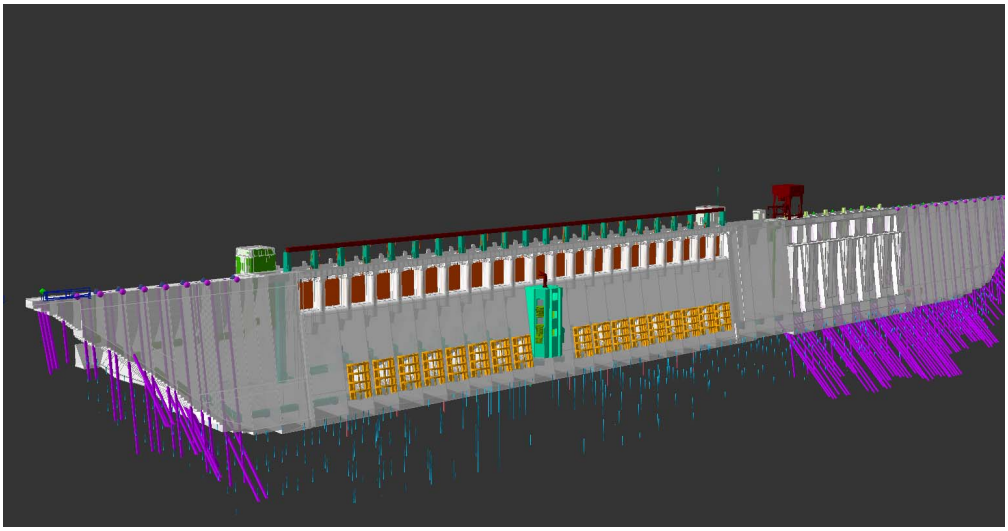


Fig. 6. 3D GIS Model showing downstream view of monoliths, trash rack, and anchors

© 2016 Deep Foundations Institute

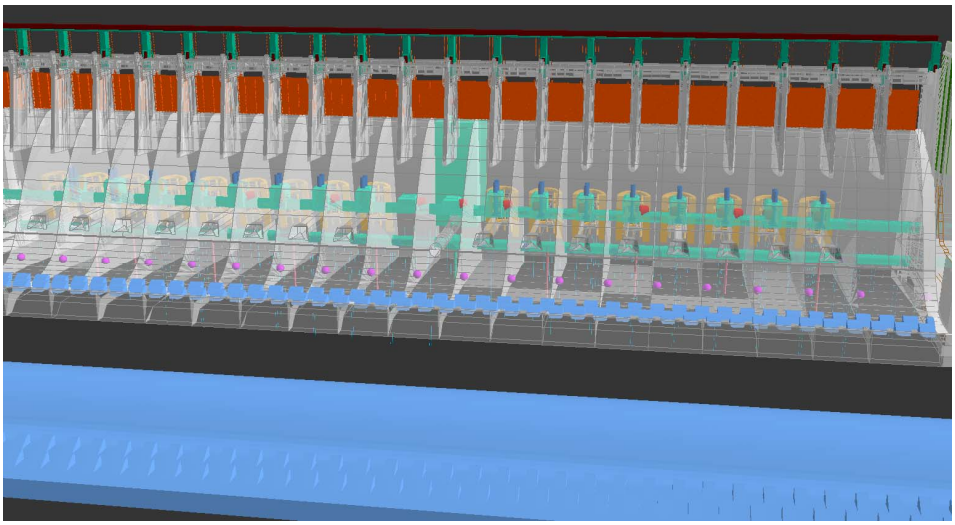


Fig. 7. 3D GIS Model showing transparent monoliths and internal structure

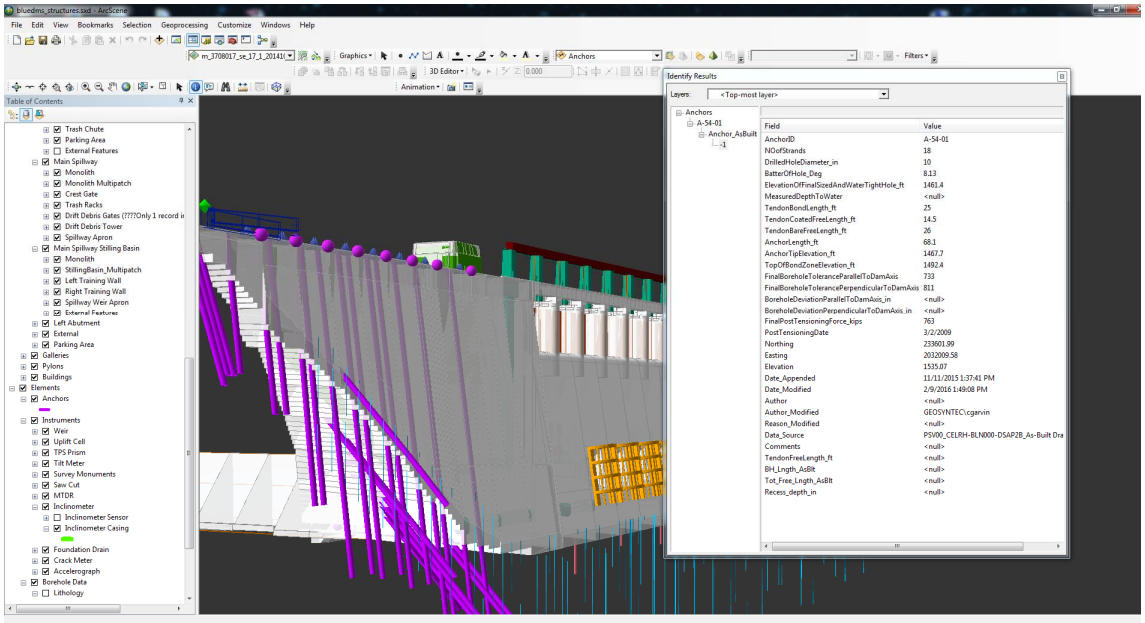


Fig. 8. 3D GIS Model showing use of Identify Tool to view database data for associated anchor

Web GIS

Two web-based GIS interfaces were developed and deployed to allow quick read-only access (i.e., without the need for software other than a web browser) to GIS data in plan and profile view. The web interfaces allow visual read-only access to data, with common GIS view functionality (navigate, measure, search, query/filter, identify, hyperlink, etc.). The data sources in the web interfaces are directly linked to the EDB, allowing instant visualization as data are uploaded. This allows USACE to track the progress as the database is constructed.

© 2016 Deep Foundations Institute

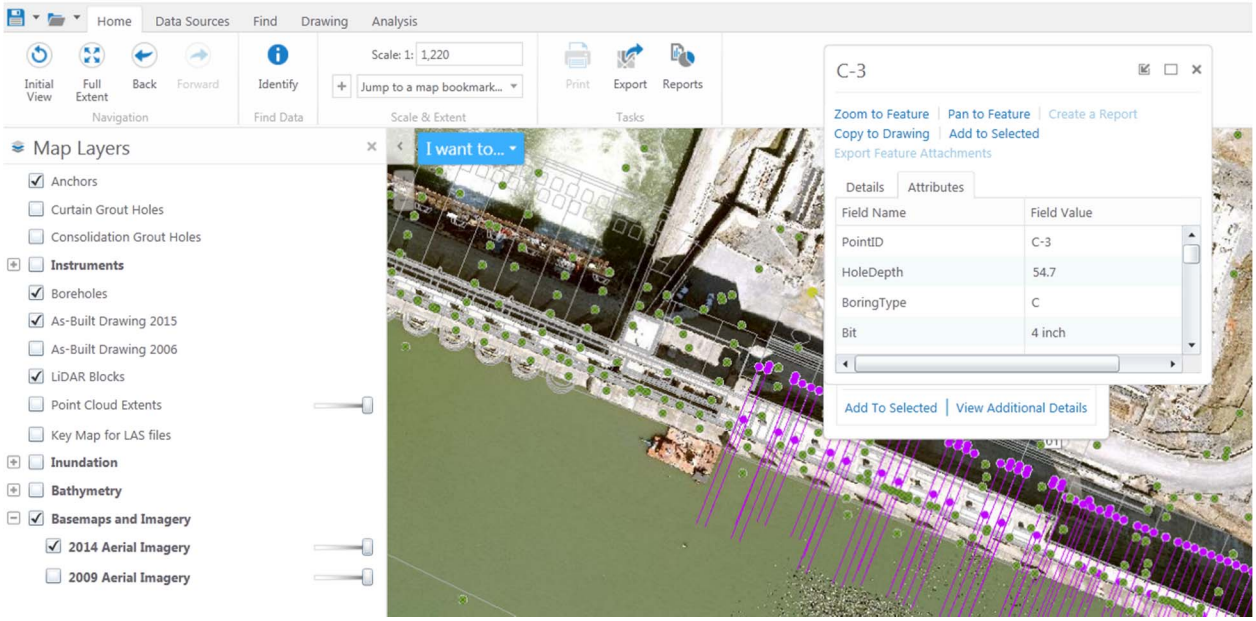


Fig. 9. Plan View Webmap showing aerial imagery, anchors, and data associated with selected borehole

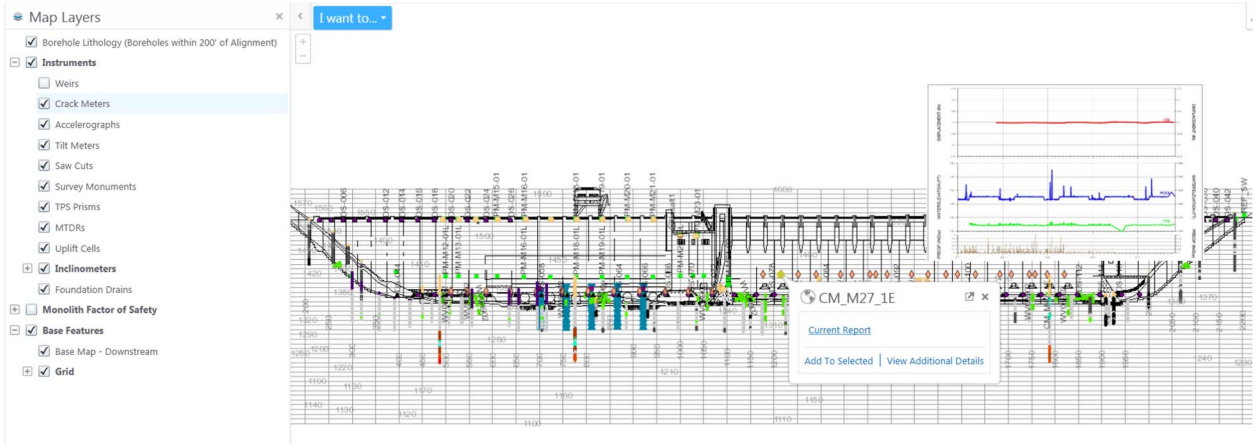


Fig. 10. Profile View Webmap showing hyperlink associated with selected crack meter

Cross-Sections

Although the 3D GIS and Microstation models are the appropriate tools to view the relative positions of structures, anchors, and boreholes in a true three-dimensional manner, in practice the use of 3D models on 2D computer screens is limiting. For example, when "clicking" an object in three dimensions to load associated attributes, one is actually clicking on array of points "in front" and "behind" the targeted object from the point of view of the user. Furthermore, it is difficult to make accurate measurements along a targeted plane. Finally, the USACE-preferred 3D GIS software has some limitations with respect to labeling and hyperlinking documents. To address these issues, we also prepared 2D cross-sections through the dam elements. Cross-sections were prepared at 110 pre-determined (by USACE) locations, including locations perpendicular to the dam axis (through each monolith), and parallel to the dam axis (through key features and spaced at appropriate intervals).

© 2016 Deep Foundations Institute

The purpose of the cross-sections is to allow access to a full set of GIS activities (symbology, labeling, hyperlinked documents, selections, etc.) for data along (or nearby) the cross-section. Thus a GIS application was developed that allows users to select a cross-section and presents them with a GIS project file that contains:

- The geotechnical cross-section created by USACE in Microstation;
- A cross-section cut through the dam structural features identified from LiDAR data and included in the Bluestone 3D Model;
- Anchors;
- Foundation Drains;
- Monolith Factor of Safety ratings; and
- Borehole geotechnical data, including:
    - Borehole positions with associated borehole logs
    - Lithology and soil classifications
    - Standard Penetration Test data
    - Core Loss data
    - Rock Quality Designations
    - Water Elevation
    - Water Content
    - Atterberg Limits

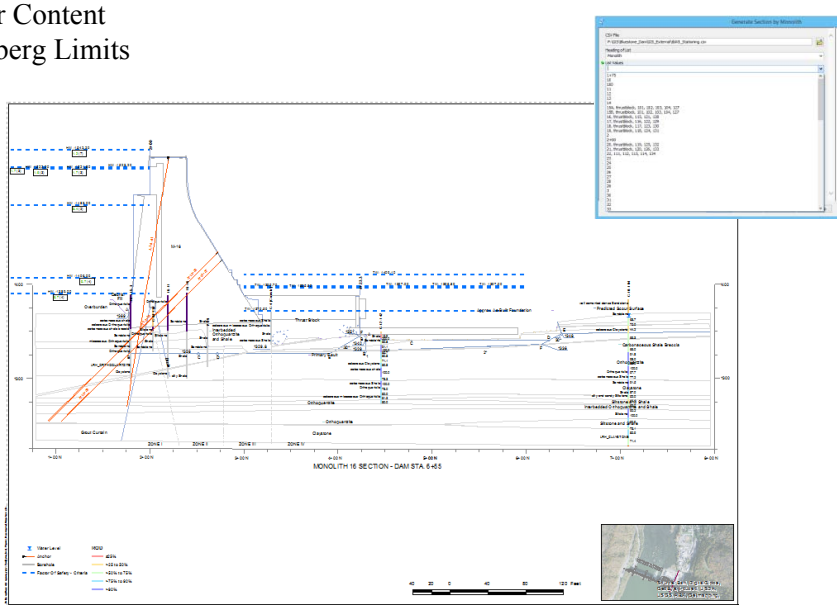


Fig. 11. Example Cross-Section Selection Tool showing dam structures, anchors, geotechnical borings, and various Monolith Factor of Safety visualizations

FUTURE USAGE

In the relatively near-future, the model will be used to verify that project construction meet design specifications and design intent. For example, the model will be used to verify that the alignment and depth of high-strength anchors are in a suitable geologic material for the required bonding zone.

Post-construction of the project modifications, the project will undergo an evaluation to determine if the project modifications result in a reduction of the overall project risk to within USACE tolerable risk guidelines and a lower DSAC.

© 2016 Deep Foundations Institute

If additional modification is required, the model will be used to aid in the future design work. For example, if additional anchors or subsurface instrumentation is required, the model will be used to aid in selecting appropriate locations such that drilling and installation will not intercept/coincide other key features and/or result in the malfunction or damage of that feature.

The model will also be used for both short–term and long-term monitoring of project performance and operations status. The project has multiple instruments installed for both construction and operational phases of the project. The BlueDMS will be used to evaluate instrumentation data and project performance during normal, high, and low pool events. The BlueDMS will be used to quickly identify the location of potentially concern data or data trends and aid in the evaluation of that data in both a structural and geologic context. In addition, operations personnel will be able to update the operations and maintenance status of the project. Activities or operational condition assessments of a project features that may affect the project performance will be readily identifiable. For example, if a gate is temporary inoperable due to maintenance needs, this fact can be tracked in the BlueDMS and that gate will be readily identifiable. This will aid operators, engineers, and emergency management personnel in making informed decisions on the operations of the project during various flood events.

Further, the USACE is currently developing a more global tool that will provide a Corps-wide status of all major dam facilities. The tool will provide senior management an overall view of the near "real time" project performance and operations status of various projects simultaneously. The BlueDMS will be included in the project portfolio and permit users with appropriate level access information about the project's historical performance, project modifications, DSAC, and operational condition.

CONCLUSIONS AND LESSONS LEARNED

Bluestone Dam is currently a DSAC II project with associated risk above USACE tolerable guideline. Project modifications are in various stages of design, construction, and completion to reduce the risk.

The BlueDMS is geospatial data management system developed by Geosyntec in collaboration with USACE to document the Bluestone Dam original construction and all associated project modifications. Project features including the dam monoliths, penstocks, stilling base, surrounding bathymetry and topography data are all included in their respective geospatial locations. Project details, such as concrete sample test locations and compressive strength test results, anchor installation locations and associated grout tests, anchor pull test, and bond zones are all be readily included in the database, and modeled in both 2D and 3D geospatial models. The 2D and 3D models will aid in quickly identifying various features, their attributes, and their spatial locations in both structural and geologic contexts. The database will aid in verify construction specifications were met by permitting querying of project feature data for "pass" and "fail" criteria. The GIS model with be used to view project features or test results that met or exceeded criteria with respect to geospatial environment, surrounding material matrix, feature geometries, and neighboring geology. This will ultimately facilitate thorough evaluations of the data and verification that the project modifications have reduced the overall risk associated with the project to within tolerable rick guidelines.

The BlueDMS will serve as a "living" model to monitor project performance and operations, design engineers, and emergency management in future decisions regarding the future operation of the project. The system will ultimately tie into a more global monitoring tool that will allow USACE to monitor multiple projects simultaneously. Being able to quickly identify the performance and operations status of multiple projects simultaneously will permit the USACE to make informed decisions concerning operations and emergency management.

© 2016 Deep Foundations Institute

EVALUATION OF SOIL-BENTONITE BACKFILL CONSOLIDATION PROPERTIES

Jeffrey C. Evans, Bucknell University, Lewisburg, PA USA 570-577-1371 evans@bucknell.edu
Hejintao Huang, Bucknell University, Lewisburg, PA USA 570-577-1112, hh012@bucknell.edu
Daniel G. Ruffing, Geo-Solutions, New Kensington, PA, 724-335-7273, druffing@geo-solutions.com

ABSTRACT

Soil-bentonite slurry trench cutoff walls are the most common type of vertical cutoff wall used in the US for seepage control and for passive control of contaminant migration at environmental sites. Design considerations are often limited to constructability considerations and a target hydraulic conductivity. With design-build contracts common, the backfill design is often left to the contractor. As a barrier soil, the hydraulic conductivity of bentonite slurry wall backfill is typically required to be equal to or below $1x10^{-7}$cm/s. The importance of stress state upon the hydraulic conductivity as well as the expected stresses are both established in the literature. Grain size distribution of the base soil and bentonite content both affect compressibility and hydraulic conductivity. This paper describes the results of studies undertaken to evaluate the interrelationships between compressibility, hydraulic conductivity, grain size distribution, and bentonite content. Three base soils were used representing a range of soils found on typical construction projects. Using these base soils, mixtures containing a range of dry bentonite contents from 0% to 5% were prepared to a slump consistent with field practice. The compressibility and hydraulic conductivity were measured in consolidometers adapted for falling head permeability tests at the end of each load increment. These experiments yielded the relationship between effective stress and hydraulic conductivity as well as consolidation characteristics including time rate of consolidation and coefficients of compressibility. The experimental results are then used to indirectly evaluate stress development and to assess the time needed for consolidation. The findings show a substantial decrease in hydraulic conductivity as effective stress increases, regardless of base soil type and dry bentonite content, a relative insensitivity of stress development to backfill compressibility, and a wide range of values for coefficient of consolidation depending upon backfill mixture and stress level.

KEYWORDS

slurry wall, bentonite, cutoff wall, slurry trench

INTRODUCTION

Soil-bentonite (SB) slurry trench cutoff walls have long been used to control seepage through dams and levees and into excavations beneath the water table (Evans 1993). For SB cutoff walls, a trench is first excavated under a head of bentonite-water slurry (the slurry trench). The slurry is then replaced with the soil-bentonite backfill comprising soils, usually those excavated from the trench, blended with bentonite-water slurry and, if needed, additional dry bentonite. Calculations for trench stability are common, but SB backfill "design" has historically been largely empirical. For seepage control applications, the principle performance criterion has been hydraulic conductivity. Depending upon project needs, upper limits of $1x10^{-5}$ to $1x10^{-7}$ cm/s are commonly specified. To achieve the necessary hydraulic conductivity, designer and contractor experience has been employed to decide upon bentonite content to great success. Often, preconstruction laboratory studies of several mixtures are conducted to validate the contractor and/or designer experience.

Studies have shown a strong correlation between stress state and hydraulic conductivity (Ruffing and Evans 2010, Evans and Huang 2016) which makes understanding the effects of soil-mixture variations upon stress state important. The results of laboratory studies are presented to show how the compressibility, time rate of consolidation and hydraulic conductivity vary with effective stress and backfill composition. The laboratory results are then used with a closed form model of stress in SB cutoff walls to predict the range of stresses expected over a range of backfill mixtures.

© 2016 Deep Foundations Institute

PREVIOUS STUDIES

Given the soft, compressible nature of SB backfill and the dependence of hydraulic conductivity on void ratio, it is no surprise that the hydraulic conductivity is highly stress dependent. Previous studies, e.g. Evans 1994, Filz et al. 2001, Yeo et al. 2005, and Ruffing and Evans 2010, present the stress dependency of SB backfill. The data plotted in Fig. 1 reinforces this known relationship.

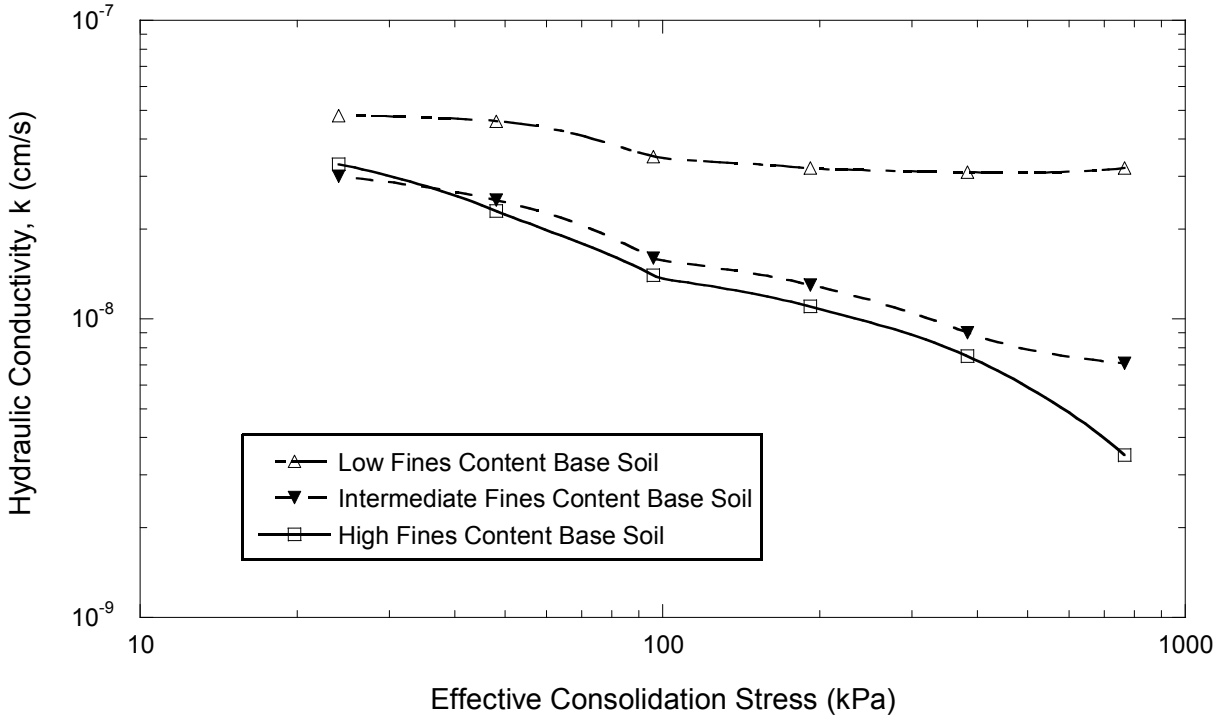


Fig. 1. Effective stress vs. *k* for SB backfill (plotted from data in Evans and Huang 2016)

The primary question is how do the grain size distribution, specifically the fines content, and the bentonite content impact the hydraulic conductivity? The current study investigated this relationship using model SB backfills created from three base soils with varying fines contents and a varying bentonite content (Evans and Huang 2016). The grain size distributions of the three base soils are presented in Fig. 2 against a recommended grain size distribution for SB base soils from an earlier design guidance document (Lagrega, et al. 1996). For the purposes of the discussions herein, the soils are identified as high, intermediate, and low fines content soils with fines contents of 77%, 42% and 6% respectively. Notice that only the intermediate fines content soil falls within the recommended fines content and all soils lack sufficient coarse materials to meet the recommended gradation.

© 2016 Deep Foundations Institute

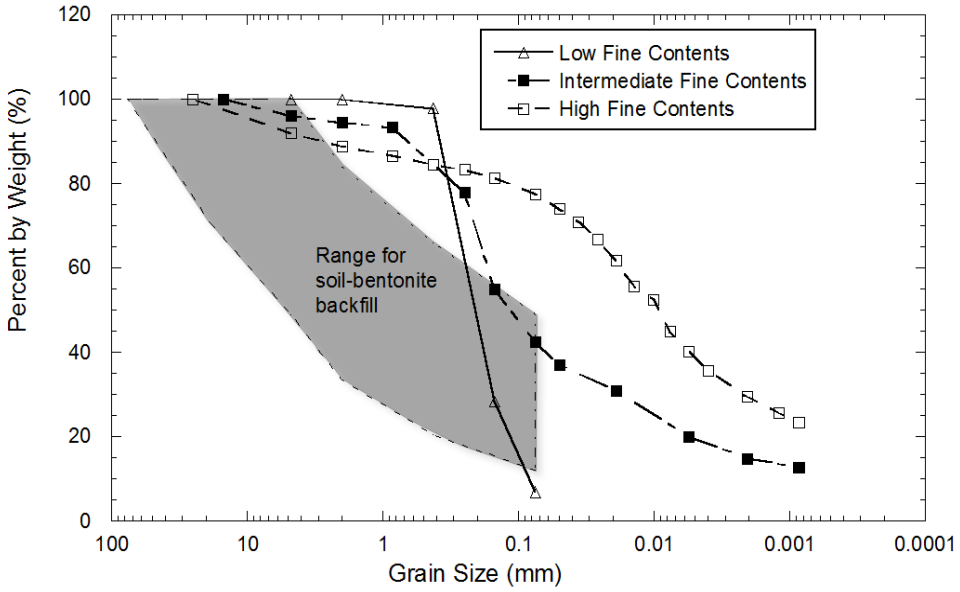


Fig. 2. Grain Size of SB Soils and Recommended Range (from Evans and Huang 2016)

The hydraulic conductivity measured at one effective stress (96 kPa) is presented in Fig. 3 as a function of base soil fines content (6% to 77%) and bentonite content (0% to 5%). Two findings of note are clear from a review of Fig. 3. First, increasing the natural fines content results in a decrease in the hydraulic conductivity of about one-half an order of magnitude for the mixtures containing dry bentonite. The hydraulic conductivity reduction, more than an order of magnitude, is greater for the mixture with no dry bentonite (slurry only). Second, the addition of dry bentonite has a significant effect as the dry bentonite content goes from 0% to 5%.

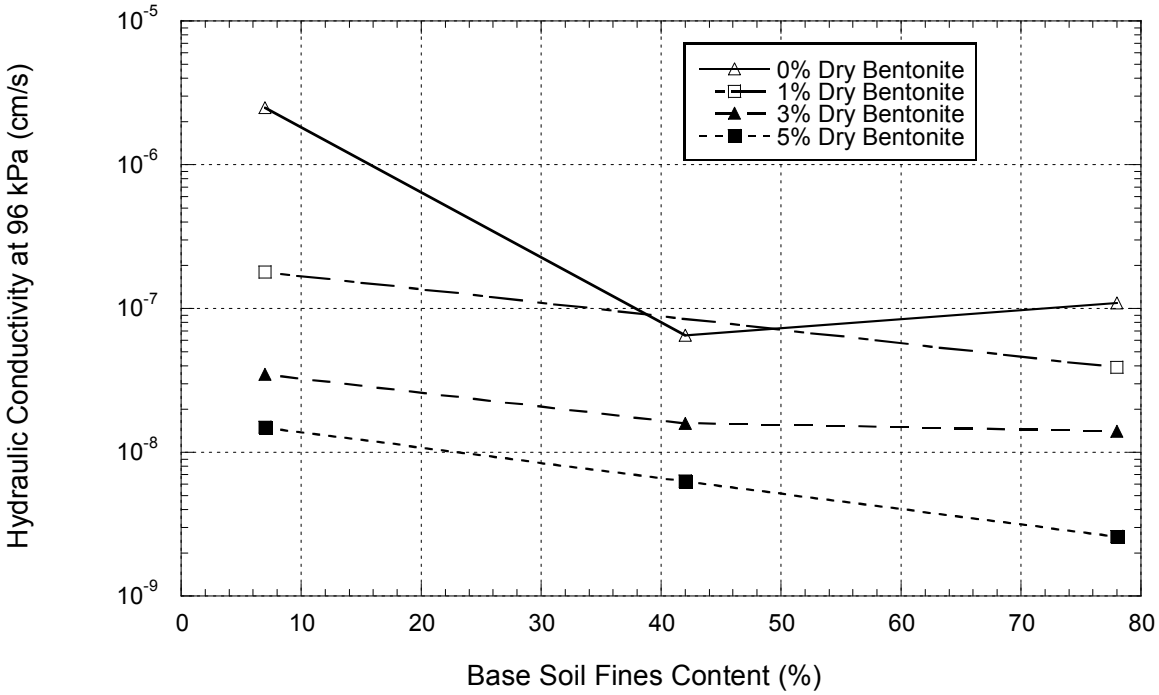


Fig. 3. Base soil fines and dry bentonite vs. *k* of SB (from Evans and Huang 2016)

© 2016 Deep Foundations Institute

LABORATORY INVESTIGATIONS AND DISCUSSION OF RESULTS

Bentonite-water slurry was prepared using 5% bentonite and 95% tap water, by mass. Tap water properties included a temperature of 25 ˚C, pH value of 7.75, and conductivity of 170 µs/cm. The average viscosity of the slurry after 24 hours of hydration was 40.7s although the slurry viscosity decreased to 35.4s after remixing in the high-speed colloidal shear mixer for 5 minutes demonstrating the thixotropic behavior of bentonite slurry.

After a minimum of 24 hours of hydration, the slurry was mixed with the three base soils (low, medium, and high fines content). Dry bentonite was added at rates of 0% (slurry only), 1%, 3% and 5% dry bentonite resulting in a total of twelve backfill mixtures. The base soils started at their natural moisture contents of 8%, 16%, and 24% for the low, medium and high fines content soils, respectively. Slump tests were conducted after mixing the backfill in order to ensure the slump was in the appropriate design range of 100 mm to 150 mm (4-6 in) using a laboratory scale slump cone (Malusis et al, 2008). The moisture content for each backfill mixture was measured. The prepared backfill mixtures were stored in sealed containers for a minimum of two days prior to testing to allow for equilibration of moisture throughout the sample.

Fixed-ring consolidometer/permeability cells with lever-arm loading were used for falling-head permeability tests at varying states of effective stress and to determine time rate of consolidation and one-dimensional stress-strain parameters (ASTM D2435-04 and Yeo et al. 2005). The hydraulic conductivity results are reported in Evans and Huang 2016.

LABORATORY TEST RESULTS

The time rate of consolidation data was interpreted using Taylor's T_{90} method and the resulting coefficient of consolidation, c_v, values for the twelve backfill mixtures are shown on Figs. 4, 5 and 6. These data show a range of c_v, values from $4x10^{-5}$ to $7x10^{-8}$ m^2/s for the range of fine contents, bentonite contents, and stress shown indicating a nearly three order of magnitude difference in time to consolidation of the SB backfill. The impact on consolidation time for representative SB cutoff walls is explored in the analysis section.

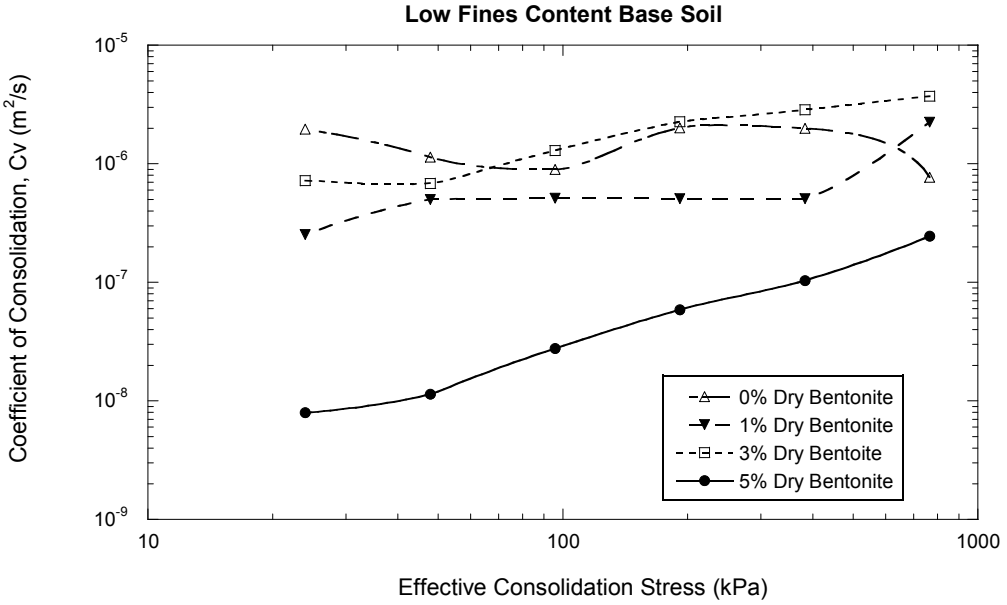


Fig. 4. Coefficient of consolidation for low fines content backfills

© 2016 Deep Foundations Institute

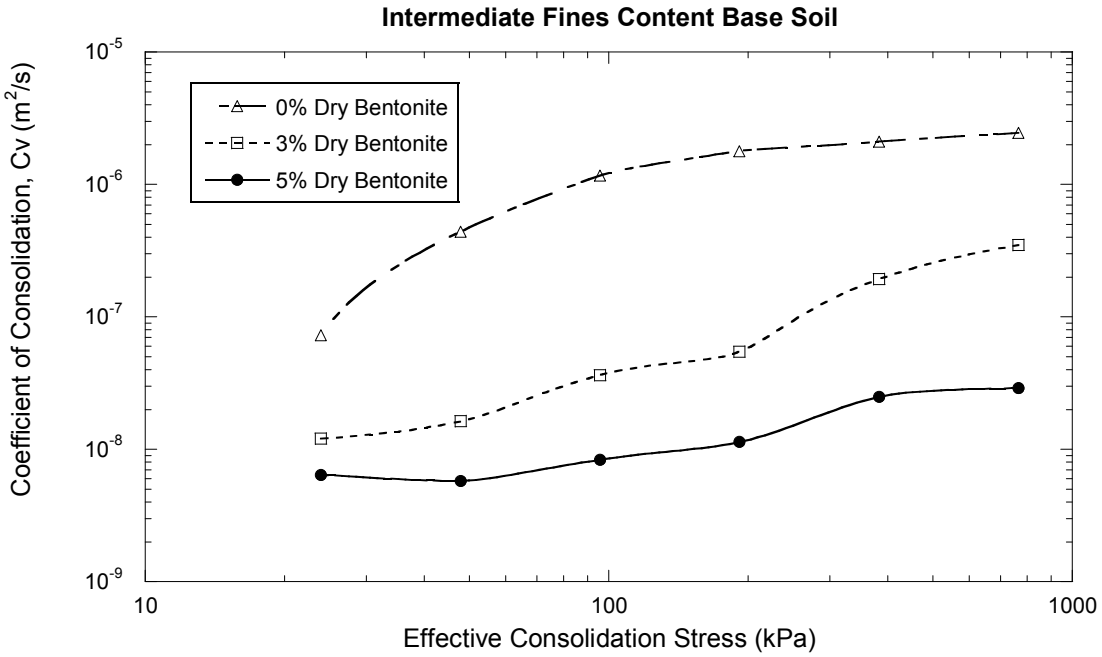


Fig. 5. Coefficient of consolidation for intermediate fines content backfills

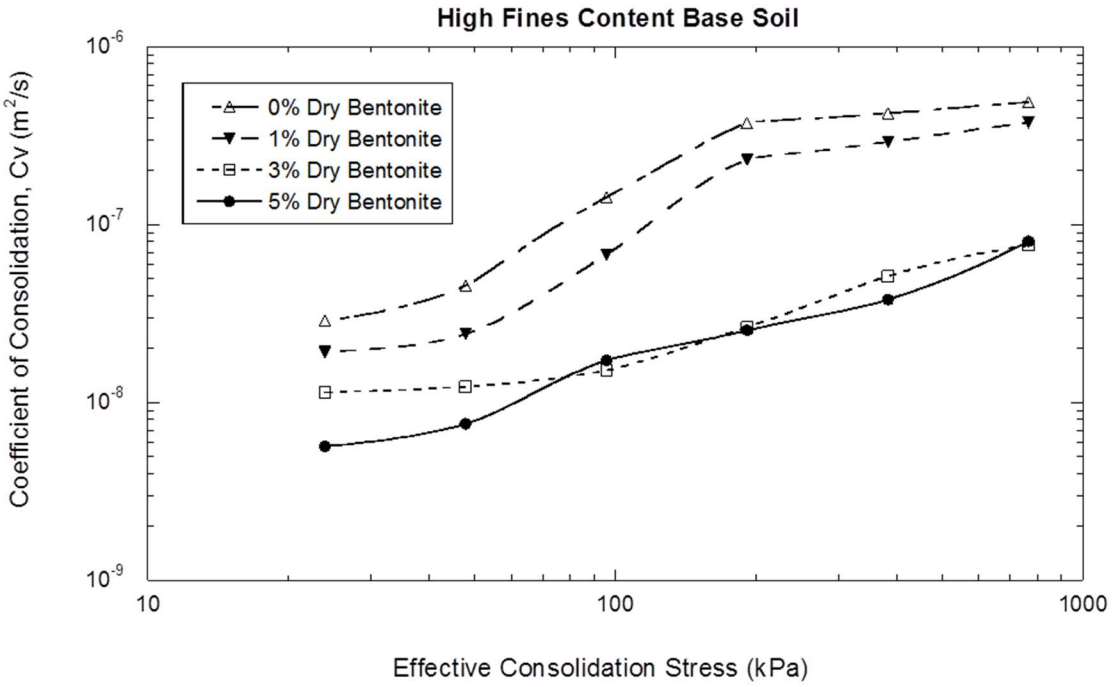


Fig. 6. Coefficient of consolidation for high fines content backfills

The relationship between void ratio and effective consolidation pressure (e log σ_v' curves) for loading and unloading was also evaluated from the consolidation tests. As the material is

© 2016 Deep Foundations Institute

completely remolded, these plots can be used to determine the modified compression index, C_{c_ε}, for loading and the modified swell index, C_{s_ε}, for unloading. The modified compression index is shown in Fig. 7 as a function of base soil fines content and bentonite content. Fig. 8 presents a similar data set with the only change being that the dependent variable is the modified swell index.

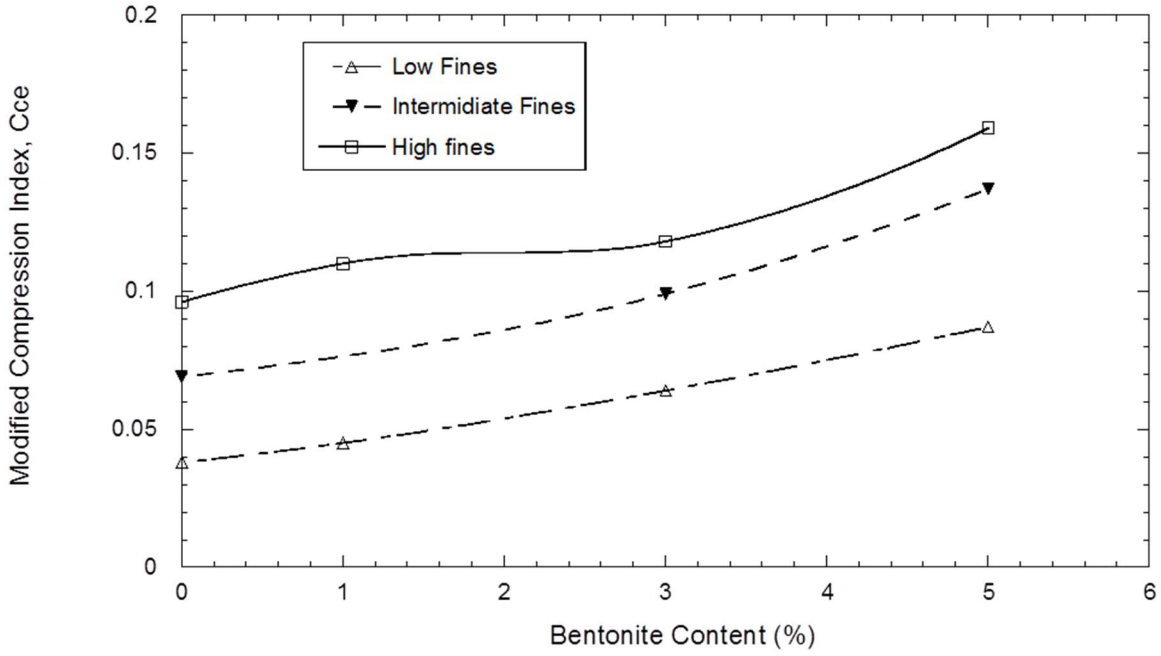


Fig. 7. Modified compression index for backfills

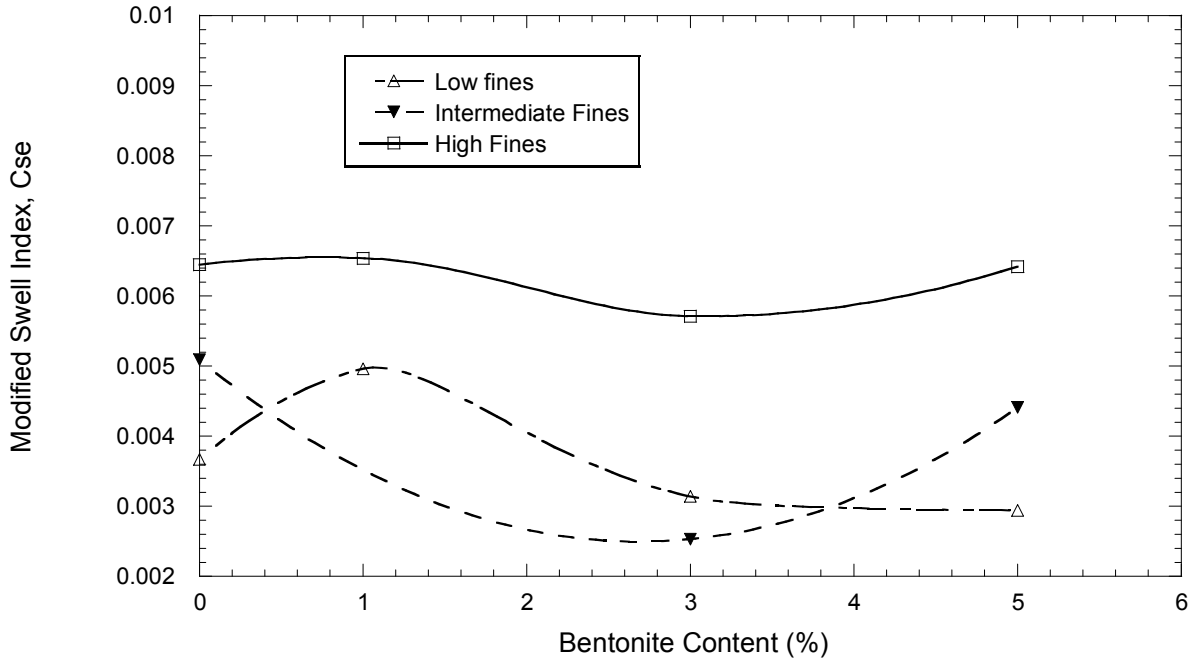


Fig. 8. Modified swell (recompression) index for backfills

As shown in Fig. 7, C_{c_ε} is sensitive to both the fines content and the bentonite content. Increasing the fines content or the bentonite content results in increasing compressibility.

© 2016 Deep Foundations Institute

ANALYTICAL INVESTIGATIONS

The hydraulic conductivity is heavily stress dependent (Figs. 1 and 3) and the stresses in the backfill are believed to depend upon the compressibility characteristics of the backfill (Filz 1996). The modified lateral squeezing model (MLSM) uses the compressibility of the backfill and the movement of the material adjacent the trench to balance the horizontal forces at the trench sidewall (Ruffing et al. 2010). The governing closed-form solution to this force system is shown as Eq. 1 (Eq. 8 in Ruffing et al. 2010).

$$\sigma'_{ho} = \gamma'_o z k_{am} = 10^{\left(\frac{2\Delta - BC_1}{BC_{c\varepsilon}}\right)} \qquad [1]$$

Fig. 9 shows input data and stress prediction results from an earlier study in which the modified compression index, $C_{c\varepsilon}$, was determined from one particular SB backfill.

Parameter	Units	Value
Lateral Squeezing (LS) Model		
γ'_o	kN/m^3	11.2
D_b (min)[1]	kPa	500
D_b (max)[1]	kPa	1600
ϕ'_o	degrees	35
Arching Model and Geostatic		
γ'_b	kN/m^3	9.7
c_b	kPa	0
ϕ'_b[2]	degrees	30
k_{ob}	---	0.5
Modified Lateral Squeezing (MLS) Model		
γ'_o	kN/m^3	11.2
$C_{c\varepsilon}$[1]	---	0.10
C_1[1]	---	-0.09
ϕ'_o	degrees	35

[1] values based on data in Fig. 5.
[2] from Baxter et al. (2005)

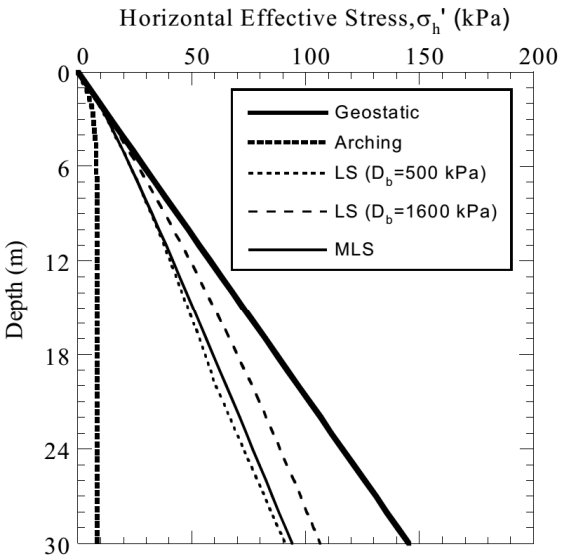


Fig. 9. Horizontal effective stress distributions in SB cutoff wall (B = 1 m) predicted by arching, lateral squeezing, and geostatic methods (from Fig. 6. in Ruffing et al. 2010)

Using the data presented on the table embedded in Fig. 9 and the compressibility information collected in this study allows for a study of lateral earth pressures over a range of base backfill soil types and bentonite contents. Equation 1 (the governing equation of the MLSM) was used along with the data presented in the embedded table in Fig. 9 to calculate effective stress across a range modified compression indices. The results of these analyses are shown on Fig. 10.

© 2016 Deep Foundations Institute

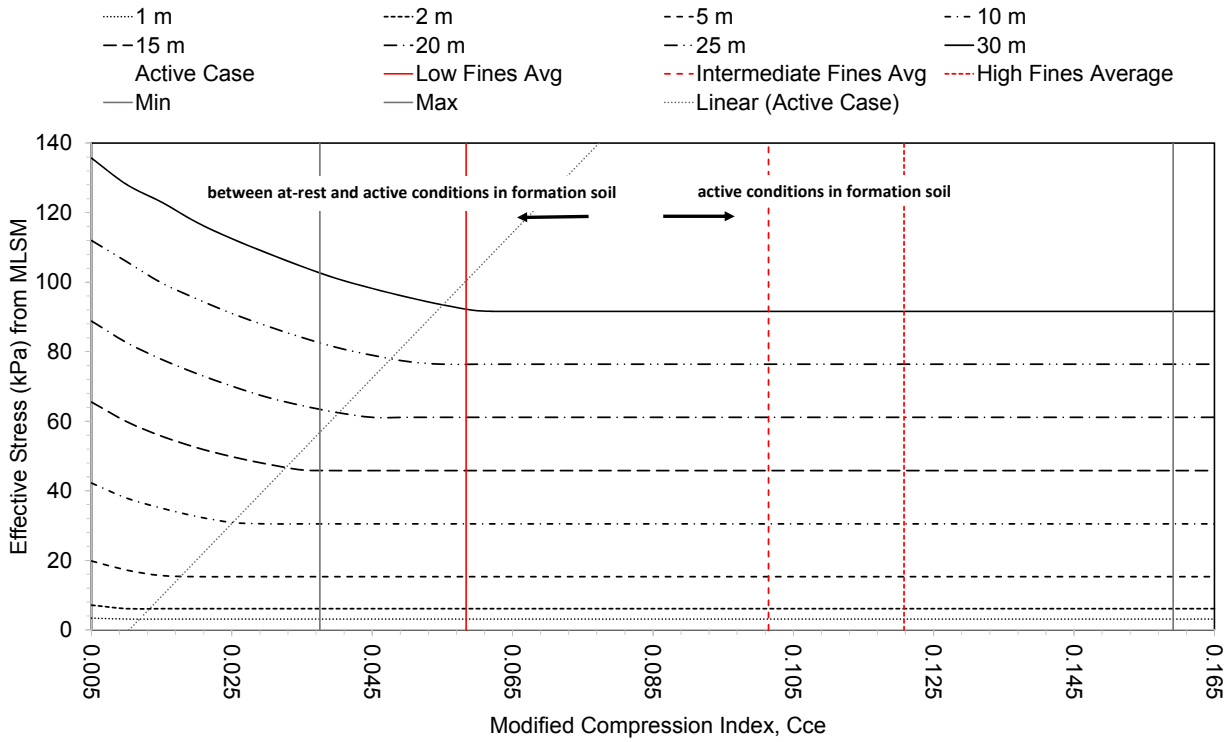


FIG. 10. Horizontal stress as a function of modified compression index and depth for a theoretical medium dense sand (Φ=35°) formation soil

Several interesting findings are clear from the information in Fig 10. First, the backfill consolidation behavior, represented by the modified compression index, only influences the state of stress prior to the active condition in the materials adjacent the trench. The diagonal dotted line shows that as the stiffness of the backfill increases, the depth required to fully mobilize active earth pressure increases. Next, above the depth where active earth pressure controls the horizontal stresses, it is found that stiffer backfills, i.e. lower modified compression indices, result in greater stresses. Finally, other than at the extremes of very low modified compression index (very stiff backfill) and deep cutoff walls, the stiffness of the backfill does not significantly influence the stress state. Notice that throughout these analyses, the controlling factor was largely the active earth pressures from the adjacent formation soils.

It is also useful to consider the effect of fines and bentonite content upon the time rate of consolidation. To do this, a wall width of 1.0 m was assumed consistent with the assumption of wall width for stress calculations. One-dimensional, double-drainage conditions were assumed with all flow laterally towards the sidewalls of the trench. Importantly, and perhaps controversially, the impact of the filter cake was ignored in these calculations. While the role of the filter cake after backfill is placed is beyond the scope of this paper, ignoring the role of the filter cake considered three factors. First, the purpose of the calculations is to examine the effect of fines and bentonite content upon the time rate of consolidation. Next, some have argued that the movement of the backfill, clearly present in the slurry filled trench, largely destroys the filter cake during backfill placement. Finally, the hydraulic conductivity of the filter cake may not be largely different than the hydraulic conductivity of the backfill rendering its inclusion in the calculations of no consequence (Ruffing et al. 2016). Note that the c_v values are stress dependent (Figs 4, 5 and 6). For this parametric study, an average c_v value was used over a stress range of 25 to 100 kPa representative of the range of stresses expected (see Fig. 10). For the purposes of studying the effect of the coefficient of consolidation, the average time for 50% and 90% consolidation was computed. The results of these computations are shown on Table 2.

© 2016 Deep Foundations Institute

Table 2 Time for 50% and 90% consolidation of SB backfill

Backfill Fines	Bentonite Content (%)	Coefficient of Consolidation (m^2/s)	Time for 50% consolidation (days)	Time for 90% Consolidation (days)
Low	0	9.6E-07	0.6	2.6
Low	1	4.2E-07	1.3	5.8
Low	3	9.0E-07	0.6	2.7
Low	5	1.6E-08	36	156
Intermediate	0	5.6E-07	1.0	4.4
Intermediate	3	2.2E-08	26	113
Intermediate	5	6.9E-09	83	358
High	0	7.2E-08	7.9	34
High	1	3.7E-08	15	66
High	3	1.3E-08	44	190
High	5	1.0E-08	56	241

An examination of the data on Table 2 reveals the fines and bentonite content can have a large effect upon the time rate of consolidation. For example, the time for 50% consolidation can vary from less than one day to fifty-six days, depending upon the backfill mixture. The trend is illustrated in Fig. 8 showing that as the bentonite content increases, the time for consolidation increases. Figure 8 also illustrates that as the fines content increases, the time to consolidation also increases. The data at 5% bentonite for the medium and high fines content base soil shows the high fines content consolidates a bit quicker than the medium fines content. This result could be real due to differences in void ratio between the two mixtures or due to experimental error as the difference in C_v values is small.

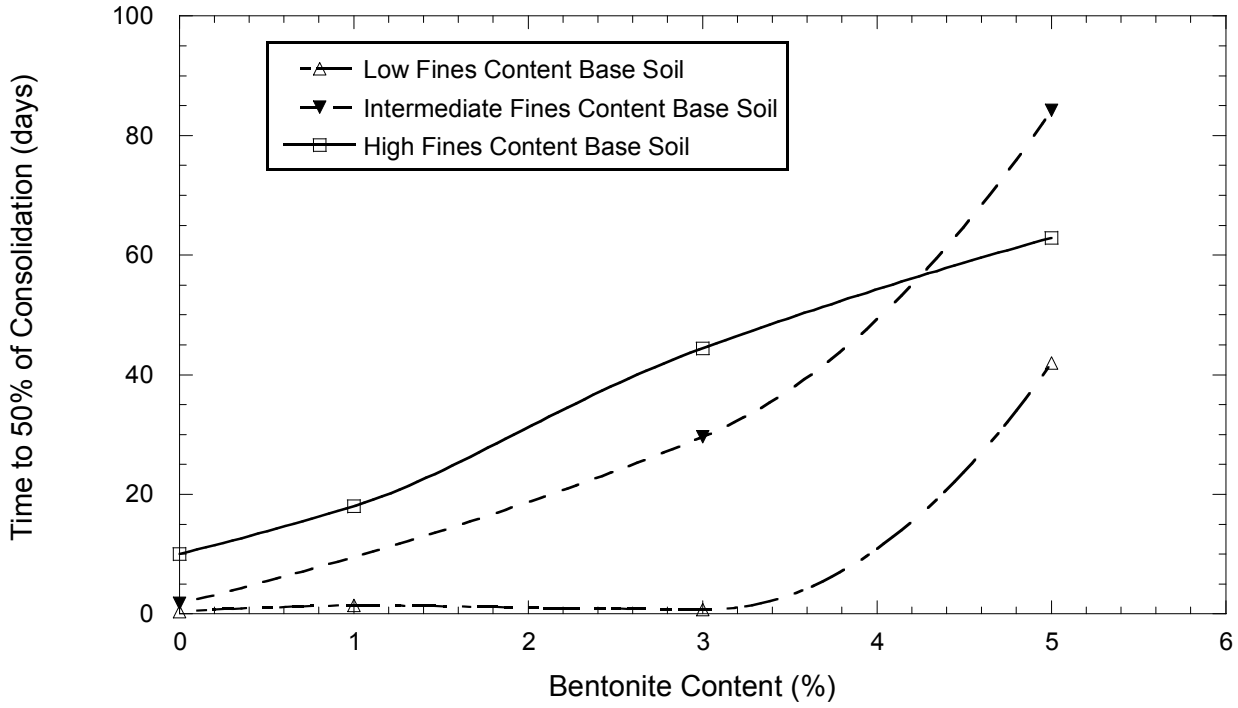


FIG. 11. Time to 50% consolidation

SUMMARY AND CONCLUSIONS

A series of consolidation tests were conducted on SB backfill mixtures created using three base soils with low, medium, and high fines content and varying dry bentonite contents from 0% to 5%. The data from

© 2016 Deep Foundations Institute

these tests were used to determine the coefficient of consolidation for time rate of settlement calculations and modified compression indices for stress calculations. The analysis shows that the state of stress is largely controlled by the active earth pressure developed in the adjacent formation soils as a result of inward movement of the trench sidewalls resisted by the SB backfill. Even over a wide range of backfill stiffness, the stiffness shows little influence on the stress state.

The backfill fines content and bentonite content did, however, have a large influence on the time needed to consolidate the backfill. For all backfills tested at dry bentonite contents of 0% and 1%, the time for 50 percent consolidation was less than two weeks but additional bentonite added to the mix increased that time to as much as three months.

ACKNOWLEDGEMENTS

The authors would like to thank Bucknell University for their support of Hejintao Huang during her work on this project. The authors would like to thank Bucknell University student Melissa Replogle and Nancy Ingabire Abayo for their assistance with some of the laboratory testing. James Gutelius Bucknell University's Director of Laboratories for the Department of Civil and Environmental Engineering provided the fine-grained soil used in the experiments. The support of Hejintao Huang by Bucknell University's Program for undergraduate Research is gratefully acknowledged.

REFERENCES

ASTM D2435-04 (2004) "Standard test methods for one-dimensional consolidation properties of soils using incremental loading, ASTM International, West Conshohocken, PA.

Evans, J.C., (1993) "Vertical Cutoff Walls" Chapter 17 in Geotechnical Practice for Waste Disposal, Ed. Daniel, Chapman, and Hall.

Evans, J.C., (1994) "Hydraulic conductivity of vertical cutoff walls" *Hydraulic Conductivity and Waste Contaminant Transport*, ASTM STP 1142, pp. 79-94.

Evans, J.C., and Huang, H., (2016) "Hydraulic Conductivity of Soil Bentonite Slurry Walls", *Proceedings of ASCE GeoChicago 2016 Sustainability, Energy, and the Geoenvironment*, upcoming in Aug 2016.

Filz, G.M., (1996) "Consolidation stresses in soil-bentonite back-filled trenches.", *Proceedings of the 2nd International Congress on Environmental Geotechnics*, M. Kamon, Ed. Osaka, Japan, pp. 497-502.

Filz et al. (2001) "Determining hydraulic conductivity of soil-bentonite using the API filter press" *ASTM Geotechnical Testing Journal* v. 24, no. 1, pp. 61-71.

LaGrega, M.L, Buckingham, P.L., and Evans J.C., (1996) Hazardous Waste Management, McGraw-Hill Book Company, New York, NY, 1996, 2nd Ed. (2001) reissued by Waveland Press, Inc. (2010).

Ruffing, D.G., Evans, J.C., and Malusis, M.A., (2010) "Prediction of Earth Pressures in Soil-Bentonite Cutoff Walls", *Proceedings of ASCE GeoFlorida 2010 Advances in Analysis, Modeling, and Design GSP 199*, pp. 2416-2425.

Ruffing, D.G., and Evans, J.C., (2010) "In Situ Evaluation of a Shallow Soil Bentonite Slurry Trench Cutoff Wall" *Proceedings of the 6th International Congress on Environment Geotechnics*, New Delhi, India, Tata McGraw-Hill ISBN 13:9780070707108, pp. 758-763.

Ruffing, D.G., Evans, J.C., Spillane, V.A., and Malusis, M.A., (2016) "The Use of Filter Press Tests in Soil-Bentonite Slurry Trench Construction", *Proceedings of ASCE GeoChicago 2016 Sustainability, Energy, and the Geoenvironment*, upcoming in Aug 2016.

Yeo, S.S., Shackelford, C.D., and Evans, J.C., (2005) "Consolidation and Hydraulic Conductivity of Nine Soil-Bentonite Backfills" *J. Geotech Geoenv. Eng.* 131 (10), pp. 1189-1198.

© 2016 Deep Foundations Institute

BENTONITE SELF-HARDENING SLURRY MIX VARIATIONS AND FIELD IMPLEMENTATION METHODS

Christopher R. Harris, Slurry Engineering, Inc., 1721 Eastern Ave. Ste. 15, Sacramento, CA USA
chris@slurryengineering.com; (916) 798-4176

ABSTRACT

Considerations are presented for developing and implementing self-hardening slurry mix designs to be utilized in barrier seepage walls. Slag-cement-cement-bentonite (SCCB) slurries are the primary focus, with some comparisons referenced to the more traditional Portland-based cement-bentonite (CB) slurries. Field and laboratory mixing methods are reviewed, including 1) single sack – single mixer method; 2) bentonite slurry mixer – cementitious addition mixer method; 3) bentonite mixer – grout mixer – CB blending mixer method. Major phenomena associated with the dynamics of barrier wall excavation, such as loss of water from the in-trench slurry and consolidation of in-trench slurry are discussed. Other topics touched upon include sand content effects, fluidizers utilized, and chemical contaminants. The role of higher bentonite to water ratio is reviewed, with demonstration of improvement of filtration properties, and an increase of water retention of the final consolidated product.

Keywords: self-hardening, slurry, slag, GBFS, SCCB, bentonite, mixing, filtration, consolidation

INTRODUCTION

Self-hardening slurry has been utilized to construct barrier seepage walls for many decades. This is essentially a single-step slurry excavation and backfill process whereby the trench-supporting fluid gains strength over a short period during installation and maintains or develops low hydraulic conductivity. The resulting curing slurry becomes the actual barrier wall as the resultant cement-augmented clay slurry now fulfills the dual purpose of both temporary trench stabilization and final cured-in-place product. This eliminates the requirement and expense of subsequent backfill/slurry displacement operations required in traditional barrier wall installations.

Self-hardening slurries have been fabricated using a number of various clays, most commonly sodium bentonite, calcium bentonite, attapulgite, and sepiolite. Perhaps due to plentiful high quality sodium bentonite deposits found domestically, the proven excavation stabilization properties imparted from sodium bentonite slurries, sodium bentonites are typically specified for formulating self-hardening slurries in the US. Cementitious materials utilized most commonly are Portland cement and slag (ground blast furnace slag or GBFS). For the purposes of this discussion, self-hardening slurry having a cementitious of Portland will be referred to as CB. Those having cementitious material of GBFS replacing Portland (partially or wholly) will be referred to as SCCB.

OVERVIEW OF MIXING METHODS

Method 1: One Sack – One Mixer Method

The self-hardening slurry SCCB, for example, is formulated into a single blend consisting of bentonite, GBFS, and Portland cement. The mixing is straightforward as the formulation would call for a certain quantity of water per sack of the dry product. The SCCB can be easily batched in a single or double batch mixer, and therefore it is easy to implement in the field. Alternatively, this may be accomplished without custom packaging by adding ingredients proportionally from normally available individual product packaging, and in a certain order; e.g. soda ash to water, then bentonite to water, then GBFS to bentonite slurry, then Portland. As per Caenn (2016) soluble calcium from the cements inhibits potential hydration of sodium bentonite. The detrimental effects of initially mixing the bentonite in a cementitious environment are mitigated by a low dosage of soda ash (sodium carbonate) added the blend. The very soluble sodium carbonate precipitates the slightly soluble lime-calcium and increases the pH, thus suppressing water hardness for the moment. This “chemical trick” allows some degree of hydration of the bentonite to proceed without the influence of detrimental amounts of solubilized calcium. The end result

© 2016 Deep Foundations Institute

is enough hydration gained in this relatively short time period to achieve a thixotropic slurry having set and cure properties of adequate strength and low permeability.

Method 2: Mix/ Hydrate Bentonite in First Mixer/Storage - Add Dry Powder Cement in Second Mixer

In a dedicated bentonite mixing and storage system, mix and hydrate the bentonite slurry with low-hardness fresh water. In a second SCCB mixing system, add the dry cementitious products to hydrated bentonite slurry transferred from the mixer and hydration tanks or pond. This method is commonly used and enables a longer time period for hydration of the bentonite prior to mixing with the cementitious materials. This is more equipment intensive, and there is more to keep track of, but it offers the flexibility desired to make field adjustments. It is more cost effective over Method 1 from a material standpoint, since it does not require custom dry blending at a facility which may be limited in availability. This method will tend to be more effective on a pound for pound basis than the "single sack – single mixer" method. This is the case with both the bentonite (better hydration equals less needed) and the cementitious (soda ash may somewhat compromise the cementitious to a degree).

Method 3: Mix/Hydrate Bentonite in First Mixer - Mix/Hydrate Cementitious in Second Mixer - Blend Both Slurries Together in a Third Mixer

As in Method 2, the bentonite is hydrated in a completely separate system. The method provides for the additional step of pre-mixing the cementitious materials with water in a dedicated grout-type mixer. This allows for better mixing and dispersion of the cementitious materials in water. Mixing dry cementitious into bentonite slurry as in Method 2 may be difficult due to flocculation thickening. Volumetric proportioning and blending into the third mixer of the two slurries may be performed with the aid of flow meters or weigh systems. With this method, equipment configurations become more complex and expensive, but this method may provide for the most consistently superior final product. Additional steps are required for quality control since there are now three slurries to contend with. This method also has the potential for the highest output, potentially reducing the cost of the project. Experiments were performed testing a mix design composition by using both Method 3 and Method 2.

SLAG VS PORTLANT CEMENT – WHICH IS BETTER?

In the past twenty years, the use of SCCB over CB has become more popular and accepted over CB due to the lower hydraulic conductivity values and higher strengths of GBFS as a replacement for Portland cement. The "green benefits" of using a waste product in place of an energy-intensive manufactured product are also touted. GBFS may also be more economical on a price per ton basis, and is more effective at lower dosage rates than Portland cement. Additionally, GBFS has less of a disruptive reaction to the viscosity and filtration properties of bentonite slurry than does Portland cement. In other words, GBFS appears to be more compatible as an additive to bentonite slurry than does Portland cement, and produces a superior end-product at a more economical price. Nonetheless, there are many situations in which CB self-hardening would be chosen over SCCB. Among these possible reasons may be 1: Availability of Portland cement over GBFS in certain geographical areas 2: Requirement for high early strengths as CB tends to be faster setting given sufficient dosage rates. 3: Groundwater or geochemistry may not be beneficial to GBFS curing 4: Environmental objections may be raised over the presence of certain heavy metals in GBFS

Figure 1 illustrates examples of data derived from the author's investigative lab work. The chart displays unconfined compressive strength (UCS) typically attained with SCCB versus CB when considering the primary factor relating to strength development, the water to cementitious ratio (W:C). It is easy to note that there is a substantial difference in the UCS development of these two similar mixtures. This difference is up to 20 to 30 times greater for SCCB over CB. Furthermore, hydraulic conductivity results, derived from the author's investigations testing of SCCB and CB, are often one hundred times lower than those of CB. It takes no stretch of imagination to see why the popularity of SCCB over CB is on the ascent.

© 2016 Deep Foundations Institute

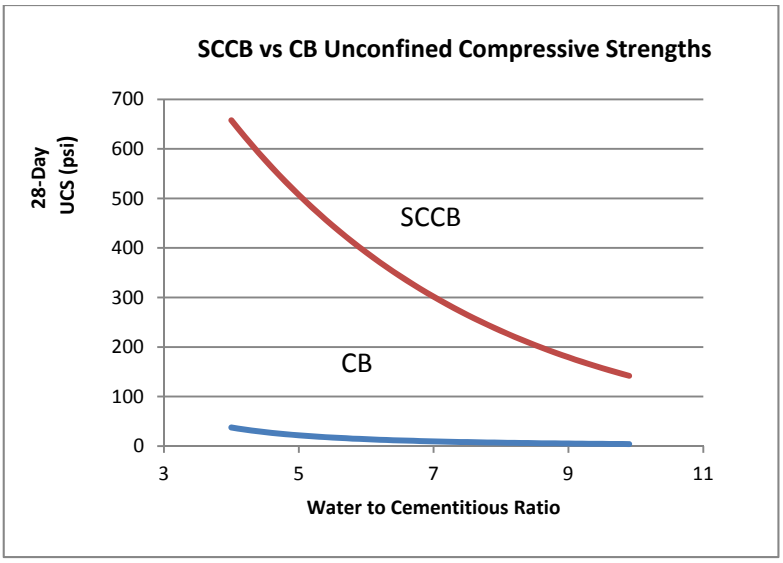


Fig. 1 Comparison of SCCB With CB Over Typical W:C Range (data trend lines derived from author's files)

TRENCH DYNAMICS

According to Carreto et al. (2016), Jefferis (2012) and Opdyke (2005) self-hardening slurry undergoes many simultaneous phenomena or mechanisms when placed into an excavation. Among these are contamination with excavated sediments and associated groundwater; subsequent sedimentation of these same soil contaminants with the virgin slurry solids in suspension; whole loss of slurry into highly porous cobble strata; filtration loss of the water phase to adjacent soil strata; consolidation of the slurry with subsequent water migration to the surface. The common thread is that they all effect volume changes of the slurry in the trench, and furthermore, they appear to the author to be all inter-related. Aside from a larger than neat-line excavation (width of the excavation tool) and some carryover of slurry with spoils during removal, the mechanisms listed above are the main contributing factors for volume changes. The facts are that substantially additional volumes of slurry greater than neat volume are almost always required for barrier wall installations. The somewhat misleading termed "over-break" is often used to refer to this requirement for additional slurry. This is commonly 30% to 80% over and above the neat-line excavation volume. This can well exceed 100% or more in many cases. Typically what is experienced at an excavation after the panel is completed is a drop in the slurry level, and water begins to collect on top of the slurry level. Consolidation of the slurry manifest itself as water on the surface with a corresponding drop in slurry level, but no change in liquid level. This can be imagined as the solids in the slurry settling under the force of gravity, what is commonly referred to as bleed. Filtration manifests itself as a drop in slurry level but does not contribute significant to water accumulation at the surface, as the water is permeated out of the slurry and into the adjacent strata.

Contamination by Dispersed Sediments

This is essentially the process of mechanical excavation through non-cohesive soils whereby there is dispersal of excavated soils with the movement of the clamshell or backhoe bucket. Also the deliberate draining of the clamshell or bucket of slurry at the surface will carry significant amounts of non-cohesive soils back into the slurry trench. API Sand Content Tests may demonstrate an increase in measurable sand to 20% or larger, with corresponding increases in density of 10 to 15 PCF. Naturally, pore water associated with these sediments also becomes incorporated into the slurry as well, with a corresponding dilution effect of the original slurry composition. Mass balance calculations can show that in porous saturated strata, additional water moisture may become significant, perhaps adding as much as 7.5% to

© 2016 Deep Foundations Institute

10% to the volume. Data derived from mix designs the author has been involved with have indicated a wide variation in results. In general, for the very high W:C SCCB mixes, incorporation of excavated/disturbed soil will have some detrimental effect on UCS, particularly at early tests. For relatively low water to cement ratio SCCB's, incorporation of soils may actually improve strength. It is therefore wise to use the actual alignment soils expected to be encountered when performing mix designs in the lab as the goal should be to determine what effect, if any, they will have.

Sedimentation and Suspension

Dispersed sediments in the slurry may settle once the mechanical agitation of the slurry from the excavation process ceases. This is the case for both the contaminating soils and the small particles making up the virgin slurry composition. For the case of the virgin slurry solids, the magnitude of gravitational settling is usually very small. According to Carreto, et al. (2016), laboratory studies have reported rates of 2.5 to 4.0 mm/h reported for the velocity of the water-slurry interface. This type of sedimentation manifest itself in what is typically termed the bleed. Percent of bleed for stable suspensions is usually on the order of 0.5% to 2.0% for SCCB and 1.0% to 5.0% for CB. The settling of larger particles is hindered by the time-dependent gel-strength increase in the slurry. Typically, gel-strengths of the quiescent slurry are high enough to suspend most, if not all, sand-sized particles. Fine and course gravels have also been suspended. In extreme cases, as has been witnessed by the author, even small cobbles may be suspended. The addition of sediments to the gel-structure of the slurry naturally increases the slurry density. According to Carreto, et al. (2015) the shear increase of mass contributes to a greater consolidating force with depth.

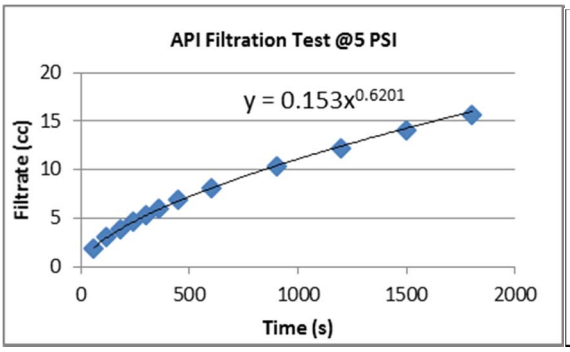
Whole Slurry Loss

As a result of the pressure exerted by the combination of slurry density and depth (hydrostatic pressure) the slurry may be forced into very permeable strata. This may be cobble-bearing strata lacking interstitial finer particles, or some gross void or conduit. Large voids are not common as most natural strata will have gravels and fines within the larger cobbles. Little to no whole slurry is able to penetrate strata finer than coarse sand Carreto, et al. (2016) under normal conditions. Lab studies performed by the author have indicated this is the case as well, It appears the loss of slurry volume is attributed mainly to the mechanisms of consolidation and filtration.

Filtration

Water is generally on the order of 90% or greater of the original volume of the slurry. Evans et al. (2005) references a "typical Slag-CB mixture" (SCCB) as having 934 kg of water per cubic meter, this formulation obviously being 93.4% by volume water. Evans et al. (2005) further asserts that some of this water will flow into permeable strata due to force exerted (hydrostatic pressure) by its mass and that of all suspended slurry solids. This will behave according the laws of filtration and will result in the deposition of a filter cake at the trench sides and bottom of the excavation. In the circumstance of static filtration, where the filter cake build-up is not continually removed by mechanical forces such as the steel sides and teeth of a backhoe bucket, the rate of filtration decreases with time and is proportional to the square root of time. Put another way, for a cumulative filtrate over time "t", it will take the passage of "4t" to double the filtrate volume passed in first time interval. This means that initial losses may be quite dramatic but they tail off after a period of time. A typical plot of cumulative filtrate increasing with time is demonstrated with Figure 2a below. During excavation of a cut or panel, the situation is somewhat different in that the filter cake is not necessarily allowed to fully develop. Therefore, filtration (dynamic filtration) may continue to proceed at rates close to the initial rate during excavation, but the phenomenon will not be noticed by casual observation. A careful accounting of the slurry placed in a geometrically well-defined panel will demonstrate the rate of loss during the excavation process. From a careful analysis of filtration, it can be shown that much of this is loss of slurry water through the filtration mechanism.

© 2016 Deep Foundations Institute



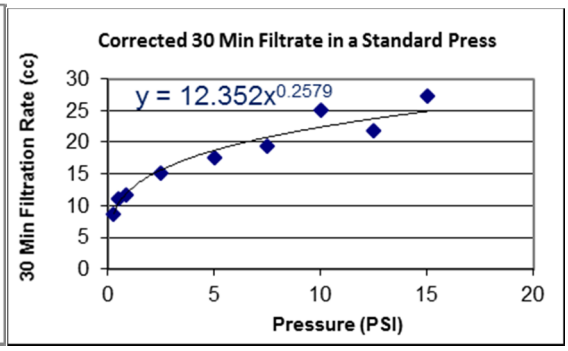


Fig. 2a Filtrate vs Time for an SCCB **Fig. 2b Filtration Rate vs Pressure of an SCCB**

The effect of increasing hydrostatic pressure with depth increases the potential rate of filtration, but this effect is less pronounced than one may initially surmise. As stated by Caenn et al. (2011) "*As pressure increases, the filter cake increases in density, lowering the void ratio and hence the coefficient of permeability. Filtration rate is proportional to pressure raised to the power of some exponent. Although the standard equation for static filtration predicts a exponent of 0.5, it is nearly always less.*" For the test data depicted in Figure 2b, the exponent is found to be 0.2579. This means for a 50 foot deep trench, the filtration rate at 50 ft. of depth is only ($50^{0.2579}$) or 2.74 times that at 1 foot of depth. That is already almost 40% the filtration rate at the bottom of the excavation! When you consider that groundwater levels diminish this pressure difference substantially, the filtration rate differences become that much less. For example, if groundwater were at 10 feet of depth in our hypothetical 50 foot deep trench, and with the slurry density of 71.9 PCF, then the differential pressure at 50 feet is equivalent to a hydrostatic pressure calculation of only 16.9 feet due to the buoyancy of the water. The same slurry will have a filtrate rate at 50 foot depth of 2.07 times the filtrate rate at 1.0 ft. So the filtration rate at 1.0 ft. of depth is 48% of the rate at 50 feet of depth, and at 2.0 ft. is 58%.

Filtration, of course, is a phenomenon associated with the fluid state of the slurry. When the self-hardening slurry becomes a solid (sets), filtration is thought to cease according to Carreto (2016). It is arguable where the boundary lies between the liquid state and the solid state of these particular slurries, and so the question may be asked at what point does the filtration mechanism cease, or become insignificant, and what other mechanisms may continue to dominate? Carreto et al. (2016) states that setting of CB slurry will occur after a 2-h rest, and that this would lead to a water loss of about 5% of the volume excavated. For SCCB investigations performed by the author, the filtration mechanism proceeds for much longer periods of time, spanning approximately 24 hours or greater. This is no doubt due to the longer set times demonstrated by GBFS when compared to Portland cement. Fluidizing admixtures, which typically act as cement retarders, may well extend this time significantly. Many of the SCCB slurries may be kept liquid for days, even up to a week! Filtration may therefore account for a greater significant portion of the volume change than what other investigators have postulated, especially for excavations which require extended time to complete. This point is further elaborated in the later proceeding section "Bentonite Role on Minimizing Volume Changes".

Consolidation

The mechanisms of consolidation may not be completely separated from the mechanism of filtration, as they appear to this author to be the result of similar circumstances or slurry composition. Put another way, slurry with high filtration rates will tend to have correspondingly high consolidation rates as well. For slurry at the boundaries of the excavation, the phenomenon is loss of water content by filtration. For slurry not intimately in contact with the boundary of the sidewalls or developing filter cake, the mechanism for loss of water content is that of consolidation. If one thinks of the slurry in the trench as a stack of very thin two dimensional horizontal layers of slurry, with the solids in the slurry bearing the weight of the solids in the layer above it, there is a squeezing effect to compress the slurry planes

© 2016 Deep Foundations Institute

according to the cumulative mass that they support. The loss of water will occur into the element of slurry directly above it, and that element, in turn, will lose water to the element directly above it. This will be translated vertically through all the elements of slurry, one above the other, then towards the surface where small water streams will coalesce. The interface between the slurry and the surface water will drop, as the zone of consolidation moves ever upward. But, as postulated above, absent the mechanism of filtration (only if the excavation was impermeable) there will be no hypothetical liquid level drop.

MIX DESIGN RATIOS

"Water" to "Cementitious Material" Ratio (W:C)

The primary parameter of concern for achieving the set and strength development properties of CB and SCCB, is the W:C of the slurry. Decrease of the W:C (increase of cement to water ratio) will naturally increase the ultimate strength of the SCCB. Unlike concrete slurry, SCCB slurry may be formulated over a wide range of W:C and these ratios may be ten to twenty times greater than that of concrete. As per Figure 1, the W:C ratios of between 4 and 10 are typical ranges for SCCB. The author has experienced most typical specifications in the United States requiring 15 psi to 75 psi in 28 days. Therefore, most specifications can be met in the lab with W:C of 10 or more. Yet, as noted previously, this ratio in the trench is will change from what it was originally at the batch plant. Even though static filtration volume loss may range between 5% and 7%, the sum of static filtration losses, dynamic filtration losses and consolidation of the slurry may be close to 50% of the in-trench slurry. This can be shown by applying measured filtration rates of the slurry to assumptions of excavation and set times. The loss of water from the in-trench slurry results a proportional decrease of the W:C .

Jefferis (2012) states that *"an equation of UCS as a function of W:C ratio does take the power law equation form:*

$$S = aW^{-b} \quad (1)$$

"Where "W" is the water to cement ratio, "a" is a constant determined by a particular mix, and the exponent b is typically in between 1 and 3." It can be readily seen that the ratio of the strength for the slurry having one-half the water to cement ratio over the original slurry is:

$$(1/2)^{-b} \quad (2)$$

Several of the author's mix designs demonstrated the "b" exponential to fall in about 2.3, so this percentage of water loss would represent a strength increase of 390% of the original mix design strength! This dramatic increase in strength is unlikely to be the case for the majority of the slurry in the trench but may likely exist within the filter cake. Since the water-loss effect of filtration and consolidation may have a substantial effect on strength and likely exceeding the designer's strength specifications, it stands to reason that modulating this effect would be beneficial and provide for increased uniformity throughout the barrier wall width and depth.

Bentonite to Water Ratio (B:W)

According to Jefferis (2012), formulations of SCCB B:W are generally kept between 3.0% and 6.0%. Bentonite slurry concentration may be adjusted up or down to accomplish mix design goals. At a minimum, the bentonite provides for the increase in gel structure (shear strength) to support alignment soils incorporated into the trench slurry, as well as providing some minimum degree of filtration control. As Figure 3 demonstrates, increasing B:W provides filtration (fluid loss) control for the system. As the B:W increases, then the filtration control properties improve, reducing loss of the water phase to the soil strata adjacent to the trench. An in-depth discussion on bentonite in SCCB continues later under "Additional Aspects".

© 2016 Deep Foundations Institute

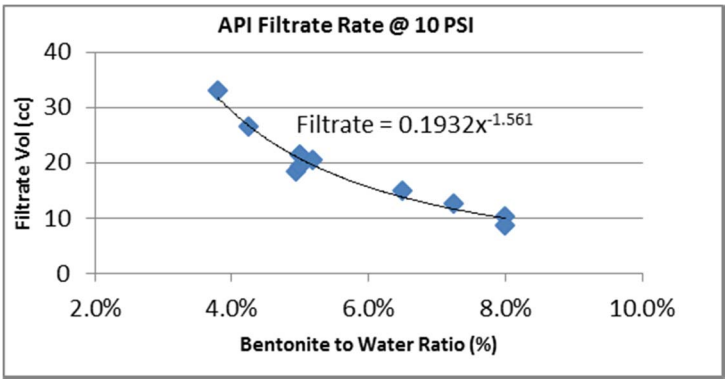
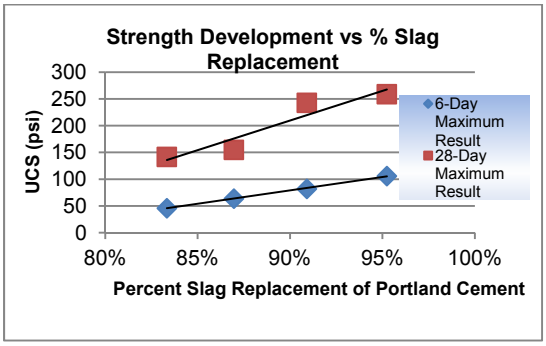


Fig.3 Data derived from studies performed in 2011

Percent Portland cement Replacement

Portland cement will help accelerate the pozzolanic reactions of the GBFS, allowing the SCCB slurry to achieve a set and develop strength in a reasonably short time of one to three days. This need not be a very high proportion of Portland cement. The author has utilized formulations with the Portland cement being only 2.5% of the cementitious material. Generally speaking, substituting Portland cement in place of GBFS will tend to decrease UCS due to the resultant decrease of the higher effective GBFS. Additionally, increasing the Portland amount will have the effect of increasing hydraulic conductivity. Using higher percentages of Portland cement also places a strain on material handling, requiring the use of additional silos and equipment. According to Evans et al. (2005) GBFS replacement of up to 80% improves the strength and permeability properties of the SCCB, but that further substitution may not offer additional improvements. The authors own testing experience, of which a set of data is depicted in Figure 4a and 4b, have demonstrated improved strength and decreased permeability values when the percentage of substitution increases toward close to 100%. This evidently will depend upon the differences among particular products and mix design ratios utilized for testing purposes.



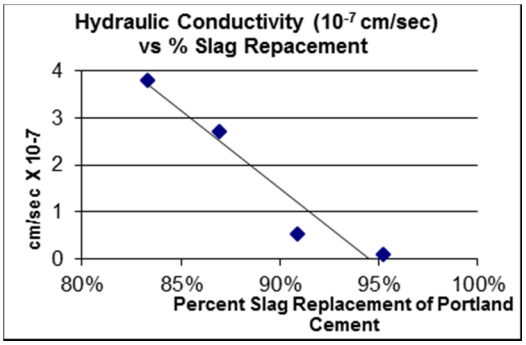


Figure 4a Increase in UCS With Increase Slag **Figure 4b Shows Hydraulic Conductivity Decrease**

ADDITIONAL ASPECTS

Bentonite Role in Minimizing Volume Changes

As discussed and depicted in Figure 3, the increase of bentonite above the minimum value required for a stable grout suspension has the benefit of imparting additional filtration control to the slurry. All things being equal, this would reduce the overall percentage of volume loss attributed to filtration and consolidation mechanisms prior to initial set. Likewise, reducing the potential for consolidation and filtration reduces the amount of bleed water at the surface and the drop in liquid in the excavation. Taking this into consideration, and that a typical trench panel would take four hours to complete, and 20 hours to arrive at an initial set, it is possible to calculate the minimum volume of water lost from the filtration

© 2016 Deep Foundations Institute

mechanism would be over this 24 hour period of time. Figure 5 illustrates such a calculation, based upon measurements taken of filtration rates of SCCB Slurry. This calculation result did take into consideration the pressure differences that occur with depth but not the increase of filtration due to the dynamic processes of removing filter cake.

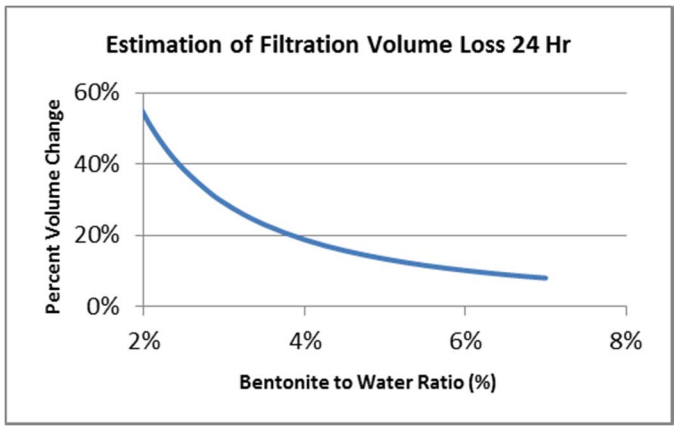


Fig. 5 Depth of Trench is 50 ft., Ground Water Depth is 10 ft.

SCCB Ultimate Composition

Due to the high water content of SCCB (often in around or even in excess of 90% by volume), the as-yet-set SCCB has a high potential for volume reduction to occur via the two mechanisms of filtration and consolidation. Volume changes due to consolidation may be as much or more than filtration. Filtration occurs immediately as soon as the slurry is placed into an excavation with permeable boundary soils or strata. This means a filter cake is formed at the boundary, and progresses towards the middle until filtration ceases due to set. Consolidation should no longer occur when the strength of the resultant GBFS-Cement-Bentonite structure is as great as the overburden forces caused by the cumulative mass of the setting slurry above it. A progressive decrease in water content may be expected for the slurry at depths experiencing higher hydrostatic pressure. Increase of B:W should tend to make for a less consolidated, more uniform in composition, and more ductile wall.

It is interesting to study the relationship between water retained in fully consolidated SCCB Slurry or Filter Cake and the B:W and W:C of original slurry composition. It noted that bentonite has a much higher capacity to retain water than the cementitious component of the SCCB. An experiment was performed in which six different B:W SCCBs were fabricated. The same weight percent concentration (W:C = 7.5) of cementitious was maintained throughout the six variations. Variations in the B:W ranged from 3.5% to 8.0%. A fluidizer was used to maintain workable viscosity, increasing the dosage toward the higher end of the range. Fabricated specimens were placed into flexible wall permeameter cells and subjected to 10 psi of confining pressure. Specimen drainage from both the top and bottom were directed to a single outlet and the "filtrate" was measured against time. Data points were taken over the course of an approximate 24 hour period. The plot of accumulated filtrate against time took the shape of a typical Filtration Test plot, as has been the case in other studies such as Bolton at al. (1999). At or around the end of 16-24 hours, there was a cessation of filtrate exiting the permeameter. Based upon mass balance, the consolidated sample volumes were calculated, and the new constituent concentrations of cementitious, bentonite, and water determined. The analysis of the results demonstrated that as the B:W increased, the water retained in the wholly consolidated slurry was proportionally greater, at an average of 3.6 lb. of water per lb. of bentonite. This is roughly ten times the adsorptive capacity of cement (neglecting longer term hydration), which was determined in a similar type test where the cement was varied while keeping the B:W constant. In short, the greater the original B:W, as illustrated in Figure 6, the greater the ratio of consolidated slurry volume to volume of filtrate passed.

© 2016 Deep Foundations Institute

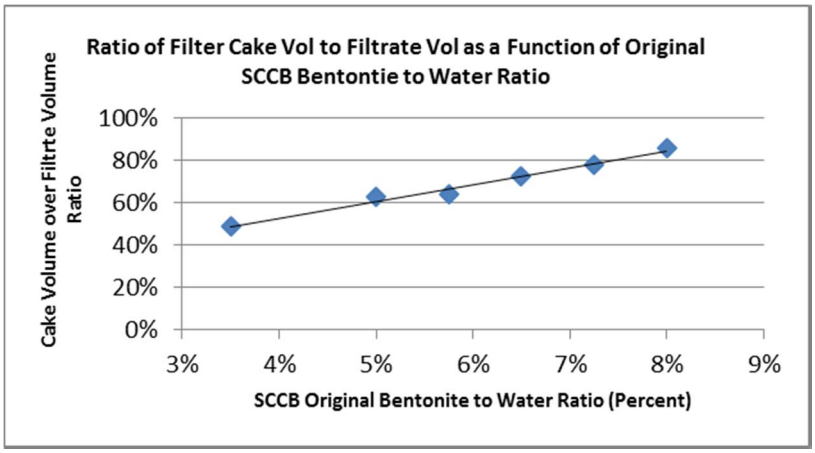


Fig. 6 Percent Consolidation at 10 psi as B:W Increases

In an extreme case, filtration in a pure grout with little or no bentonite would result in a very dense high cement-low water content filter cake, with very high filtration rates from the slurry. The resultant cake would set at a very high strength, and would tend to be thin, extremely high in strength, and very brittle. Likewise, having just enough cementitious to get a set and have some strength, one would get very thick cakes with high water content (and high clay content of course). This would have relatively low strength yet could well meet permeability and strength specifications, as well as provide for a more uniform composition across the width of the trench or boundaries.

Fluidizing Agents:

Many SCCB specifications allow for the use of fluidizing agents. The specifications will typically state this as being "mud or slurry conditioning agents used commonly in oil well drilling muds". This should not be confused with water softening agents such as sodium carbonate or lime. In light of the discussion above regarding bentonite in SCCB, it is often desired to formulate SCCB with higher than minimum bentonite concentrations. This creates a somewhat troublesome thickening of the slurry, which in turn tends to make mixing and pumping more challenging. By use of a proper fluidizing agent, the higher bentonite concentrations may be achieved without this effect. More efficient mixing and pumping may proceed, with a higher degree of control over slurry properties.

Make-up Water:

The water chemistry should promote excellent hydration of sodium bentonite. In short, make-up water should have low hardness and salinity. Water harness and salinity will prevent the expansive bentonite clay particles from fully adsorbing the great quantity of water that they are capable of. The water should also be free of detrimental organic matter (algae blooms or contaminants) that might have some adverse effect on the GBFS setting and curing. Treatment of hard water may be effectively done with soda ash. This will normally boost the water to a pH of 10 or more, and will also tend to increase the pH of the SCCB slightly. Overtreatment of soda ash may likely have a detrimental effect on the setting and curing of the resultant SCCB.

***Mixing Energy*:**

Additional considerations in both on the lab and field include how much energy is applied to each step of mixing. It is recognized that regardless of which method of mixing is employed, if mixing effort is inadequate a poorer final product may result. Bentonite mixing should have high shear applied to properly mix, disperse, and hydrate the bentonite. Cementitious agents also should be subject to high shear during the mixing process. If Method 3 is used, there needs to be thorough mixing of the bentonite and cementitious slurry such that complete homogenization of the two is accomplished

© 2016 Deep Foundations Institute

Order of Additions: By changing the order of additions, the set times, cure properties, and ultimate properties can be affected. For example, if bentonite were to be added dry to the cementitious and water grout, a different product would result than if the order was reversed. Adding fluidizing agents directly to the grout (Method 3) instead of to the SCCB blend affects how the SCCB cures. Mixing things up with Method 2 will yield slightly different outcomes for flow properties and UCS. For the purposes of following a particular mix design, this should not be neglected as an important factor.

Chemical Contaminants:

Chemical contamination of SCCB Slurry occurs when ground conditions or hydro geologic conditions or the excavation process contribute a soluble chemical species which have an adverse effect upon the slurry itself, and/or contribute to sub-design results of desired properties of the cured SCCB. The author has observed this on several projects where pulverization of mineral-rich rock by the excavation process appeared to aid in the solubilizing of minerals such as iron. Adverse slurry property changes include increase in filtration rate, changes in viscosity, and changes in shear strength. Adverse effects on cured properties are usually measured in increased hydraulic conductivity over mix design (or baseline), and decrease (or delay) in strength development. The neutralization of contaminants is sometimes possible, preserving the original design properties. This is a topic in which further much further discussion and investigation is warranted.

SUMMARY AND CONCLUSIONS:

Mixing methods in the field and laboratory may be approached differently, with different results. Mix design mixing method should correspond to the method implemented in the field. Further research and testing may be performed to further quantify advantageous of a particular method.

GBFS replacement of Portland cement above 80% may be beneficial in improving both UCS and permeability rates, despite prior studies indicating the contrary. There is a substantial difference in the UCS development of SCCB and SB. This difference is up to 20 to 30 times greater for SCCB over CB. Furthermore, hydraulic conductivity results, are often one hundred times lower than those of CB.

Increasing the percentage of bentonite above minimum acceptable values has a positive effect of filtration control and minimization of consolidation, the two mechanisms responsible for the greatest volume changes that CB and SCCB undergo. The ultimate ability for retention of water from bentonite is ten times that of cementitious materials in self-hardening slurries utilizing bentonite. The use fluidizing agents enable higher quantities of bentonite to be used and offer further control and flexibility over slurry properties. Improvements to final cured product may include improved homogeneity throughout the width and depth of the excavation.

A careful study of stress-strain relationships with varying ratios of B:W could be a fruitful area of research. Since the potential for consolidation is a largely a function of the percentage of water which would be retained while the slurry is under confining pressure, and prior to set, it is seen that bentonite will decrease filtration and consolidation potential much greater than cementitious materials. Lab studies scaled up closer to actual trench depths could measure the consolidation mechanism, both as a function of varying B:W and suspended sediment concentrations.

It is apparent that many considerations may be applied in developing a mix design for SCCB. It is important that a clear understanding of the project parameters are made available to both the owner's design team and representative, and the potential contractor expected to install the product specified.

© 2016 Deep Foundations Institute

ACKNOWLEDGMENTS

I would like to thank the many individuals and companies that have given me the opportunity to engage in their various self-hardening barrier wall projects. These include most notably Magnus Pacific (GLEI), Envirocon, Remedial Construction Services, and Contractor Services Group.

In particular, I would like to thank Ken Criley of Geo-Logic Associates for giving me free roaming privileges. Of special mention is Jodi Laferte for her adept work and attitude in performing many tests over the years while being a valued assistant in the lab.

REFERENCES

Bolton, M.D., McKinley, J.D. 1999. A Geotechnical Description of Fresh Cement Grout - Filtration and Consolidation Behavior. Magazine of Concrete Research 51, No. 5, October 1999, 295-307

Caenn R., Darley H.C.H., Gray G.R. 2011. "The filtration properties of drilling fluids", Composition and Properties of Drilling and Completion Fluids, 6th Ed. Gulf Professional, Waltham, MA. 272-305

Carreto J.M.R.; Caldeira, L.M.M.S.; Maranha das Neves, E.J.L. 2016. Processes Involved in the Formation and Performances of Self-Hardening Slurry Walls: Santa Clara-a-Velha Monastery Cutoff Wall. Journal of Geotechnical and Geoenvirinmental Engineering. 10.1061/(ASCE)GT.1943-5606.0001483

Carreto J.M.R.; Caldeira, L.M.M.S.; Maranha das Neves, E.J.L. 2015. Hydromechanical Characterization of Cement-Bentontie Slurries in the Context of Cutoff Wall Applications. Journal of Materials in Civil Engineering, Vol 28, Issue 2 (Feb 2016). 101.1061/(ASCE)MT.1943-5533.0001365, 04015093

Evans, J.C.; Opdyke, S.M. 2005. Slag-Cement-Bentonite Slurry Walls. Journal of Geotechnical and Geoenvirinmental Engineering. Vol. 131, No, 6 June 1, 2005. ASCE. ISSN 1090-0241/2005. 673-681

Jefferis, S.A. 2012. "Cement-Bentonite Slurry Systems" Grouting and Deep Mixing, L.F. Johnsen, D.A. Bruce, and M.J. Byle, eds., ASCE, Reston, VA . 1-24

Lebedenko, F., Letellier, M.; Plee, D.; Van Damme, H. 1988. Microstructure, Permeability and Rheology of Bentonite - Cement Slurries. Cement and Concrete Research. Vol. 20 ooo8-8846/90 Copyright © 1990 Pergamon Press plc. 45-61

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

CASE STUDY: FEATHER RIVER WEST LEVEE PROGRAM CUTOFF WALLS

Jared P. Williams, GLEI, Rocklin, CA USA 916-462-6400 jaredwilliams@gleis.com
Kyle R. Lackner, GLEI, Rocklin, CA USA 916-462-6400 kylelackner@gleis.com

ABSTRACT

The Sutter Butte Flood Control Agency initiated the Feather River West Levee (FRWL) Program in 2010 to bring 35 miles of levee along the Feather River into compliance with applicable Federal and State standards for levees protecting urban areas. A combination of soil-bentonite cutoff walls, soil-cement-bentonite cutoff walls, and seepage berms will reduce flood risk by correcting under and through seepage deficiencies and remove more than 34,000 properties from the FEMA Special Flood Hazard Areas. The FRWL program includes a combined total of 7,744,803 square feet (SF) of cutoff wall planned to be constructed between 2013 and 2015. Cutoff wall construction methods to meet the design depths included conventional excavation to a depth of 85 feet and deep soil mixing for depths up to 120 feet. A key component of the FRWL program was the development of site specific mix designs using the design boring logs to evaluate the expected backfill gradation and permeability results. Issues that required a partnership between Contractors, Owner and Engineers included substantial Archaeological and Cultural Resource delays, significant depth changes due to a change in competent key, right of way acquisitions for abutting land, and the magnitude of the project in a resource limited industry.

INTRODUCTION

The Feather River watershed basin is located 50 miles north of Sacramento, CA and incorporates the agriculture producing communities of Yuba City, Live Oak, Gridley and Biggs. These communities are protected from flooding by levees that contain under-seepage and through-seepage deficiencies and were the cause of major failures in 1955, 1986 and 1997. The Feather River West Levee (FRWL) program was initiated by the Sutter Butte Flood Control Agency (SBFCA) in 2010. The long term objective of SBFCA is to correct the levee deficiencies and provide 200 year flood protection to more than 34,000 at risk properties located within the FEMA Special Flood Hazard Areas.

Although FRWL contained a variety of construction components such as earthwork and pipe replacement, the slurry cutoff wall is the primary construction element for addressing under and through seepage deficiencies. From 2013 to 2015 approximately 6,649,210 SF of cutoff wall were installed using conventional excavation and deep soil mixing (DSM) techniques leaving 1,095,593 SF to be completed in 2016. The work was designed, solicited and constructed under two separate contracts, titled Project Area C and Project Area B&D. A summary of the contracts and cutoff wall quantities is presented in Table 1.

Table 1. Slurry Cutoff Wall Quantities

Contract / Year	Conventional Excavation	Deep Soil Mixing
Project Area C		
• 2013	151,906 SF	544,364 SF
• 2014	2,091,589 SF	225,513 SF
Project Area B&D		
• 2014	2,247,470 SF	NA
• 2015	1,388,368 SF	NA
• 2016	1,095,593 SF	NA
Totals	6,974,926 SF	769,877 SF

© 2016 Deep Foundations Institute

FRWL Project Location

FRWL's first contract was Project Area C which is located within the Yuba City and Live Oak city limits with a total length of over ten (10) miles. Project Area's B&D were the second with Area B located just south of Yuba City extending for just over six (6) miles and Area D which spans from Live Oak to the Thermalito Afterbay encompassing a length of eleven and a half (11.5) miles. Figure 1 shows the geographic locations of the different reaches as well as the designed remedy.

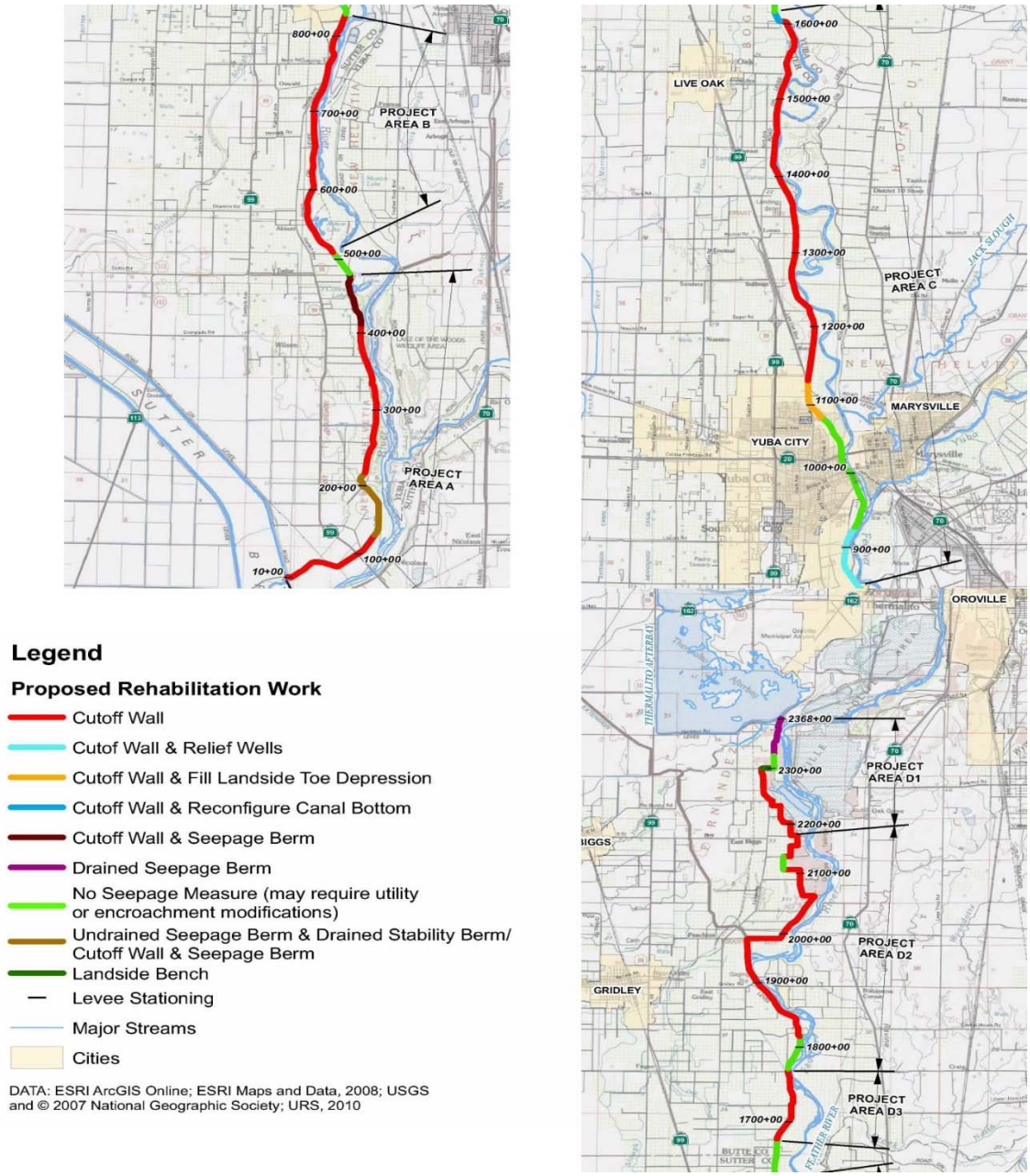


Figure 1. FRWL Project Areas – Credit Sutter Butte Flood Control Agency

© 2016 Deep Foundations Institute

CUTOFF WALL METHODOLOGY AND DESIGN

Although multiple cutoff wall techniques were allowed per the Contract, the two chosen for FRWL were conventional excavation and deep soil mixing (DSM). DSM was utilized for depths exceeding 85 ft. and provides the opportunity of reaching depths greater than 200 ft.

Conventional Excavation Equipment

The specialized equipment utilized for conventional excavation cutoff wall construction includes large stock excavators with custom built booms and sticks. Historically, the boom and stick are designed and built by the cutoff wall contractor and an independent equipment manufacturer, and not by the manufacturer of the hydraulic excavator. Consequently, careful planning and design considerations have to be given to details including overall weight, balance, geometry, connections with the parent machine, and hydraulics to achieve a functional boom and stick capable of achieving the maximum depth possible. Figure 2 illustrates the range of motion with regard to maximum reach and excavation depth of a custom built PC1250 long boom and stick.

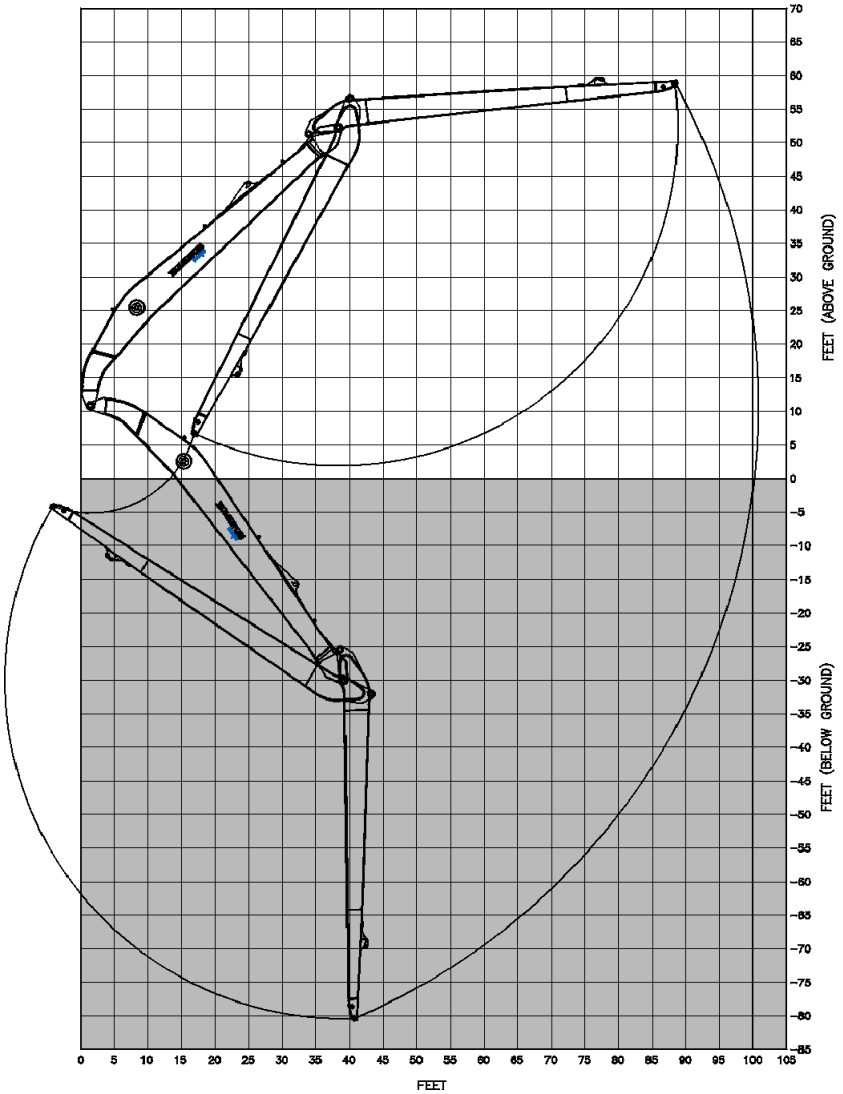


Figure 2. PC1250 Range of Motion

© 2016 Deep Foundations Institute

Equipment limitations for the custom boom and stick, require that the constructed trench width is 36 inches. This measurement is determined and monitored in the field by measuring the width of the excavator bucket. The following are the advantages to utilizing this method for cutoff wall construction:

- The cutoff wall can be excavated to the full required depth in a single pass
- The cutoff wall backfill is mixed ex-situ thus allowing for visual inspections
- Keyed-in cutoff wall samples can be verified through visual inspection of removed samples

In addition to the specialized excavation equipment, there is also standard size equipment used for mixing and placing backfill. This includes at a minimum, one excavator and one dozer (i.e. PC 200 excavator and CAT D6 LGP dozer) for mixing the backfill materials on the surface of the work pad. Soils can be pre-mixed by the excavator to break down large clumps of clay or soil, prior to tracking through the material with the dozer.

When sections of levee experience work pad width limitations (reduced mixing platform) or geotechnical challenges, as was prevalent during FRWL, a standard long reach excavator as well as haul trucks (i.e. Hyundai 290 excavator and HM350 haul trucks) can be used to move suitable material to and unsuitable material from the backfill operation. Also, import material was evenly distributed based on design borings along the cutoff alignment, ahead of the excavation using a front end wheel loader (i.e. Komatsu WA270).

Slurry mixing equipment included a jet shear mixer, multiple six (6) inch trash pumps for circulation, four (4) inch HDPE pipe for conveyance, and an all-terrain forklift for handling 3,000 pound bentonite super sacks. The slurry pond consists of constructing a minimum 50,000 gallon capacity cell, usually by excavating a location within a staging area or on the workpad, into the ground surface by 2-3 feet, and building 3-5 foot berms around the perimeter of the pond. The size of the pond is generally dependent on the project demands, including constructing one slurry trench heading versus multiple slurry trench headings, anticipated daily production rates, and other slurry demands. The length (linear feet) of the project would also dictate the need for intermediate slurry ponds and transfer pumps for slurry conveyance.

Contract Acceptance Criteria

The contract acceptance criterion for conventional excavation includes the following:

- Wall Depth
- Minimum Wall Thickness of 36 inches
- Maximum Bulk Sample Permeability of $5x10^{-7}$ cm/s
- Continuity and Homogeneity: Evidenced by the preponderance of Quality Control and Quality Assurance test results and inspection of the soil-bentonite backfill placement by the Agency.

The contract acceptance criterion for the DSM method includes the following:

- Wall Depth – Depth's based on the verification drilling results which were performed before construction and at the Agency's direction.
- Wall Thickness – Minimum 27 inches
- Wall Continuity – The maximum incremental deviation between two adjacent panels at any depth shall not be greater than twenty (20) percent of the specified minimum wall thickness. The accumulated total deviation of any single panel from vertical shall not exceed one (1) percent of the panel depth.
- Continuity and Homogeneity: Evidenced by the preponderance of Quality Control and Quality Assurance test results and inspects.
- Maximum Bulk Sample Permeability of $5x10^{-7}$ cm/s
- Bulk Sample Strength of minimum twenty (20) psi at 28 days, and maximum 100 psi at 28 days
- Minimum Cement Content of 0.5% of the cutoff wall mix design

© 2016 Deep Foundations Institute

The wall depth is a critical criterion established for correcting the under seepage deficiencies. Although typical cutoff walls are keyed into an underlying confining layer, the analyzed head pressures and groundwater hydrology concerns could be addressed by the design of a hanging, or non-keyed, cutoff wall. In these cases, the design depth of the cutoff wall was independent of identifying an underlying confining layer. However, other locations required a deeper cutoff wall, and confirmation of keying into a previously identified confining layer, such as verifiable and competent clay or silt.

The minimum wall thickness for conventional excavation was specified at 36 inches on all contracts for FRWL. This was achieved during the conventional excavation method by making a single unobstructed pass with the excavator through the slurry trench and dictated by the width of the excavator bucket. For DSM, although the auger is 36 inches wide, the wall thickness requirement was only 27 inches to accommodate the width of the radical line between the overlapping augers.

The maximum bulk sample permeability requirement for all contracts was $5x10^{-7}$ cm/s. Testing was performed in a flexible wall permeameter in accordance with ASTM D5084, using the following parameters:

- Average effective confining stress of one (1) to five (5) psi
- Maximum hydraulic gradient of fifteen (15) psi

Unconfined compressive strength (UCS) applied only to the DSM method in which soil-cement-bentonite (SCB) was used. The specified parameters were between 20 and 100 psi at 28 days.

Continuity and homogeneity acceptance criteria were met based on results of the following tests:

- Slump testing in accordance with ASTM C143/143M to be between four (4) and seven (7) inches
- Slurry testing per applicable American Petroleum Institute (API) procedures
- Backfill density testing per ASTM C117 and C136 to be at least fifteen (15) pcf greater than the maximum in place slurry density
- Visual inspection performed by both experience quality control and quality assurance personnel
- Soil gradations testing in accordance with ASTM D1140 and D422 within the limits of Table 2

Table 2. Specified SB Cutoff Wall Backfill Gradation

Sieve Size or Number	Percent (%) Passing by Dry Weight
2-inch	100
No. 4	40 to 100
No. 40	25 to 90
No. 200	20 to 60

The lower end of the No. 200 sieve gradation is based on having sufficient material to meet the specified maximum bulk sample permeability requirement. The upper end of this requirement is based on minimizing long term consolidation of the backfill material due to an overabundant presence of fine clays or silts.

Bentonite Slurry Quality Control

Implementing proper quality control procedures has become a critical element of slurry cutoff wall construction. Today's procedures and specification requirements, versus those as few as ten (10) years ago, are more stringent and refined in relation to defining the overall performance of the cutoff wall. Although the API procedures are relatively similar with the past, the specifications may require additional testing or parameters that are unique to individual site conditions rather than simply following the standard API requirements. The advancement in testing requires that the Contractor maintain a technically knowledgeable staff to both perform and analyze the results.

© 2016 Deep Foundations Institute

FRWL required field testing of the initial bentonite slurry for viscosity, density and filtrate loss to be within the following specified parameters:

Table 3. Initial Bentonite Testing Methods and Parameters

Test Method	Parameters
Viscosity – Marsh Funnel	Minimum of 40 seconds
Density – Mud Balance	Minimum of 64 pounds per cubic foot
Filtrate Loss – Filter Press	Maximum 1.75 cubic inches in 30 minutes at 100 psi

For FRWL, the trench slurry was tested at multiple depths and locations within the trench. This included a top, middle and bottom sample at a distance no less than 5 feet from the toe of the soil-bentonite backfill slope. Testing procedures were the same as the initial bentonite slurry with differing result parameters for both density and filtrate loss.

Table 4. Trench Bentonite Slurry Testing Methods and Parameters

Test Method	Parameters
Viscosity – Marsh Funnel	Minimum of 40 seconds
Density – Mud Balance	Minimum of 64 pounds per cubic foot, maximum of 85 pounds per cubic foot
Filtrate Loss – Filter Press	No parameter just submit result of the filtrate loss in 30 minutes under 100 psi

As seen with most slurry cutoff walls, the quality control testing for FRWL revealed a direct correlation between the depth of the trench and the density of the slurry, with denser slurry being found at the bottom of the trench likely due to settlement of the suspended silts and sands. However, the bentonite slurry viscosity and gel strength allow for suspension of the silts and sands, generated during the excavation process, which if managed correctly, do not settle rapidly enough to cause quality control concerns.

A major focus of the quality control program was managing the density of the trench slurry. At times, particularly when performing deep excavations, densities would rise above 85 pounds per cubic foot. To mitigate this becoming an evolving issue, we would install a desander unit that would be moved along with the excavation. Additional measures also included maximizing the distance between the backfill toe and the excavation to introduce fresh slurry or in extreme cases, pumping out / disposing the trench slurry while also adding fresh slurry.

Soil Gradations and Settlement Monitoring

Soil gradations were performed in the past generally to verify if the percentage of silt and clay was acceptable to produce a low permeability mixture. However, design concerns in levee restoration involve the potential for long term secondary consolidation of the cutoff wall which could potentially create a void between the top of the cutoff wall and levee fill material. Long term secondary consolidation concerns are minimized by limiting the allowable amount of silt and clay in the backfill material to less than 60% passing the No. 200 sieve.

Because the soil-bentonite backfill is placed in a loose and saturated state, some initial settlement or consolidation is anticipated immediately after backfilling the cutoff wall. Therefore, backfilling over the top of the slurry cutoff wall is delayed until the initial settlement is complete. Locations of settlement plates include the beginning of each cutoff wall heading and at significant changes in depth, over 20 ft. Settlement plates were installed every 200 ft. at 1000 linear foot intervals and surveyed using convention surveying equipment and closed level loops over a 21 day period. The consolidation rate outcome was nearly undetectable within the settlement period signaling that the initial settlement was complete.

© 2016 Deep Foundations Institute

Permeability Testing

Hydraulic conductivity, commonly referred to as permeability, is the primary acceptance criteria element and key function of the slurry cutoff wall. Permeability testing of the soil-bentonite backfill was performed on wet bulk samples collected at frequency of once every 50 LF of top out. Permeability testing was performed in accordance with ASTM D 5084 with results typically ranging from $5x10^{-8}$ to $2x10^{-7}$ cm/sec. To date, all samples have achieved passing results for permeability and are shown below in Figure 3.

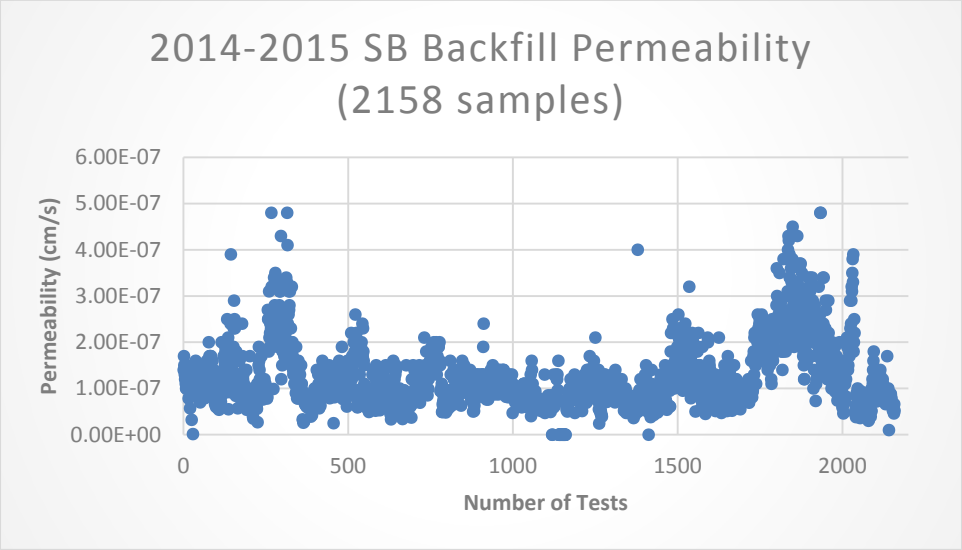


Figure 3. FRWL SB Backfill Permeability Results

Proportioning and thorough blending of material in the field can often be a difficult task due to the operation being performed in an open and unconfined setting. Because no measuring devices were used, the consistent and successful sample results for soil-bentonite backfill permeability is a monument to the overall quality control program and the performance of each slurry wall contractor.

PROJECT OBSTACLES

Obstacles are inevitable on any construction project, particularly one of this scope complexity and sheer magnitude. It is important that an ever evolving industry learns from these oversights and mistakes and come up with ways to improve the design, contracting, and construction for future projects.

Archaeological and Cultural Resource Delays

The original levees, built as early as the 1850's along the FRWL, were constructed primarily by local farmers with the intention of protecting their homes and crops from flooding. As they were constructed, the levees incorporated several mounds that are actually now known to have been Native American archaeological sites containing buried remains and artifacts.

In 2014, construction commenced on Project Area C without knowledge of the extent of Cultural sites that would be uncovered. During levee degrade it became apparent that the amount of these sites were severely underestimated and an action plan between the regulatory agencies, tribes, owner and contractor had to be developed. Because construction was already in progress, the best way forward was to have construction occur around the site investigations leaving gaps that would need to be completed after the artifact recoveries were complete. This substantially increased the cost of construction due to the amount of moves

© 2016 Deep Foundations Institute

and inherent inefficiencies, however this approach allowed for substantial completion of the Project Area C contract without extending into another construction season. Project Area B, which occurred during the same season, only had two such sites and the impact was minimized by pre-planning heading start and finish locations.

In 2015, SBFCA began working with the regulatory agencies, namely the US Army Corps of Engineers and California State Historic Preservation Office, as well as the local tribes to come up with pre-construction site specific treatment plans. This process was held up due to the handlings of the artifacts recovered from Project Area C and differing policies between Federal regulatory agencies that conflicted with the tribal requests. Federal policies stated that they must study the artifacts while the State agreed with immediately transferring all findings to the rightful tribe. SBFCA, wanting to comply with Federal permits while also having a good neighbor policy with the local tribes, were put in a very challenging position. Ultimately, negotiations and the development of agreeable site specific action plans resulted in a two and a half month delay that extended the completion of Project Area D into the 2016 construction season. The outcome of the site specific treatment plans allowed tribal monitors to be present during all work activities however, artifact investigations and recoveries would occur after construction was complete and in a manner that would not impact the newly constructed levee prism.

Competent Key Changes

Design investigations included subsurface drilling and discovery using various methods including core drilling, cone penetration testing (CPT), and sonic drilling every 1,000 ft. along the cutoff wall alignment. Because of the total length of the Project, this interval was viewed as a comprehensive analysis that would accurately identify the subsurface conditions and allow the cutoff wall tip elevation to be determined by the Geotechnical Engineer.

For the sections of levee that were determined to be at a depth greater than 85 ft., DSM was the chosen method. For DSM, the contractor was required to perform verification drilling at intervals of 100 ft. along the levee to determine the actual depth of each element. During this investigation process, it was determined by the Geotechnical Engineer that a confining layer was found much higher in elevation than was shown in the design borings. This significant change in elevation altered the methodology from DSM to conventional excavation which created many challenging contractual implications.

The Contract was specific to the method and payment based on depth intervals. For example, there were pay items for depths ranging from 50 to 78 ft., 78 to 95 ft., and 95 to 120 ft. This required the Contractor to price the methodology and any work challenges within those depth increments. The Contract also states that the Engineer is limited to decreasing or increasing the depth by maximum of ten (10) ft. This caused two questions to arise: 1) was the Geotechnical Engineer allowed to change depth by such a drastic amount? And 2) if allowed to make such a change which bid item would the work be paid under? Both items required a significant amount of time and effort to reach an agreement. After all issues were closed the one outstanding question was if this scenario could have been prevented.

Resource Limitations

The cutoff wall industry is a specialized market with limited resources that typically places a cap on how much cutoff wall can be included under a single contract. The Natomas Levee Improvement Program (NLIP), which was a large program with a similar scope, required a maximum per year of one and a half million square feet of cutoff wall under two separate contracts. FRWL, in comparison, had a maximum per year of four and a half million square feet of cutoff wall completed under two separate contracts. This magnitude made the FRWL Project Area B&D Contract, the single largest levee project in California.

© 2016 Deep Foundations Institute

To complete the size, scope and complexity of the FRWL program, a JV partnership was formed between two competitors in Great Lakes Environmental & Infrastructure (formerly Magnus Pacific) and Nordic Industries. Even with two cutoff wall Contractors, there was too much work in 2014 that the JV hired key cutoff wall subcontractors, who are also industry competitors, in Geosolutions, Inquip and Raito. This type of collaboration and partnership was required to fulfill the seasonality project constraints, which only allowed work to proceed between April and November, as well as the completion milestones mandated by the contract.

Restricted Cutoff Wall Alignment

The alignment of the levee in Project Area D contained a variety of tight corners that could not be constructed per the geometry shown in the plans. The JV came up with a cross configuration that would allow for continuity of the cutoff wall as show in Figure 4.

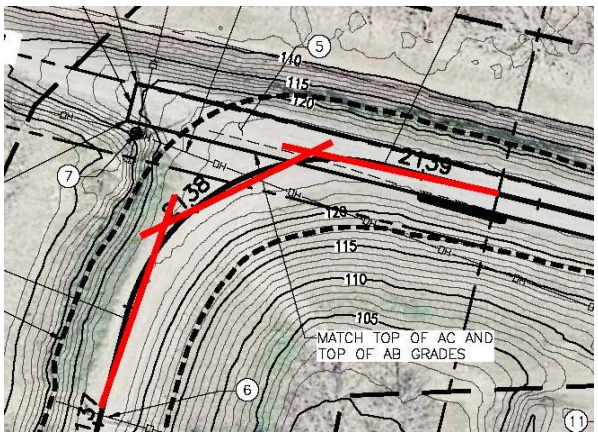


Figure 4. Cross Excavation for Tight Corners

This type of excavation requires talented operators and precise GPS layout to ensure the as built cutoff wall stays within the allowable distance from the alignment. Corners were monitored visually in the event of substantial settlement or any other unforeseen abnormalities such as slurry loss and trench instability.

LESSONS LEARNED AND CONCLUSIONS

Every project comes with key lessons learned that can help to improve the efficiency and outcome of future contracts. FRWL is no different with improvements that, if implemented, could have saved the project both time and construction costs.

Archaeological and Cultural Resource Delays

The delays associated with Archaeological and Cultural Resource findings were due largely in part to a permitting process that indicated there were only a few known sites on the entire project. Knowing now that there were clear deficiencies in the process, there is a lessoned learned on how to improve investigations during the design. This would include additional investigations such as potholing along the levee that could expose midden soil, a common indicator of a cultural site. This work could be accomplished at large enough intervals to be both informative and cost effective.

© 2016 Deep Foundations Institute

Competent Key Changes

Changes to competent key were expected and anticipated for ranges up to ten (10) ft. However, the drastic changes that occurred on FRWL Project Area D were so substantial that the as-planned methodology had to change from DSM to conventional. With this significant change came an array of contractual discussions that could have been avoided.

During the design phase, the Agency performed subsurface investigations at approximately 1,000 ft. increments along the levee that determined the design depths of the cutoff wall. For the areas that had a design depth requiring DSM, verification drilling was to be performed by the Contractor at 100 ft. intervals that provides the Geotechnical Engineer with additional information necessary to then determine the depth of each DSM element. While the investigation intervals are reasonable, the verification drilling scope of work under future contracts should strongly be considered a design responsibility. By performing this work ahead of construction the design would be much more accurate and drastic depth changes, at least in the areas of DSM, would be eliminated.

Conclusion

Final completion of FRWL will bring 35 miles of levee along the Feather River into compliance with State and Federal standards for levees protecting urban areas. The slurry cutoff wall was the key design component that addressed the under and through seepage deficiencies of the existing levee system. The selected techniques for FRWL included 6,974,926 SF of convention excavation cutoff wall and 769,877 SF of DSM.

Although the project contained issues, FRWL was a historic undertaking of magnitude in an industry that is limited by its resources. The partnership formed between the Agency, Engineer, Construction Manager, Prime Contractor, and key subcontractors was paramount to completing the work meeting stringent quality control demands, safely, and on an aggressive schedule.

ACKNOWLEDGMENTS

The authors wish to acknowledge the following individuals for their contributions on the FRWL program:

Louay Owaidat, James Beebe, Sean Rhodes, Matthew Marks, Ezekiel Wilson, and Jon White.

REFERENCES

Recommended Practice for Field Testing Water based Drilling Fluids. API Recommended Practice 13B-1, Fourth Edition, 2009, 104 p.

Standard Test Method for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing. Designation D1140 – 14, ASTM International, 2014, 6 p.

Standard Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing. Designation C117 – 04, ASTM International, 2008, 3 p.

Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter. Designation D5084 – 10, ASTM International, 2010, 23 p.

Standard Test Method for Particle-Size Analysis of Soils. Designation D422 – 63, ASTM International, 2007, 8 p.

© 2016 Deep Foundations Institute

Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates. Designation C136 – 06, ASTM International, 2008, 5 p.

Sutter Butte Flood Control Agency | Reducing flood risk and protecting lives and livelihoods. (2010). *Sutterbutteflood.org*.

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

COCKATOO ISLAND SEAWALL SEEPAGE BARRIER – PERMEATION GROUTING, JET GROUTING AND A SOIL/CEMENT/BENTONITE MIXED SLURRY CAPPING TRENCH

Russell J. Denny, Wagstaff Piling Pty Ltd, Queensland, Australia, +61419031265, r.denny@wagstaffpiling.com.au

Chris L. Powell, Geotechnical Engineer, Cockatoo Island, Australia, +61429631235, Chris.Powell@outlook.com.au

ABSTRACT

The remotely located Cockatoo Island iron ore mine off the Kimberly coast in Western Australia, which has been in operation since 1951, was recently expanded up to 50m below sea-level using staged construction of a rock-filled seawall around the open cut mining operation. The most recent stage of the seawall expansion required a seepage reducing cut-off barrier to be installed through the seawall into underlying seabed sediments. A line of grout holes at 1m centres was proposed to form the seepage barrier, with grouting extending 17m below the top of the seawall. Top hammer drilling techniques were used to penetrate the 12m deep rockfill to seabed level, and permeation grouting carried out within the rockfill to fill voids and stabilise material around grout holes. The local 10m tides meant that tidal washout of fines and grout would be a major risk, and so a combination of cementitious and chemical grout was used to create rapid gel times. Following permeation grouting, holes were re-opened to allow jet grouting to hydraulically mix and grout permeable seabed sediments below the seawall. Using the duplex jet grout system in a single pass to 17m below platform level, the jet grouting was extended upwards into the seawall fill itself to overlap with the earlier permeation grouting. Finally a soil/cement mixed capping trench was installed using slurry trenching techniques with soil/cement/bentonite mixed backfill in order to complete the grout treatment. The approach taken allowed considerable onsite flexibility in technique. An automated slurry batching plant allowed adjustment of mixes to suit the varying subsurface conditions which were encountered during the works.

Keywords: Curtain Grouting, Permeation Grouting, Jet Grouting, Cement, Bentonite, Sodium Silicate

INTRODUCTION

Late in 2013 Wagstaff Piling was engaged to install a seepage barrier through a rockfilled seawall built to allow expansion of sub-sea iron ore mining operations on Cockatoo Island off the remote Kimberley coast in the North West of Western Australia. Earlier stages of seawall construction had used a variety of different construction techniques with varied success, and the history of sub-sea mining on the island had been plagued with failures, including failures in the slope of the seawall itself, hanging wall failure underneath and inside the seawall, and footwall failures as the ore was mined leaving the rock face inclined at 55 degrees, parallel to the bedding. The current expansion, known as the Stage 4 expansion, was more tightly constrained by the pit geometry and available materials than previous seawall expansion stages that adopted clay core and sheet pile seepage barrier solutions. In Stage 4, an innovative grout curtain seepage barrier solution was selected to ultimately provide the necessary cut-off to allow access to high grade ore, while protecting the mine from inundation by the sea.

© 2016 Deep Foundations Institute



Figure 1 - Cockatoo Island Iron Ore Mine

SEAWALL CONCEPT

Figure 1 indicates the arrangement of the open cut mine. The seawall concept can be seen in Figure 2. The open cut sub-sea mining operation consisted of a 3km long narrow pit along the South side of Cockatoo Island, extending up to 50m below sea level, and exposed to the open sea on the South side of the pit. Massive tidal variations of over 10.5m required a seawall founded on the seabed, and extending upwards to RL+13, to safely protect the pit from the high tide level rising to above RL+10.

GEOLOGY

The geology comprises the main ore body known formally as Seawall Hematite, dipping at 55 degrees towards the south into the seabed along the southern side of Cockatoo Island. The 30m thick high grade Hematite unit (68% Fe) was uplifted by regional folding to its current dip angle which is clearly visible from the air upon approach to the island. Commencing approximately 130m above sea level, earlier stages of open cut mining began in 1951 and had removed approximately 2km of accessible ore down to sea level by 1985. The formation underlying the ore body is known as Quarry Schist. The Elgee Siltstone unit directly overlies the ore body and consists of a highly weathered material. Above the Elgee Siltstone are a variety of materials including Hematite Scree, Marine Clay and a 20m thickness of Coralline Sediments, all overlain by thin layers of coral reef debris and sands to a maximum elevation of around RL0, approximately equal to the elevation of lowest tide.

EMBANKMENT CONSTRUCTION

The various stages of seawall are founded on the existing sea bed, effectively bearing on the very soft Coralline Sediments. Construction required strictly controlled stages of progressive placement of fill

© 2016 Deep Foundations Institute

material / loading, allowing for strength gain to occur in the weak Coralline Sediments as a result of pore pressure dissipation and consolidation. Stage 4 was constructed using an observational method making use of data being continuously collected from geotechnical instrumentation installed prior to critical stages of construction. Information collected contributed to a process of progressive assessment and stability modelling. An array of instrumentation and monitoring devices were used including multi-level vibrating wire piezometers, vertical and horizontal extensometers, inclinometers (SAAs), continuous automated laser surveying of field prisms and a slope stability radar system. Where appropriate, visual observations were also used to provide feedback during construction.

Seawall fill material comprised locally won/quarried quartzite and sandstone that was screened through a Grizzly screen deck with 300mm spacing between lateral slats. Some small bands of siltstone interbedded within the parent material meant that although the final product typically had less than 5% fines (sub 75um) by weight, the resulting fines content of blasted rock varied with each lot produced. The material was notoriously variable, non-homogeneous and anisotropic. Permeability characteristics of the fill material were therefore difficult to predict both laterally and vertically within rockfill placed to form the seawall embankment. Placement criteria during low tide windows also meant that fines would be washed out more predominantly in the lower sections of seawall, with massive movement of seawater across the construction zone during tidal cycles.

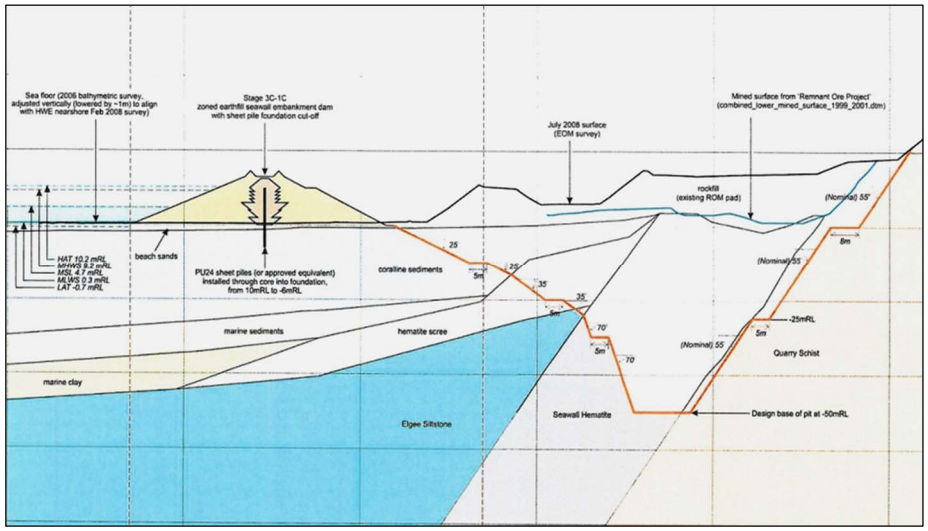


Figure 2 - Seawall Function

CUT OFF WALL SYSTEM SELECTION

The first seawalls were commenced in 2002, with stage 1. Stages 2 and 3 then followed and kept mining operations going until 2012. These earlier stages of seawall construction involved the placement of rockfill won from the local quarry and during mining activity, combined with low permeability clay core and filter zone material. Permeability barriers in Stages 1 & 2 made use of clay core while Stage 3 comprised a clay/fines core and also included a line of sheet piles driven through the centreline of the embankment and keying into underlying sediments. By the end of construction of Stage 3 much of the accessible low permeability core material had been exhausted on the island. Between 2003 and 2009, there were at least 3 major preventable failures associated the first 3 stages of seawall construction, all causing suspension of

© 2016 Deep Foundations Institute

mining activity and major recovery work. The Stage 4 expansion needed to take advantage of the previous lessons learned, and be designed and constructed so as to avoid similar issues leading to any repeat of the earlier failures. Using a grout injected cut-off barrier was recognised as a suitable solution without significant negative impact to embankment stability. The remote location of the site was also a major driver of system selection, 24 hours by barge from the port of Derby on the remote North West coast of Western Australia, 2400km by road from the Western Australian capital Perth.

The Stage 4 Seawall required 261m of seawall to be constructed. At the time of the Stage 4 expansion, clay fines were not readily available on the island. Plenty of overburden and quarry rock fill was available, but with variable fines content, and so a rockfilled seawall would again provide the structure and stability of the barrier wall, but not the water cut-off properties required. The Stage 4 seawall cut-off would need to extend from RL+12 down and into the Coralline Sediment aquiclude with a suitable founding level determined to be RL-5. Consideration of additional construction techniques to provide for water cut-off, were as follows;

- **Sheet piles** – Sheet pile installation would have required placement of fine material not easily produced on the island and also a relatively prolonged placement strategy requiring thin layers to be compacted during progressive raises. The confined geometry of the Stage 4 expansion would have also made it more difficult to provide a fines core during seawall raising especially where the embankment would tie into existing earthen structures. During seawall construction the raising of the wall by working from RL0 in low tide windows meant that fines placement would be very slow and difficult as the tide flooded the work area twice daily causing fines washout and softening.
- **Slurry Trench Cut-off Wall** – Similar to the sheet piles, large rocks in existing in situ rockfill would be likely to obstruct the passage of grabs. Also problematic for a slurry trench would be the loss of fluid during trenching operations, with highly permeable rockfill placed, and with fines washed out during the tidal cycles. Particularly at the lower levels in very short tidal timeslots, fines washout would be very difficult to prevent, and the potential for trench fluid loss would be difficult to assess or to control.
- **Grouting** – The idea of injection grouting was proposed, but the form of the grouting not developed. Injection grouting was presented as enabling faster construction times that would allow ore to be mined out of the pit more quickly. It also had the advantage of requiring light rigs and a batch plant, so relatively easy to mobilise, with ongoing raw material supply feasible by road and then barge to the island. The grouting method was developed from earlier work done by Wagstaff Piling on a rockfill seawall in Gladstone, Queensland. In that case a rockfill seawall with limited fines was successfully grouted to create a seepage barrier using rapid gelling chemical grout to fill voids, thereby preventing dredged fines from washing out through the seawall on tidal cycles.

TECHNICAL PROPOSAL AT THE OUTSET - GROUTING

A significant advantage of grouting was that the seawall could be raised simply without the need of fines or clay core material, and then a grout curtain barrier injected centrally through the rockfill would provide sufficient seepage cut-off. Grouting was proposed to be done along the wall alignment using a combination of techniques, as follows;

© 2016 Deep Foundations Institute

Stage 1 - Permeation Grouting; 0 – 10m Depth, Upper Seawall Fill Materials;
A top hammer drilling rig, using percussion drilling techniques, would drill through the rockfill with fluid flushing techniques, and then grout using a rapid gelling chemical and cement grout combination. The grout would be placed through the rockfill down to the seabed level. Holes were initially at 4m, then reducing to 2m centres, and finally infilling at 1m centres, drilled by top hammering through the seawall fill to 10m depth, using a grout formulation of bentonite, cement and chemical (sodium silicate).

Stage 2 - Jet Grouting; 10m–17m depth, Lower Seawall Hematite Sands and Coralline Sediments;
Jet grouting was proposed in the coralline sediments underneath the permeation grouted upper seawall fill, utilizing the passage of the holes already drilled for the purpose of permeation grouting. Jet Grouting would follow on through the same drill holes, and be focused below the seabed to provide the cut-off through the permeable coral reef debris and into the coralline clay. Jet grouting was proposed to be done using a duplex system, with fluid grout and air simultaneously injected, and being a soft soil technique, required the pilot holes formed and stabilized through the rockfill by the 1st stage of top hammer drilling and permeation grouting.

After commencement of works on site with initial production trials, the subsurface conditions would more precisely determine the grouting methods, with a range of possible grouting techniques available. The 2 techniques would utilise the same batch plant, and would provide considerable flexibility in the approach. The major material component would be cement, which would be provided in 22tonne bulk containers (ISO's). Other materials required in much less quantity, included bentonite, which would be supplied in bulker bags, and Chemical (Sodium Silicate) also supplied in 22tonne bulk ISO's. The spacing of injection holes was estimated to be nominally 1m centres, to be done in a sequence of primary / secondary injection, with initial injection holes (primary holes) at 4m centres.

TECHNICAL SOLUTION – EXECUTION OF THE GROUTING WORKS

The conditions encountered proved to be extremely variable, and required constant monitoring and change of technique. The key techniques found to be required during construction are described as follows;

Stage 1 - Permeation Grouting; 0 – 10m Depth, Upper Seawall Rockfill Material;
Top hammer drilling proved to be similar to that anticipated in the area of the new seawall construction, although large portions of the grout alignment turned out to be located over tapering transitions to existing seawalls. In these transitions, large boulders and possible waste debris formed obstructions which made top hammering through the fill much more challenging. The volume of grout taken in each hole was extremely variable, with drill hole circulation to surface the best indicator of permeable zones, porosity, cavities and the like. Recording grout volume taken at each hole position and over the profile of the depth of each hole, served to provide evidence that cavities were firstly in existence, and secondly, that they were being filled. As infill secondary holes were completed, grout volumes taken would progressively reduce in zones initially identified as highly permeable.

Hole stability was difficult to achieve in the often loose rockfill, and many top hammered holes needed to be drilled and grouted a second time to create the passage for jet grouting. The rapid gelling properties of the Sodium Silicate and Cement grout combination, were essential to even achieve a collared and stabilized

© 2016 Deep Foundations Institute

hole through the rockfill, especially with high volumes of tidal seawater flowing in and out continuously as the tides changed, and more particularly on the spring tides where a 10m tidal variation would occur in a 6 hour period. This equated to a tidal rise and fall approaching 30mm per minute.

Stage 2 - Jet Grouting; 10m–17m depth, Lower Seawall Sands and Coralline Sediments;
A stabilized and collared hole created by the Stage 1 drilling and grouting proved to be essential and was a much more difficult task than anticipated at the outset. Jet Grouting would follow on through the collared drill holes, and was initially focused below the seabed to provide the cut-off through the highly permeable coral reef debris and into the Coralline Sediments.

Jet grouting proved to be very effective in the sediments underneath the upper seawall rockfill, and this was seen in the volume and nature of the up hole circulation (returning fluid) during jet grouting. For example, as initial holes were drilled at 4m centres, limited uphole returning volume would be seen, however as progressive infill of holes occurred (2m infills and then 1m infills), uphole volume would gradually increase by proportion according to the proximity of existing grout columns. This effect provided some confidence in the effectiveness of the grouting.

Permeation grouting alone proved not to be sufficient to provide an effective permeability barrier and therefore it was decided to extend jet grouting upwards through the seawall rockfill to within 2 to 3m below the working surface. The returning volumes up through the seawall fill over successive phases of grouting provided similar initial indications / confidence of filling of void spaces and of achieving continuity between grout columns at depth.

Soil/Cement/Bentonite Capping Trench

Excavation to inspect the tops of grouted columns revealed that grout effectiveness close to the platform was diminished, and this was anticipated due to the reducing overburden pressure up high just beneath the working surface. Utilising the batch plant and site excavators, a trench was cut from surface down as far as 3m to meet the tops of the columns, with cement/bentonite grout filled to about 1m depth, and the fill with larger rock particles screened out, then finer material returned to trench and bucket mixed with the excavator to combine in a slurry of cement, bentonite and rockfill. By observation this capping trench proved to be a simple method to provide a cut-off over the upper 3m, where continuity of jet grouting was found to be difficult to achieve.

LOGISTICS

Supply logistics were critical in the remote location. Limited barges, tight scheduling due to tides, and other general freight deliveries to the island mine, meant that materials supply dictated progress on site. Ultimately the quantity of materials required (predominantly cement) determined the progress and duration of the works, given the barging availability and capacity. At times the freight choices and project progress was determined simply by comparing the urgency for cement versus the urgency for other supplies to sustain the workforce.

Cement - The predominant material required was cement, with 4000tonne required over the 10 month duration of works, equating to 20 tonne per day. Cement was supplied by ship from China to Port Hedland, and then road freighted in 22 tonne bulk ISO's, 800km to the port of Derby. A total of 18 no. ISO's were

© 2016 Deep Foundations Institute

used in constant circulation. ISO's were loaded onto trailers by container forks, rolled on and off the barges, and then the cement blown into silo's at the seawall batch plant immediately adjacent to the works. The return of empty ISO's was then just as critical in the timing and feasibility, to maintain a workable long term supply chain.

Bentonite – Approximately 200 tonne of Sodium Bentonite was supplied in 1.2 tonne bulker bags, delivered by road from various sources, but primarily ex-mine in central Queensland, by road some 4,500km across the top of Northern Australia to Derby port, and then by barge to site. The bulker bags were fed into an upright 10 tonne bulker bag silo.

Sodium Silicate – Approximately 240 tonne of liquid Sodium Silicate was used, and this was supplied in 20 tonne bulk liquid containers. The sodium silicate was manufactured in Perth, and road freighted to Derby, then transferred to trailer and barged to the site. Averaging one bulk container per month, consumption was difficult to predict, especially at the closing stages of the grouting works.

MIX DESIGN AND INSTALLATION PARAMETERS

Permeation Grout – We refer to Figure 3, which shows the way the cement, bentonite and sodium silicate chemical was combined. Firstly bentonite and water were combined and transferred to a holding tank for hydration. Part A and Part B of the final mix were then prepared and held in separate circulation tanks, as follows;

- Part A - Sodium Silicate (1 part by volume) was added to hydrated bentonite/water (9 parts). This mix was not yet reactive, so could be held for long periods, including overnight, etc.
- Part B - Cement and water was then mixed at w/c=0.75, and combined with the hydrated bentonite/water mix in equal parts by volume. This mix was prepared just prior to use.

	Bentonite & Water			
	B+W	kg	SG	Litres
	B	50	2.2	22.7
	W	977.3	1	977.3
				1000.0

	Cement & Water			
	W/C=0.75	kg	SG	Litres
	C	936	3.15	297.1
	W	702.0	1	702.0
				1000.0

	Part A										Part A + B combined by flow meters			
	Combine by vol. 10% SS + 90% Bentonite					Factored back to 1000L					Q [L/min]	100		
	B+W+SS	kg	SG	Litres	factor	B+W	kg	SG	Litres	factor	B+W	kg	SG	Litres
	B	50	2.2	22.7	0.900	B	45.0	2.2	20.5	0.5	B	22.5	2.2	10.2
	W	977.3	1	977.3		W	879.6	1	879.6		W	439.8	1	439.8
90%				1000.0										
10%	SS	160.0	1.45	111.1		SS	144.0	1.45	99.3		SS	72.0	1.45	49.7
			Sum Vol.	1111.1					999.3					499.7

	Part B													
	Combine by vol. 50% SS + 50% Bentonite					Factored back to 1000L					Q [L/min]	100		
	C+B+w	kg	SG	Litres	factor	B+W	kg	SG	Litres	factor	B+W	kg	SG	Litres
	B	50	2.2	22.7	0.500	B	25.0	2.2	11.4	0.5	B	12.5	2.2	5.7
	Wb	977.3	1	977.3		Wb	488.6	1	488.6		Wb	244.3	1	244.3
50%				1000.0										
	C	936	3.15	297.1		C	468.0	3.15	148.6		C	234.0	3.15	74.3
	Wc	703.0	1	703.0		Wc	351.5	1	351.5		Wc	175.7	1	175.7
50%				1000.1					1000.0					500.0
			Sum Vol.	2000.2									Sum Vol.	999.7

Figure 3 - Permeation Grout Mix Design

© 2016 Deep Foundations Institute

Using parallel plunger pumps of equal performance, the Part A and Part B mixes were combined in equal parts at equal flow rate through a "Y" fitting just behind the drill rig, at 5 to 10 Bar pressure subject to depth and length of delivery lines. Once combined through the "y" piece, the grout would be reactive and subject to gel within 10 to 20 seconds after exiting the drill string and losing the velocity in the drill pipe. The thixotropic nature of this mixture would ensure pumpability, but would lead to gel outside the drill bit as the velocity reduced. The speed of the gel could be varied by varying the components Part A and Part B. For example if there was a delay in proceedings, Part B could be flushed through the lines without Part A the other, to prevent gelling of the mixture in the lines.

The top hammer rig used was a Hutte 605, with percussion drilling bits, drilled by flushing the fluid grout mixture. Drilling rods were difficult to maintain in the aggressive conditions, with a variety of rod types used, the most successful rods being T51 Guide Tubes, which were a stiffened version of the T51 percussion rods.

Jet Grout – The injected mix comprised a combination of hydrated bentonite with cement and water grout, w/c=0.98 with cement 763kg/m3 and bentonite 15kg/m3. Observation of drill hole returns indicated the inclusion of variable amounts of fines in the mix including rockfill fines, hematite sands, and coral reef debris, but in all cases the returning mixture had sufficient cement content to easily set overnight, with significantly more strength than necessary.

Jet grout was installed using the duplex system, with air at 8 to 10 Bar pressure and grout at 300Bar pressure. Typical installation method and parameters were by flushing downwards at low pressure (no jetting) and then jetting upwards at 300Bar, with approximate flow Q=230L/minute, an upwards velocity of 25cm/minute, and rotational velocity of 6 to 8 RPM. The jetting nozzles used were 2 x 3.5mm diameter, typical targeted jetting energy E=40 MJ/m, the aim was to achieve a column diameter in the order of 1.2m to 1.5m.

Slurry Capping Trench – The slurry mix used comprised bentonite 25kg/m3, cement 468kg/m3 and water 840Litres. When rockfill fines were added, the final mix comprised bentonite 8.3kg/m3, cement 156kg/m3, water 280Litres/m3, and rockfill fines.

VERIFICATION

Verification of the success of the grouting proved to be difficult, with a variety of methods attempted including falling/rising head testing, excavation of observation trenches, diamond core drilling and water level monitoring. The most successful methods of verification are described below;

Piezometers - The strategic use of standpipe piezometers placed along the inside of the grout alignment to the depth of the base of rockfill proved to be a reliable verification technique. The standpipes were installed at the start of the grouting process at approximate 10m intervals and offset from the landward side of the grout alignment by between 3m to 10m. Vibrating wire piezometer sensors were installed into the standpipes and attached to data loggers at the surface recording water levels continuously at 20 minute intervals. The data loggers were then programmed to send data via telemetry to a central computer which could be accessed remotely over the internet.

As the grout curtain seepage barrier was progressively closed groundwater levels on the landward side of the seepage barrier were compared to baseline readings and tidal variations on the seaward side.

© 2016 Deep Foundations Institute

Groundwater levels on the landward side of the seepage barrier were observed to diminish significantly as grouting progressed to achieve 1m centres. Sensor readings indicated initial coupling of groundwater and tide levels however de-coupling (or significant reduction) was observed as work progressed. Figure 4 shows a typical series of piezometer readings over a three week period of the tidal cycle, i.e. spring tide to spring tide. In the snapshot shown in the figure, the second cycle of spring high tidal movement shows significant changes in groundwater behaviour, including delayed response to tidal rise and fall, and a progressive reduction in the magnitude of the rise and fall of groundwater within certain standpipes as the grout curtain continued to be closed.

Uphole Circulation - Figure 5 is a tabulation of the behaviour of the fluid returning during jet grouting operations. Primary holes were grouted at 4m centres, and it was noticed that during grouting of secondary infill holes at 2m centres, and more noticeably while grouting 1m (tertiary) infill holes, that progressively more fluid would return to surface. This was seen as an indication of the progressive success of the grouting, and although not strictly occurring every time, there was a distinct pattern of behaviour that would not only indicate success, but would also highlight porous areas, which likely corresponded to zones where variations in seawall fill existed.

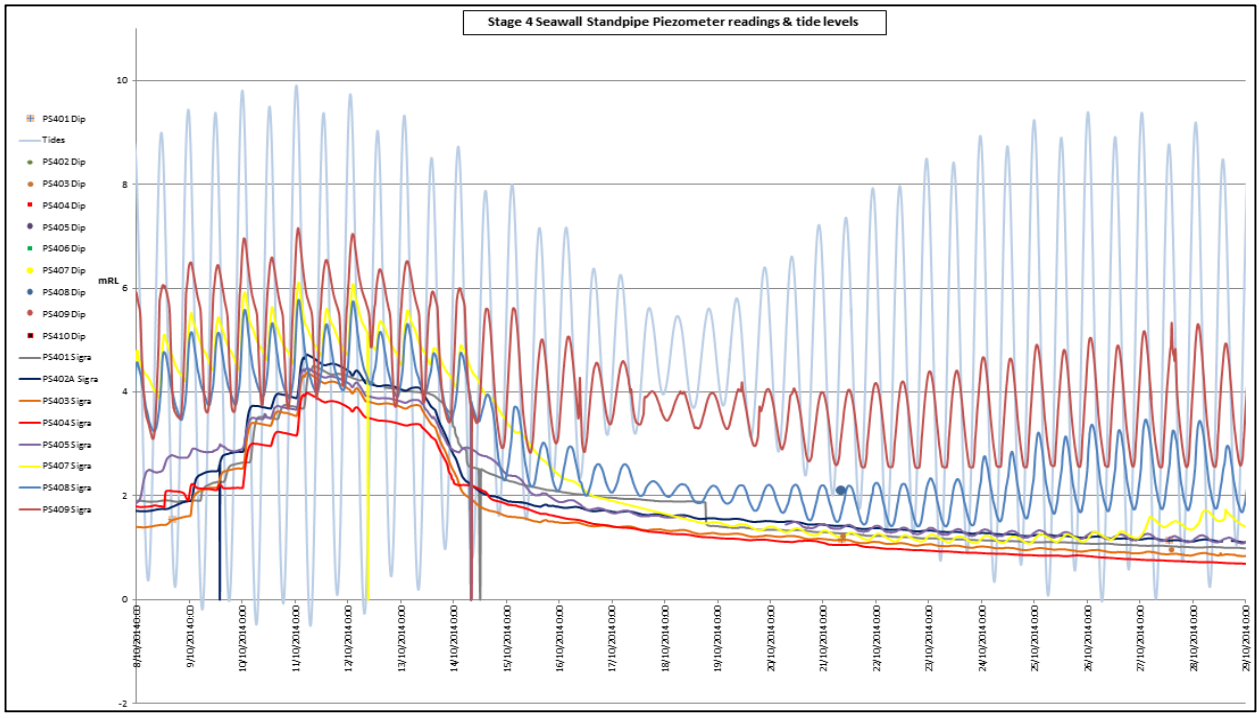


Figure 4 - Piezometer Monitoring

Comprehensive assessment – A combination of available data arising from piezometers, drilling / grouting records, logged diamond drill core through the grout centreline and variable head testing was assessed using weighted distributions. In order to undertake the evaluation the data was normalised and then combined using a single format to provide an indication of the effectiveness of the seepage barrier along the centreline and at depth across the alignment. The highest weighting was given to standpipe readings.

Final Spring Tide Testing - More convincing verification was by observation of performance inside the seawall on the first king tide after the nominal closure of the grout curtain. At this stage, grouting plant and

© 2016 Deep Foundations Institute

equipment was maintained on location in standby mode, in the event that additional grouting was required. The performance during the king tide testing was considered suitable and fit for purpose by the client, and the order to demobilise plant and equipment given. In the following months, with the seepage barrier in place, mining operations were then able to safely access the Seawall Hematite ore body and progress with drilling, blasting and excavation below sea level (refer Figure 6).

Column No.	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68
Date	26-Jun	1-Aug	28-Jun	3-Aug	26-Jun	1-Aug	28-Jun	15-Aug	27-Jun	3-Aug	28-Jun	15-Jun	27-Jun	14-Aug	29-Jun	16-Aug	27/06/14	14/08/14	29/06/14	21/08/14	27/06/14	22/08/14	21/06/14
Prim / Sec / Tert	prim	Tert	sec	Tert	prim	Tert	sec	Tert	prim	Tert	sec	Tert	prim	Tert	sec	Tert	prim	tert	sec	tert	prim/ sec	tert	prim
Tide at start																	high	low	low/ med	med	med	high	med
Tide at End																	high	low	low/ med	med	low		low
Depth																							
1	H	H	H	H	M	H	H	M	H	H	M	M	H	M	L	M	M/H	M	M	H	H	M	H
2	H	M	M	H	M	H	H	M	H	H	M	M	H	M	M	M	M/H	M	M	H	H	M	H
3	M	M	M	H	M	M	M	M	H	H	L	M	M	M	L	M	M	M	M	H	M	M	H
4	M	M	M	M	L	M	M	L	M	M	M	M	M	M	M	M	L	M	H	H	M	N	H
5	N	H	M	N	L	M	M	L	M	M	M	M	M	M	M	M	N	L	M	H	H	M	H
6	N	M	L	L	M	M	L	L	L	M	L	M	M	M	L	M	L	L	M	H	M	M	M
7	H	M	L	L	M	M	L	M	L	M	M	M	M	M	M	M	L	L	L	H	M	M	M
8	L	M	L	L	L	M	L	M	L	L	L	M	L	H	L	L	L	L	L	M	L	L	N
9	N	M	M	L	M	M	M	M	L	L	L	M	L	H	L	M	L	M	L	M	L	L	N
10	L	M	L	L	M	H	M	M	L	L	L	M	N	L	L	M	L	M	L	L	L	M	L
11	N	L	L	M	N	H	L	M	L	L	L	M	L	L	L	M	L	M	L	L	L	M	N
12	N	L	L	M	L	H	N	L	N	L	L	L	N	L	N	M	L	M	L	L	L/N	L	N
13	N	N	N	M	L	H	N	L	N	L	N	L	L	L	N	M	L	M	N	L	L/N	L	N
14	N	L	L	M	N	H	L	L	N	L	N	L	L	L	N	M	L	L	N	L	L/N	L	L
15	L	N	N	L	N	M	L	N	N	L	N	L	N	N	L	L	L	L	N	L	L/N	N	N
16														L				L					N
17														L				L					N

Figure 5 - Jet Grout Profiles - Monitoring of returning Fluid Volumes

DISCUSSION AND CONCLUSIONS

1. Permeation grouting effectiveness was extremely variable. In transition zones adjacent to existing earthworks containing uncontrolled fill, large volumes of rapid gelling grout were required, but in other zones of controlled fill, grout volumes were lower. Particularly in the areas where up hole circulation was maintained, the profile obviously contained sufficient fines to prevent grout penetration, and in these cases jet grouting the profile would be more effective.
2. The jet grouting duplex drill strings were found to be sensitive to rockfill obstructions leading to critical damage of the smaller diameter inner high pressure grout passages. The use of rapid gelling chemical grout, combined with top hammer percussion drilling through the seawall rockfill, proved to be an essential method of collaring the holes i.e. opening up and stabilising the rockfill, which in turn would allow the passage of the jet grouting drill string. Without the rapid gelling cementitious grout, holes would simply collapse and prevent the passage of the jet grout drill string.
3. Jet grouting, although normally a technique suited to soft / loose soils without large particle sizes, was proven by the successful end result, to be an effective way to grout both the sediments underlying the seawall and also the seawall fill itself following the initial permeation grouting. It was however recognised that the large maximum particle size of the rockfill introduced issues that would likely have been circumvented if a smaller maximum particle size rockfill had been used.

© 2016 Deep Foundations Institute

4. Tidal water movement was a constant factor and more pronounced during spring tide variations. Jet grouting therefore, without rapid set or rapid gel chemical added, needed to be carefully monitored for effectiveness during periods with high water movements. This aspect was also a concern during wall closure, where the potential for funnelling of water through closing sections might lead to tidal washout. The risk of dilution and washout was sufficient reason to maintain relatively high cement contents during jet grouting, so as to promote more rapid set and to buffer against dilution in saturated permeable zones where subterranean tidal flow paths may exist.
5. The grout curtain solution was a progressive and relatively adventurous proposal, but with the excellent co-operation of all interested parties over a 12 month duration, the result was proven to sufficiently reduce and control the seepage through the seawall, which allowed the safe advancement of the Stage 4 pit below sea level.



Figure 6 - Mining the Hanging Wall under the Stage 4 Seawall and Seepage Barrier

DEFINITIONS

Footwall – The 55 degree exposed face of Quarry Schist that followed natural bedding angle as mining progressed downwards.

Hanging Wall – The Elgee Siltstone over the Hematite, exposed on the seaward side of the sub-sea pit, as the pit progressed downwards.

Sodium Silicate – N grade, Na^4SiO^4

Cement – Ordinary Portland Cement.

© 2016 Deep Foundations Institute

Hematite – Ore comprising naturally occurring high grade iron oxide grading at 68% Fe locally.
ISO – Bulk containers (20 to 24tonne) for supply of cement and Sodium Silicate.
Spring Tide – High tides which occur on lunar cycles at full moon and new moon.

ACKNOWLEDGEMENTS

The author would like to acknowledge the co-operation, support and contribution throughout the project, of all interested parties including Watpac Civil and Mining, Pluton Resources, Coffey Mining, and other Wagstaff Piling project team members. The comments and views in the paper are those of the authors and not necessarily those of other parties.

REFERENCES

1. CIO Seawall Presentation, Overview of Scheme, Watpac, 15/07/13.
2. Construction Method Statement, Cockatoo Island Operation Stage 4 Seawall Embankment Construction, Watpac, 15/07/13.
3. CIO Seawall Presentation, Cockatoo Island Stage 4 Seawall Presentation, Coffey, 13/06/13
4. Cockatoo Island Operations, MINEWPER00062AS, Stage 4 Seawall Investigation, Site Geology, 23/12/11.

© 2016 Deep Foundations Institute

CONSTRUCTION OF THE TAILINGS STORAGE FACILITY FRESHET MANAGEMENT EMBANKMENT CUT OFF WALL AT MOUNT POLLEY MINE

Keith MacKay, Golder Associates, Vancouver, BC, Canada, 604 345 5797, KMacKay@golder.com
David Siddle, Golder Associates, Vancouver, BC, Canada, 604 296 2893, DSiddle@golder.com
Luke Moger, Mount Polley Mining Corporation, Likely, BC, Canada, 250 790 2215, LMoger@MountPolley.com
Terry Eldridge, Golder Associates, Vancouver, BC, Canada, 604 296 4200, TEldridge@golder.com

ABSTRACT

On August 4th, 2014, a breach occurred in the Perimeter Embankment of the Tailings Storage Facility (TSF) located at the Mount Polley Mine, releasing approximately 10.6 million m^3 of supernatant water, 7.3 million m^3 of tailings solids, 6.5 million m^3 of tailings interstitial water and 0.6 million m^3 of construction materials. A Temporary Upstream Rockfill Berm was constructed between August 2014 and February 2015 within the area of the tailings dam failure in order to retain the remaining tailings solids during the low flows of the winter season. The Temporary Upstream Rockfill Berm was constructed as part of the initial response phase, with the primary objective to abate the flow of tailings into the receiving environment. A secondary objective of its construction was facilitating the required investigation works downstream of the tailings dam failure. The selection of the coarse grained material used in the construction of the Temporary Upstream Rockfill Berm was based on the availability of the material (timeliness of construction) and the non-water retaining design of this initial response phase. Consequently, an alternative method was required in order to prevent further loss of mine-affected water and tailings from the TSF, as well as to provide short-term contingency water storage, during high flow conditions expected in late spring. Golder Associates Ltd. designed the 2015 Freshet Management Embankment, a deep mixing cut-off wall, to meet these site requirements, which was constructed by Mount Polley Mining Corporation and Golder Construction Inc. between December 2015 and April 2015. The works included 132 soil cement panels constructed to depths of up to 27 meters. This aspect of the works was completed using an RG25S Bauer CSM rig on a 24 hour per day, 7 day per week schedule. The cut-off wall specifications included a target unconfined compressional strength (UCS) of between 1 and 2 MPa, a maximum hydraulic conductivity of $1x10^{-6}$ cm/s, and a ductility to enable the soil cement to undergo a minimum of 5% strain without a reduction in strength or increase in hydraulic conductivity. In order to control and assure the quality of the soil cement, 2553 wet grab specimens were retrieved from the fresh soil cement panels using a project-developed sampler. Four of the soil cement panels were cored with a mud-rotary drill rig and optical and acoustic televiewers were placed into the boreholes in order to log several relevant parameters, including image, texture, relative hardness, borehole-shape, and temperature.

Keywords: Cutter Soil Mixing, Seepage Control, Deep Mixing, Dam Remediation, Quality Control and Assurance

INTRODUCTION

Golder Associates Ltd. designed the 2015 Freshet Management Embankment, a deep soil mixed cut-off wall, downstream of the Temporary Upstream Rockfill Berm as part of measures to prevent the further loss of mine-affected water and tailings from the Tailings Storage Facility (TSF) and provide short-term contingency water storage required prior to the 2015 freshet. The cut-off wall structure was constructed by Mount Polley Mining Corporation (MPMC) and Golder Construction Inc., with the latter party retained by MPMC for the completion of the cut-off wall panels. The cut-off wall was constructed using Cutter Soil Mixing (CSM) technology. CSM is a form of the deep mixing method consisting of two cutter wheels rotating about a horizontal axis, penetrating the ground while cement and/or bentonite slurry is injected from a nozzle located between the two wheels. In this application, CSM was used to cut through placed

© 2016 Deep Foundations Institute

aggregate materials, inject bentonite and then cement slurries, and mix the resulting combination into a soil cement material. In this application, panels were required to have tie-in to natural soils.

SITE LAYOUT AND LOCATION

The Mount Polley Mine is located near the eastern edge of the Fraser Plateau physiographic subdivision, approximately 10 kilometres west of its boundary with the Quesnel Highland subdivision.. The 2015 Freshet Management Embankment construction was completed at the TSF, which is located to the southwest of Polley Lake and to the west of Quesnel Lake as shown in Figure 1 and 2.



Figure 1: A view of the site prior to tailings dam breach. (Google Earth 2016)

© 2016 Deep Foundations Institute



Figure 2: A view of the site following tailings dam breach. (Google Earth, 2016)

Following the initial response phase to the tailings dam failure in August 2014 (including construction of the Temporary Upstream Rockfill Dam), a geotechnical site investigation was undertaken and the 2015 Freshet Management Embankment design (cut-off wall construction) completed and permitted. The design included a cut-off wall, which consisted of 132 soil cement panels following an alignment downstream of the Temporary Upstream Rockfill Berm and tie-in to the existing dam structure. The design (and subsequent construction) included exposing foundation till in the area of the failure and footprint of the 2015 Freshet Management Embankment as well as the areas requiring tie-in with the existing dam structure (i.e., along each of the North Abutment and South Abutment). Once suitable foundation and tie-in areas were exposed, inspected, and approved, surface preparation was completed and was followed by placement of the zoned construction materials (including aggregates, downstream rock filters and rock buttressing). The cut-off wall panels were constructed through the aggregate material, once placed to its final design height.

SITE GEOLOGY, GROUND CONDITIONS & CUT-OFF WALL ALIGNMENT

A sandy gravel material, referred to as Cut-Off Wall Aggregate, was placed along the alignment of the proposed cut-off wall. The design parameters of the 2015 Freshet Management Embankment required the majority of the Cut-Off Wall Aggregate to be compacted in lifts during construction; however, the cut-off wall alignment was left un-compacted for 2 meters on either side, so as not to impede the penetration of the CSM. The design of the cut-off wall called for it to be 1m in thickness along the alignment and installed through the Cut-Off Wall Aggregate and 1.5 meters into the approved foundation surface as depicted in Figure 3. Piezometers were installed inside the Temporary Upstream Rockfill Berm to monitor the groundwater elevation, and sump pumps were installed and could control groundwater elevation through the Temporary Upstream Rockfill Berm and 2015 Freshet Management Embankment as necessary. At the north and south limits of the 2015 Freshet Management Embankment alignment, portions of the cut-off wall were to be constructed within the North Abutment and South Abutment (areas of the TSF remaining from pre-tailings dam failure). The alignment was chosen so that the cut-off wall would tie in with the existing clay-till core of the abutments, and to avoid areas where coarser grained units (which could potentially impede the penetration of the CSM rig) were expected to be present.

© 2016 Deep Foundations Institute

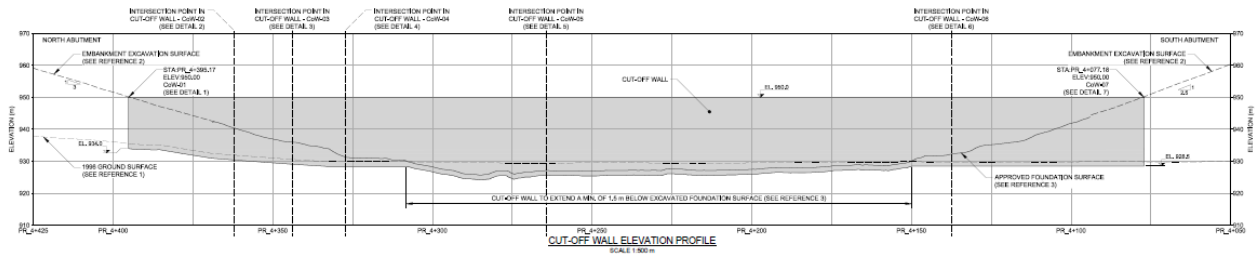


Figure 3: Profile view showing cut-off wall embedded into foundation glaciall till and TSF abutments

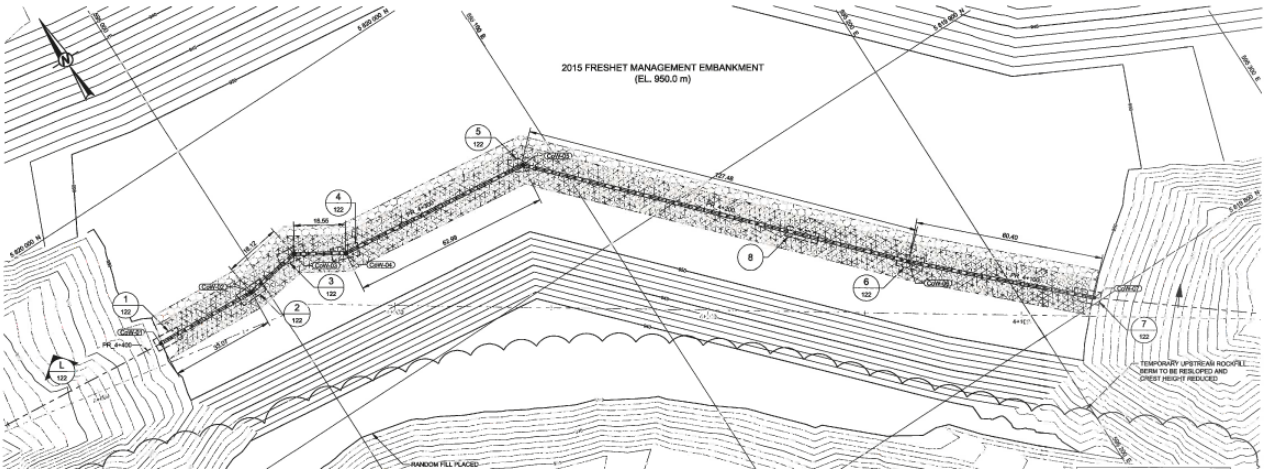


Figure 4: Plan view showing cut-off wall alignment of 2015 Freshet Management Embankment.

LABORATORY AND FIELD TRIALS

A laboratory trial was conducted in order to determine if an appropriate cut-off wall material could be constructed using the deep mixing method. Samples of the foundation till and Cut-Off Wall Aggregate were mixed into 9 different batches with varying cement and bentonite contents, and re-constituted into Soil Cement specimens. Samples underwent UCS testing, hydraulic conductivity testing, and Consolidated Undrained (CU) Triaxial testing. UCS test results are displayed in Figure 5 below. After the results of the laboratory mix trial were analyzed, a mix design was chosen for use during full-scale production. A blend of Ground Granulated Blast Slag (GGBS) and Portland Cement (PC) was chosen as a binder, and bentonite slurry was chosen as a cutting fluid. Typically, deep mixing projects include a field trial component, where mix designs can be assessed versus the in-situ native ground conditions. Unlike most deep mixing projects, the majority of the soil cement material used in the 2015 Freshet Management Embankment was to be constructed from Cut-Off Wall Aggregate of known properties; reducing uncertainty in the properties of the final soil cement material. As such, and considering the critical timeline of the project, it was decided to choose a production mix design based on the results of the laboratory mix trial, and go into full-scale production as soon as placement of the necessary construction materials was complete. Although the results of a field trial were not assessed prior to going into full production, 3-day and 7-day UCS test results from the first panels installed were closely monitored, and details of the production mix design were fine-tuned and optimized during construction. Ultimately, a two-phase mix was chosen for production panels, cutting down with bentonite slurry to a bentonite content of approximately 30 kg/m^3 and filling up with cement slurry to a cement content of approximately 100 kg/m^3. Slight adjustments were made to the production mix design based on Quality Assurance (QA)/Quality Control (QC) results received during production.

© 2016 Deep Foundations Institute

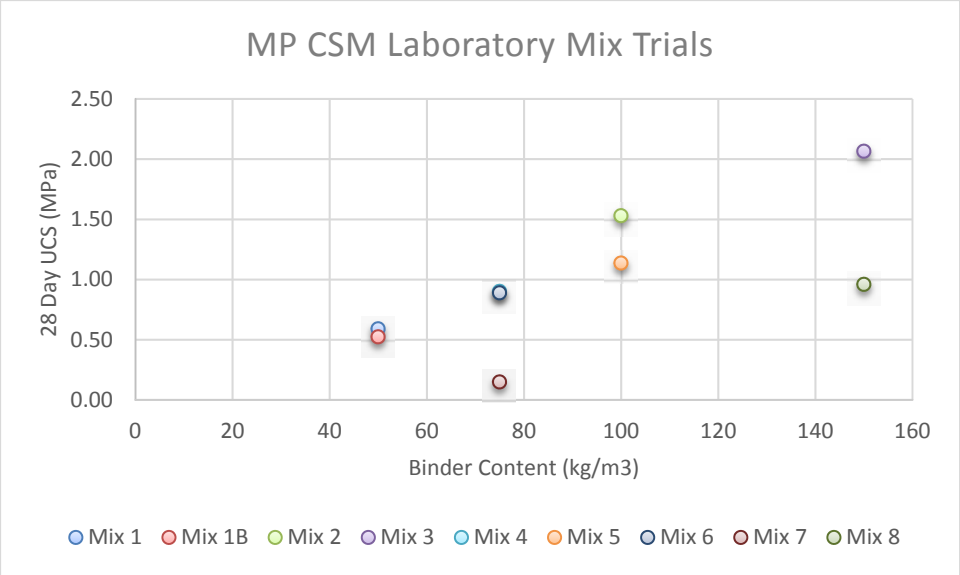


Figure 5: Laboratory Mix Trial results, 28-Day UCS versus Binder Content.

EQUIPMENT AND LOGISTICS

A Bauer RG25S CSM rig was mobilized to site for soil cement panel installation. The project was scheduled for late winter, and below freezing temperatures were expected during this period. Measures had to be taken to ensure that production would not be hindered by freezing conditions; including heat tracing and insulation for all exposed slurry lines, construction of a large tent to shelter the slurry batching and storage area, and a portable shelter along the alignment to provide shelter for ground staff working at the front end of the CSM rig. A high-shear venturi mixer was brought to site in order to combine the bentonite with water and ensure it was properly sheared prior to hydration. Two 80,000 liter (L) tanks with agitators were placed on site to store the bentonite while it was being hydrated. A MAT SCC 30K batchplant was used to batch cement slurry, and could produce over 30,000 L of slurry per hour at full production speed. The batch plant was fed by two cement silos, one at 90 mT capacity and one at 50 mT capacity to store the PC – GGBS blend which was used to form the cement slurry. Cement was delivered by B-Train truck from Seattle, Washington and a two to three-day journey was required before arrival at site. Generally, a truckload of cement would be consumed during each 24-hour production shift. Enough storage to house three truckloads of cement was required in the silos at all times, in order to store the cement enroute in the case of an unexpected shutdown. The batch plant fed into two agitator tanks which could each store over 5,000 L of cement slurry prior to it being injected into the ground during CSM production. Due to the setup time of the cement slurry, batching would start only once the CSM panel penetration had reached a certain depth. CSM panels were injected with bentonite slurry while cutting down to target depth, a re-stroke of the cutter was conducted, and then cement slurry was injected into the panels as the cutterhead retracted from the panel, mixing the wet soil cement mixture as it rose to surface.

It was identified at the start of the project, that the coarse gravel contained in the Cut-Off Wall Aggregate could cause significant degradation to the CSM cutter wheels, and hence could induce significant production delays if left un-checked panel after panel. In order to reduce potential downtime caused by wear and tear, carbide studs attached to a ceramic ferrulea were welded onto the cutter wheels. Another measure taken to prevent un-planned downtime was the utilization of a three shift system. Each day there would be a 10-hour day shift and 10-hour night shift of CSM production, followed by a 4-hour shift of planned preventative maintenance; in which hard-facing would be applied to the cutter wheels, carbide studs would be installed, fins and various degraded parts would be replaced, and additional maintenance would be conducted on the equipment as necessary. The system resulted in greater overall production, and very little in the way of unplanned downtime during the 20 hours of CSM production each day. A picture of the RG25S drill rig directly prior to entering a panel is visible in Figure 6.

© 2016 Deep Foundations Institute



Figure 6: BAUER RG25S cutter wheels entering a panel with South abutment in the background.

TERMINATION CRITERIA AND AUGER PRE-DRILLING

The Cut-Off Wall Aggregate located in the alignment was found to be quite difficult to penetrate, and the RG25S could only reach approximately 12 meters depth before meeting the advancement rate based termination criteria defined in the project design technical specifications. To penetrate deeper than this depth in the Cut-Off Wall Aggregate material, a BAUER BG-18 auger rig was mobilized to site. While mobilizing the pre-drill rig to site, panel construction continued on the north and south areas of the alignment, where Cut-Off Wall Aggregate depths were lesser, and the RG25S could reach the target panel termination elevation without reaching this advancement rate based termination criteria. Once the BG-18 arrived on site, several different pre-drill hole configurations were trialed in order to determine which would be the most effective to increase the CSM penetration productivity. The most effective pre-drilling configuration was found to be drilling a 1150 mm diameter auger hole at the center of each panel location down to the bottom elevation of the Cut-Off Wall Aggregate, filling the hole with approximately 8000 L of bentonite slurry, and then backfilling the hole with the Cut-Off Wall Aggregate spoils previously removed. The volume of bentonite slurry added during pre-drilling was subtracted from the volume to be added during CSM panel penetration.

QUALITY CONTROL AND QUALITY ASSURANCE

Many stages of QA/QC were necessary during completion of the project. A detailed Inspection and Test Plan was created, summarizing all of the quality steps for the soil mixing process. Water samples were retained from the water source on site and tested for contaminants which could potentially affect the curing of the soil cement. Shipments of cement and bentonite arriving onsite were tracked in an onsite log, and reconciled with quality documentation received from the suppliers. When bentonite slurry was batched as cutting fluid, it was tested by an onsite quality control technician for specific gravity, marsh viscosity and temperature prior to being injected, in order to ensure it was mixed to the correct consistency and properly hydrated. When cement slurry was batched, it was tested for specific gravity prior to being injected, and batching proportions were recorded by the mixing computer. During panel construction, the CSM's onboard computer measured and recorded cutter-head depth, injection volume per depth, cutter wheel

© 2016 Deep Foundations Institute

rotation speed, cutterhead verticality in the X-Y plane, cutterwheel rotation speed, cutterwheel pressure, and other relevant panel construction parameters. Not only did the onboard instrumentation of the CSM provide documentation which confirmed that panel installation was occurring as per the project requirements; but it was identified partway through the project that the CSM panel installation was able to maintain verticality to a much tighter tolerance than initially planned. This allowed for the overlap between panels to be reduced for a portion of the alignment, accelerating the project schedule, and reducing the amount of cement usage required.

Of the 132 completed panels, 53 of them were sampled in order to confirm properties of the soil cement mixture. After panel construction was completed a specialized wet grab sampling device would be lowered by crane into the fresh soil cement and "wet-grab" samples would be retrieved. The wet grab sampling device originally consisted of a hydraulically actuated gate, which could open and close at a desired depth. It was identified during the first panel sampled that the hydraulic gate was being jammed by the gravel contained within the Cut-Off Wall Aggregate. An alternate version of the sampling device was configured that night: a box with a trap-door bottom, which could be lowered to the desired depth with soil cement flowing through, and when pulled back to surface would close the trap-door and bring back sample from the desired depth. The trap-door sampling device is displayed in Figure 7 below. The wet grab samples were cast into 75 mm by 150 mm sample cylinders and tested for UCS and hydraulic conductivity at various different curing times. UCS tests were conducted to ASTM D1633, hydraulic conductivity was tested to ASTM D5084-10 and CU triaxial testing was conducted to ASTM D4767-11. An onsite curing room was set up, with controlled temperature and humidity. The temperature of the curing room was documented at regular intervals, to confirm the soil cement samples were curing in consistent conditions. An onsite UCS testing laboratory conducted UCS testing in order to reduce the risks of damage to the samples at an offsite facility. Samples on which hydraulic conductivity and CU triaxial testing needed to be conducted, were transported to Golder Associates' lab in Burnaby, BC.



Figure 7: "Trap-door" wet-grab sampling device.

At the end of the project, a mud rotary coring drill rig was mobilized to site and four cured soil cement panels were cored. Due to the coarse grained material and weaker target UCS of the soil cement matrix, it was identified early on that it would be very difficult to retrieve representative core specimens. Optical and acoustic televiewers were lowered into the cored soil cement panels to record various relevant parameters, including sidewall image, sidewall geometry, borehole fluid temperature, borehole fluid resistivity, orientation of instrument, and relative hardness of the sidewall. The information from the televiewer logs was useful in confirming the consistency of the soil cement panels. Falling head permeability tests were also conducted on the cored panels. See an example of an optical televiewer log in Figure 8 below.

© 2016 Deep Foundations Institute

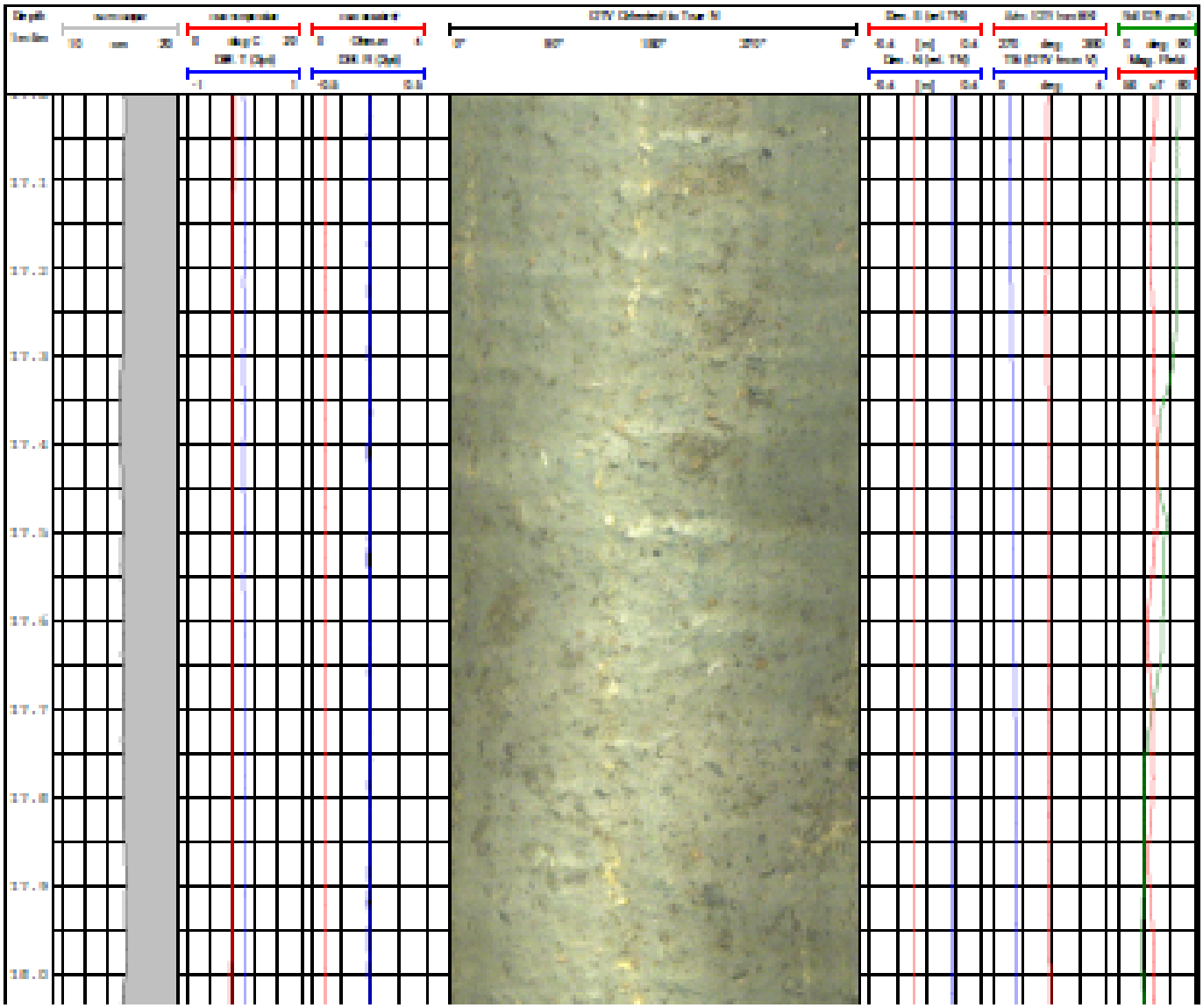


Figure 8: Optical teleview log of sidewalls of cored soil cement panel.

RESULTS

As the majority of the soil cement was constructed in a man-placed material of known properties, this project presents an ideal "sand-box" to assess some theories about soil cement formation in situ, as there is no interference in the data created by native material variability. It has been determined that soil cement samples placed in situ have the effects of in situ consolidation created by the weight of soil cement material on top of them. This consolidation typically increases the strength of soil cement specimens in which consolidation has taken place (Consoli, 2000). To assess the effect of depth of material retrieved versus water content, a graph was created comparing the water content and depth of samples retrieved from fresh soil cement in the Cut-Off Wall Aggregate section of the site. See this assessment in Figure 9 below.

© 2016 Deep Foundations Institute

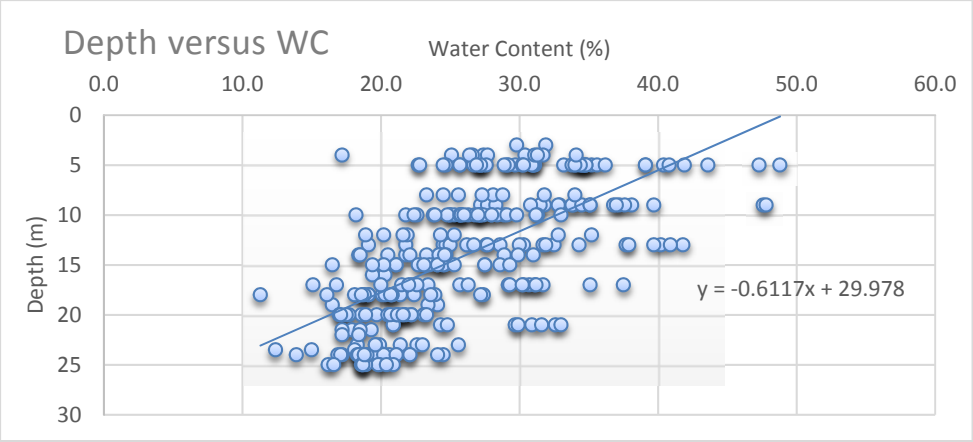


Figure 9: Depth versus Water Content for samples retrieved from the Cut-Off Wall Aggregate portion of the site.

From this analysis, a sample would lose approximately 1.6% of water content on average for every meter of depth it was mixed at. It should be noted that this is considering only water content losses which would occur in approximately the first hour of the soil cement mixing and curing process, as the wet grab sampling occurs directly after panel completion. Further study would be useful on the amount of water losses of soil cement that is cured in situ for a longer period of time (i.e., core samples); as significant amounts of representative core samples could not be collected at this site.

A total of 2553 samples were retrieved from the fresh soil cement panels. 1395 of the wet grab specimens were tested for UCS, with an average 28 day UCS of 1.89 MPa. 39 of the un-disturbed wet grab specimens were tested for hydraulic conductivity, with an average value of 1.92×10^{-8} cm/s. 21 of the wet grab specimens were sheared during CU triaxial testing, and all results met the project requirements for ductility. The coefficient of variation of 28 day UCS results was approximately 0.47 for all results, and approximately 0.36 for samples retained from within the zone of Cut-Off Wall Aggregate; this is towards the lower end of what would be expected for soil mixing projects (Filz and Navin, 2010).

CONCLUSION

The proceeding was a synopsis of a challenging CSM project with a constrained timeline, undertaken in a challenging environment with many technical hurdles. Lessons learned include the following:

- The CSM mixing method can be used to construct a cut-off wall through a porous gravel medium in a situation when construction is required in a time-sensitive manner and conventional dam construction techniques are impractical.
- In order to avoid termination due to advancement rate based refusal criteria, pre-drilling was required in order to penetrate deeper than approximately 12 meters depth through the Cut-Off Wall Aggregate at its in-situ level of compaction.
- When using the deep mixing method in man-made soil deposits with controlled properties, material uncertainty and variation is reduced; Construction may progress successfully without a field trial program. The coefficient of variation for UCS results of this project is towards the lower end of what would be expected for soil mixing projects.
- Different soil-cement sampling techniques may be required for different material types. For wet-grab sampling in fresh coarse grained soil cement, a trap-door wet-grab sampler works well.
- For core sampling of cured coarse grained soil cement with a weak matrix, little core recovery can be expected. However, material consistency can be assessed through use of acoustic and optical televiewers, as well as in-situ falling head permeability testing.
- In situ consolidation of coarse grained soil cement material may result in approximately 1.6% of reduction of water content per meter depth of overlaying soil cement at time of mixing.
- CSM production in coarse grained deposits requires a well-planned and thorough preventative maintenance program in order to keep up with wear and tear on the CSM cutter wheels.

© 2016 Deep Foundations Institute

ACKNOWLEDGEMENTS
The Mount Polley Mine Cutter Soil Mixing project was a challenging, rewarding, and exciting project, throughout which many participants collaborated for a positive outcome. The authors would like to thank all those who were a part of its successful completion, specifically the staff of Mount Polley Mining Corporation, project team, design team, and site team.

REFERENCES

Consoli, N.C.; Rotta, G.V. and Prietto, P.D.M., *"Influence of curing under stress on the triaxial response of cemented soils"* Géotechnique, 50 (1) (2000), pp. 99–105

Filz, G.M. and Navin, M.P. (2010). "A Practical Method to Account for Strength Variability of Deep-Mixed Ground," *GeoFlorida 2010: Advances in Analysis, Modeling & Design*, (GSP 199), ASCE, Reston, 8p.

Golder Associates Ltd., 2015. 2015 Freshet Management Embankment Cut-Off Wall Technical Specifications. Vancouver, BC, Canada.

Google Earth, 2005-2015. Accessed April 3, 2016.

© 2016 Deep Foundations Institute

INNOVATIVE GROUTING TECHNIQUES FOR INSTALLATION OF A SEEPAGE BARRIER AT BARRAGE DES QUINZE DAM (QUEBEC, CANADA)

Michael Byle, D.GE, F. ASCE, Peter Bowman, Danny Déry-Chamberland, Ing.

Abstract

The Barrage des Quinze dam is a concrete sluice dam consisting of 19 spillway sections, each of which is approximately 7.6 metres wide and 14.5 metres deep. Rehabilitation efforts being performed on the dam including demolition and improvement of the roadway and spillway sections and grouting to mitigate underseepage. Initially, conventional cement grouting was attempted and was successful in 14 of the spillway sections, but unsuccessful in 5 of the sections due to the high water flows which washed the grout away. The removal of the roadway and portions of the piers during construction temporarily reduced the stability of the structure limiting grouting pressures that could be applied. This and the substantial high water flows (leakage) beneath the dam between the concrete/rock contact and within fractures in the upper rock formation created a challenging situation to create a cutoff.

A grouting solution was developed that involved implementing several different grouting methods and materials including balanced high mobility grouts (HMG); low mobility grouts (LMG); polyurethane grouts; and sodium silicates. Because of the critical scheduling for the work, multiple approaches were planned and made available on site so that the work could proceed with the best technology with little, if any, time loss. The end goal of the grouting program was to create a seepage cutoff beneath the 5 sections of the dam in a stepwise process. The initial step was to fill the large openings and create a grout barrier to reduce the high water flow. The second step was to seal the contact between the base of the dam and the bedrock; and the third step was to complete closure by grouting a deep grout curtain into the underlying rock. Other features of the work included an extensive grout mix testing program, water pressure testing, borehole camera survey, and monitoring of the grout injections.

Critical to the success of the project was the careful planning and execution of the hole pattern and injection sequence to minimize grout loss due to high water flows and optimize cutoff of the high flows to permit closure with a HMG grout curtain. A novel use of a sodium silicate as a setting agent together with a balanced stable cement grout proved to be a highly effective technique to stop the high water flows streamlining the operation and eliminating the need for LMG and polyurethane grout. The project was successfully completed through the winter months in northern Quebec Province using special measures to protect the work space and provide heating for grout materials for proper injection and cure. The work was completed within the required schedule to enable use of the dam gates to manage spring flows.

Background

Des Quinze Dam is a concrete sluice dam consisting of 19 spillway sections (figure 1) for a total length of 470 feet. The hydraulic head of the dam is 25 feet and has a spillway capacity of 94 000 CFS. It is located on the upper portion of the Outaouais River in northwestern Quebec. The approach road is built upon the embankment dam on both sides and a bridge sits over the piers of the concrete structure.



Figure 1: Downstream view of Des Quinze Dam before the project

© 2016 Deep Foundations Institute

The embankment dam and concrete sluice dam were originally built in 1911 to control floods. At that time, a log pass was designed at the center of the structure. The piers of the old log pass were longer than the others and extended into the location of current sluices 12 and 14.

Reconstruction work at the dam site occurred in 1939, relocating the dam just downstream of the previous one to its current location. The foundation for the new dam was drilled and blasted into the exposed rock. Historical drawings from that construction noted a zone of 'water bearing faults areas' between sluices 6 to 14.

Repairs were made to the dam in 1954. A new concrete apron on the downstream side of the sluices was constructed between the piers to mitigate scouring problems (figure 2).

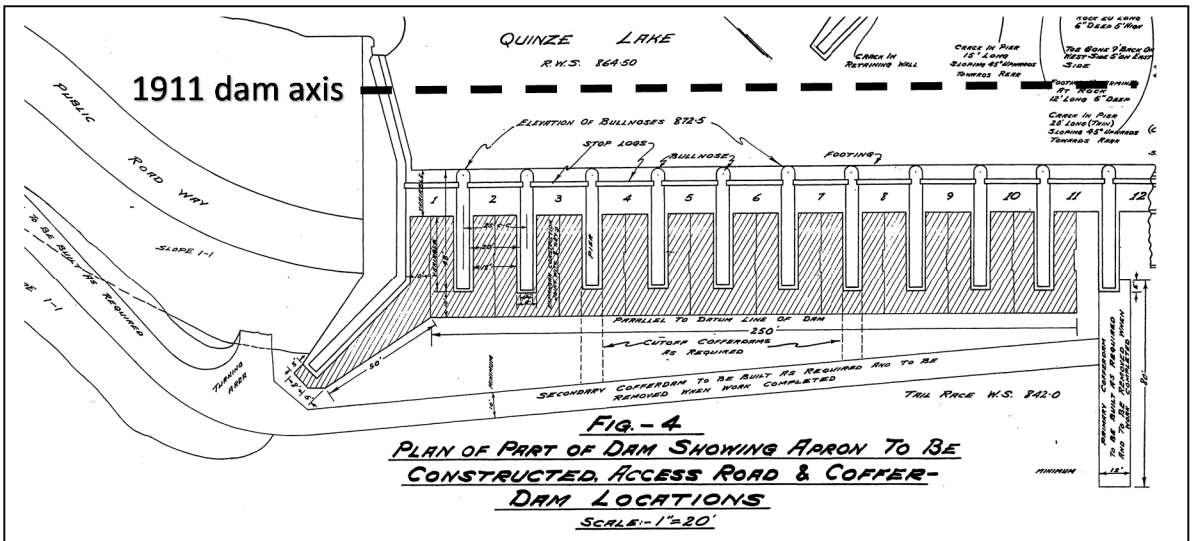


FIG. - 4
PLAN OF PART OF DAM SHOWING APRON TO BE CONSTRUCTED, ACCESS ROAD & COFFER-DAM LOCATIONS
SCALE:- 1"=20'

Figure 2: Historical for the 1954 apron construction showing alignment of previous dam.

Scope of the project

In 2013 improvements were undertaken at the Des Quinze Dam. These improvements included enlarging the roadway across the crest of the dam, extension of the piers, resurfacing of the concrete apron and relocation of sluice gates downstream of the existing ones. As part of this work post-tensioned anchors were installed to improve stability, and grouting was implemented to reduce seepage beneath the dam.

A significant challenge on this project was to complete all of the improvements while maintaining the spillway capacity throughout the year to be able to meet the requirements of downstream hydroelectric facilities. Dewatering of the working area in each sluice was realized using the existing gates and stoplogs with the addition of a geomembrane and clay installed by divers. While this setup was not completely waterproof, the leakage was managed by pumping of residual water behind the stoplogs. A downstream cofferdam was constructed at the downstream end of the apron, using waterstops and speedcrete between concrete blocks. While this provided a dry working area, it did not reduce the hydraulic head or flow under the dam.

Geologic setting

The dam is located in the Superior geological province, which is from the Archean period. The rock at dam site is a fine grain, dark green to black, basalt. The rock quality is fair to excellent and the UCS is higher than 100 MPa. The foliation dips 48^{o} to the south west and three additional joint sets having various easterly and westerly orientations with dip angles ranging from 46-60^{o}.

© 2016 Deep Foundations Institute

Investigations

Geotechnical data were available when designing the initial grouting program. Televiewer data and water testing were available from 6 boreholes drilled in a single line downstream of the stoplog gates through the concrete structure. Half of those boreholes were drilled directly into the piers and piezometer were installed.

The investigation data suggested a wide range of conditions throughout the dam, with a pretty tight rock-concrete interface on the left side while major openings have been identified between the structure and the underlying rock in the middle and on the right side. The spacing of those openings varied from 20 to 51 mm with an associated permeability between 20 to 62 liters/min/m, which was the maximum capacity of the pump. The piezometers installed in the structure has shown that the water pressure distribution was almost at the level of the headwater even if they were installed downstream of the stoplogs.

High flows were encountered from boreholes through the apron. Discharges carrying sediment were encountered in many holes (figure 3).

Initial grouting

The remediation grouting was planned with a plain cement grout with a W/C ratio varying from 1:1 to 0.6:1 in volume (0.75:1 to 0.45:1 in weight). Common additives such as superplasticizer and strength accelerator were used as needed. For difficult and flowing conditions, a powerful thixotropic agent (Celbex 653 from Multiurethanes), which had been used successfully in previous projects, was included.



Figure 3: Artesian conditions in an apron hole, washing the sand and fine gravel away

The typical drilling pattern was designed and applied in each sluice (figure 4). An upstream blanket row of 3 holes was drilled at 2 meters behind the stoplogs followed by a row of 8 m deep grout curtain, both using the split-spacing method. Finally, a contact grouting was done in 3 different rows to cover the footprint of every pier (and apron in between).

The grouting went as planned for the first part of the project, which was at the left side of the dam. Water flows and pressures tended to increase as work progressed through the center of the dam. In order to close the grouting holes that had high flow conditions, Celbex 653 was used to get refusal. The initial dosage started at 25 grams per 40 kg of cement and was increased up to 100 grams per 40 kg of cement. Dosage higher than 100 grams per 40 kg of cement could not be used because of the reduced pumpability of the mix at higher dosage. Verification holes were drilled at proximity of high take holes to assure the completion of the work.

© 2016 Deep Foundations Institute

The second part of the project was to complete grouting from the middle and right side of the dam. The first 5 sluices from the right went well. When the center portion of the dam was accessed, it was discovered that there was extensive degradation of the apron concrete from sluice 10 to 14. Some cavities at the surface of the apron released high flows of water from beneath the dam. In order to seal these openings, the cavities were filled with clean gravel and topped by a steel plate bolted into the surrounding concrete. The steel plate was fitted with valved ports to release the pressure while installing the plate. Grout was injected through the ports to seal the opening.

After many trials and 1 month of works, this method was found to be ineffective for the encountered conditions. While refusal conditions were developed while grouting into the ports in the plate, adjacent drilling proved to return plenty of water at the same rate and pressure as before the grouting operation. So, it was concluded that the high flows into wide openings were eroding the grout before its initial set.

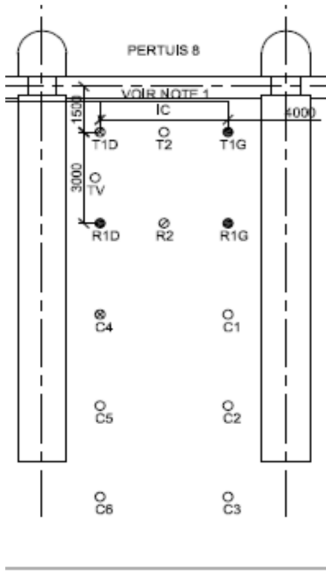


Figure 4: Drilling pattern used during initial Phase 1 grouting (dimensions are in mm)

Phase 2 Grouting: Mitigation Approach for High Flow Leakage

Additional coring in the sluices 10-14 revealed openings in the rock and gaps at the concrete/rock interface. Some fine gravel was noted and significant flows observed. Core recovery varied and it appeared that significant voids were present.

Pattern and Sequencing

The grouting consisted of three aspects, a barrier line to cut off major flow, a deep curtain and contact grouting at the concrete interface. Given the 45-60º angle of the foliation planes, absence of vertical features, and physical constraints within the sluices, it was decided to use vertical holes for the cutoff. The plan was to grout the stone infill that had been placed earlier, complete a line of barrier holes 6.5m deep using low mobility grout (LMG) or polyurethane grout to cut off the major flows, then complete a curtain grouting in a line of 8m deep holes to complete the cutoff. Because of some openings at the downstream edge of the

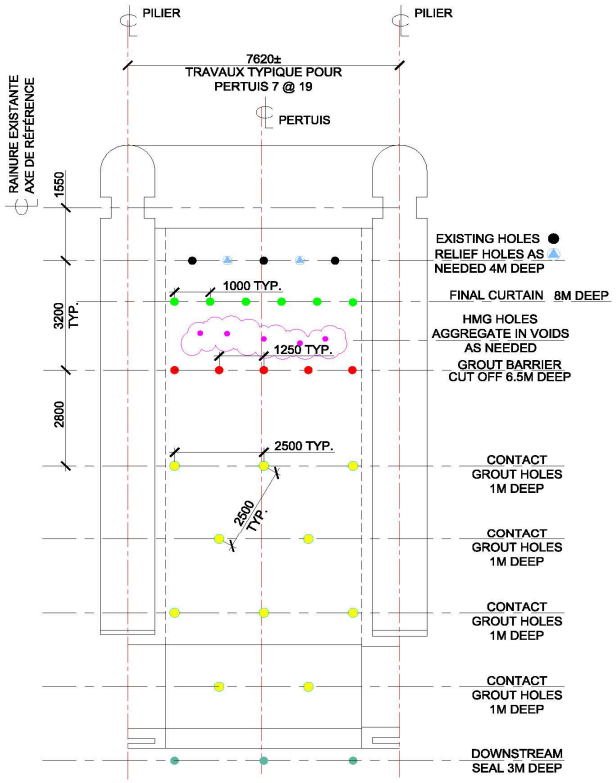


Figure 5: Layout of Phase 2 Grouting

© 2016 Deep Foundations Institute

dam, a line of holes was planned using LMG as containment for the contact grouting. The curtain and contact holes were planned as high mobility grout (HMG) using a balanced grout. An option was included to use sodium silicate addition to the cement based grouts to accelerate setting and prevent washout.

The plan called for each grout hole to be examined with a borehole camera to assess the size of the voids present. In this way, the correct grout could be selected. LMG or polyurethane was planned for holes having voids greater than 0.15m.



Figure 6: Sluice Enclosure for Winter Work

Phase 2 Grouting

The project was subjected to a number of challenging conditions which included extreme winter conditions with temperatures as low as -40° Celsius and significant snowfall accumulation. The extreme winter

© 2016 Deep Foundations Institute

conditions required that the work locations be enclosed and heated in order to protect personnel and equipment and maintain adequate grout temperatures (Figure 6). On the platform bridge, the grout storage and batching areas were enclosed with temporary shelters, and at each spillway section large sections of tarps were secured across the downstream face of the spillway piers in order to protect the work space. The use of diesel fired portable heaters with an output of approximately 300,000 btu provided heat for these work areas (Figure 7).

An over water platform was utilized to stage all the grouting equipment and materials and provide access to each spillway bay. A mobile crane positioned on the platform was used for placement of the drilling rig inside each spillway section as well as for setting grout materials and equipment.

Equipment

A heated water distribution system supplied all the required water for the drilling and grouting operations. The system was designed to provide and maintain a constant supply of heated water of 375 lpm @ 1,000 kPa utilizing a 1 million BTU heater. The water plant was positioned at the end of the spillway sections at the entrance to the over water platform. From this location, water supply across the site was accomplished via an insulated hardline system to the drilling and grouting equipment locations. Predetermined valve locations were installed along the length of water line in order to allow the drilling and grouting crew access to heated water at each spillway section.

A containerized grouting plant was used for batching the high mobility cement based grouts (HMG). The grout plant included a colloidal mixing plant with storage agitator, a progressive cavity (moyno) grout pump, dedicated water testing pump, and an admixture dosing station for the addition of grout additives such as superplasticizer. The container was selected for this project due to the fact that it was self-



Figure 7: Heaters on dam crest to heat sluice enclosures.

© 2016 Deep Foundations Institute

contained, heated, and could easily be positioned on the over water platform with the use of a crane.

A site-built temporary shelter was installed adjacent to the containerized grout plant which provided the necessary space for storage of grout materials, and a central location for the low mobility grout (LMG) grout plant.

Drilling equipment included a small track mounted rotary percussion drill equipped with water flush, and a track mounted rotary diamond core drill equipped with N and H size casings (Figure 8). Upon reaching the final target depth of the hole, the hole was flushed with water through the drill string until clear water returned at the surface. Holes were protected with plastic cones or with a mechanical packer to ensure that debris did not enter the holes after they have been cleaned.

Environmental Controls

Waste water from the drilling and grouting operations was collected at the downstream end of the spillway section with the use of temporary containment. A submersible pump was utilized to pump the waste water from the temporary containment to a settling tank located at the end of the spillway. The settling tank contained a baffle to allow any solids to settle out and the clean water to be pumped backed to the river. The solids that accumulated in the separation box were periodically cleaned out and disposed of offsite at an approved facility.

Anchors

Due to the rehabilitation work being performed there was a concern that the structure would be susceptible to sliding, so it was determined that mechanical anchors would be installed in the spillway sections to improve temporary stability. The anchors that were selected were mechanical spin-lock type anchors which provided the benefit of immediate anchorage and tensioning. The anchors were installed prior to grouting and it was a requirement that at least the adjacent spillway bay next to the one being grouted be anchored prior to any grouting on that spillway section.

Most anchor holes were drilled using the rotary percussive drill equipped for water flushing, as the use of air flushing was not permitted. A nominal three inch diameter drill bit was used for all anchor holes. During the initial drilling, installation problems occurred when attempting to



Figure 8: Drilling rig inside enclosure

© 2016 Deep Foundations Institute

lower the anchor in the drilled hole due to broken rock and collapsing holes that prevented the insertion of the anchor. The solution was to drill the anchor holes using a rotary diamond core drill and steel casings that kept the hole open during anchor insertion. An NW size casing was reamed on the outside of NQ size rods to a depth that would exceed the broken concrete/rock zone. The NQ rods were then removed leaving the NW casing in place. The hole was flushed with water, and the anchor was lowered into the hole inside the NW casing. Once the anchor was lowered to the bottom of the hole, the NW casing was removed.



Figure 9: Mechanical Anchors

Urethane Grouting

Polyurethane grout was originally considered to act as the grout barrier wall due to the possibility of large void areas beneath the concrete spillways, as well as the potential for a high water flow rate which would be problematic for cement based grouts. However, during the preliminary mix testing procedures it was discovered that the urethane had a slower and ineffective set time due to the cold temperatures of the flowing water and therefore was ruled out as a grout treatment method.

High Mobility Grouting (HMG)

For the Phase 2 grouting, high mobility grouts consisted of balanced-stable grouts (Naudts et al, 2003) comprised of water, Type GU Portland cement, bentonite, welan gum, anti-washout agent, superplasticizer, and accelerator. Three different grout mixes (A, B and C) were formulated with Marsh Funnel Time of 41±s, 58±s and 85±s, respectively, to meet the possible site specific conditions.

The grout components utilized were:

1. Portland cement, Type I or GU;
2. Bentonite, Cetco Premium Gel;
3. Superplasticizer, Polyheed 997;
4. Viscosity modifier, CP Kelco Welan Gum,
5. Accelerator, BASF MasterSet FP20
6. Water

These conventional HMG mixes produced setting times raging from approximately 15 to 21 hours at ambient temperature.

High Mobility Grouting (HMG) with Sodium Silicate

Sodium silicate was introduced to the HMG when unconfined zones with high ground water flow were encountered which did not allow a stage to properly refuse through the injection of regular HMG. Sodium silicate was introduced directly to the grout stream at the point of injection through a mechanical packer. Using a piston pump, the liquid flow was gradually increased up to a maximum of 15% by flow.

Low Mobility Grouting (LMG)

The low mobility grout was made of pre-blended grout materials. The pre-blended grout mix contained Portland Cement (Type GU), Fly ash (Type F), and a mason's Brick Sand, and packaged in 66-lb bags. Each bag contains 3.3-lb (5%) Portland cement, 27.7-lb (42%) Type F flyash, and 35.0-lb (53%) minus 30 Brick Sand. The range of LMG slump were adjusted based on site conditions, and were in the range of 2 to 9 inches.

© 2016 Deep Foundations Institute

Grouting Performance

Pressures and volumes were limited to assure stability of the dam and avoid creating excessive uplift. Refusal was defined based on the tolerable uplift on the dam based on structural analysis and the capacity of the anchors. An example of the refusal diagram is included in Figure 10. A summary of the grouting results is presented in Table 1.

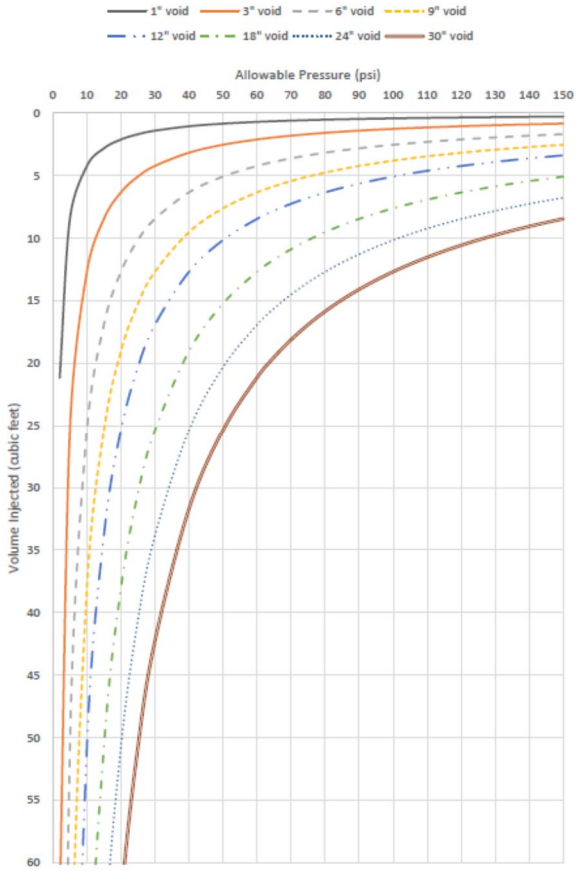


Figure 10: Allowable Pressure Based on Volume of Grout Injected and Depth of Void

Barrier and Curtain Grouting

The video examination of the barrier grout holes during the grouting program found no openings larger than 0.15m, so LMG was not used. High velocity flows were encountered, so HMG modified with sodium silicate was used to stop the major flows in the barrier holes. The silicate addition produced a rapid set that enabled the flow to be effectively cut off. Grout was monitored based on apparent Lugeon theory (Naudts, 1996). Initial injections produced apparent Lugeon values in the range of 25-40 after the injection of the silicate amended HMG. During initial injections, the grout traveled laterally and appeared from vent holes in adjacent sluices. In several instances, grout was observed two sluices away.

Grouting with packers in rock was not possible due to the openings and flow present. Given the open angled foliations, the grout would have migrated to higher levels while grouting at depth. Therefore, to improve performance, the barrier holes were grouted in a downstage manner. This was accomplished by grouting the upper 3m of each hole. The following day, the hole was redrilled to the full 6.5m and grouted to refusal. This procedure enabled greater pressures to be applied at depth and sealed the high flows in the upper zone to enable more effective grouting at depth.

The grouting was completed using a split spacing approach. The secondary holes on each line proved to have lower volumes to refusal (Table 2). The curtain line injections had considerably better results and proved that the barrier holes had been effective in stopping the major flows. The curtain holes were able to achieve closure to less than 5 Lugeons for all final closure holes.

Table 2 clearly shows the effectiveness of the silicate amended HMG in producing an effective cutoff under flowing water conditions. The grout take was reduced by 98% between the primary and secondary injections along the barrier hole

Table 1: Average Take by Hole Type

Type	Number of Grout Holes	Average Take (kg/m)
Barrier	25	62
Barrier, Downstream	17	51
Barrier, Lateral	8	34
Contact, HMG	33	29
Contact, LMG	32	174
Relief	10	60
Curtain	25	12

© 2016 Deep Foundations Institute

line. The primary injections in the curtain line were on the same order of magnitude as the secondary barrier injections, but split spaced holes indicated a further 32% reduction.

Table 2: Average Take by Hole Sequence

Type	Number of Grout Holes	Average Take (kg/m)	Take Reduction between Primary and Secondary Holes
Barrier			
Primary	15	97	
Secondary	10	11	89%
Curtain			
Primary	15	13	
Secondary	10	9	32%

Contact Grouting

Contact holes were injected with either LMG or HMG depending upon the opening sizes. Larger openings were found in the downstream holes. After the barrier holes were grouted, little difficulty was encountered in the contact hole grouting. Verification was through observation of grout surfacing at adjacent holes and achieving refusal.

Conclusions

A total of 157 holes were drilled, video inspected and grouted. This was accomplished in the months of January through March in northern Quebec in extreme weather. The use of sodium silicate as a measure to effect rapid set of a balanced stable grout proved effective and highly controllable as a means to stop high velocity flows. The program methodology to create a barrier line followed by a conventional curtain grouting proved sound. The grouting was highly successful in stopping under seepage and restoring contact to the dam foundation.

References

Naudts, A., Landry, E., Hooey, S., and Naudts, W. (2003) Additives and Admixtures in Cement-Based Grouts. Grouting and Ground Treatment (Eds. L. Johnsen, D. Bruce, M. Byle), ASCE Geotechnical Special Publication 120, pp. 1180-1191.

Naudts, A. (1996) Grouting to improve foundation soil. In Practical Foundation Engineering Handbook (ed. R. W. Srown), pp. 5.277-5.400. McGraw-Hill Book Co., New York.

© 2016 Deep Foundations Institute

SEEPAGE BARRIER AT BOLIVAR DAM

Authors: Mohamed Elgouhary, TREVIICOS, 38 Third Ave, Charlestown, MA, USA 617-241-4800, melgouhary@treviicos.com; Daniel Koziol, TREVIICOS, 38 Third Ave, Charlestown, MA, USA, 617-241-4800, dkoziol@treviicos.com and Carol L. Tasillo, U. S. Army Corps of Engineers, 1000 Liberty Avenue 11th floor, Pittsburgh, PA, Carol.Tasillo@usace.army.mil

Contributors: Jeff Taylor, TREVIICOS and Seth Lyle, U. S. Army Corps of Engineers

ABSTRACT

The Bolivar Dam, located near the Village of Bolivar, Ohio is one of a system of dams designed to provide flood control and water conservation in the Muskingum Watershed. The Bolivar Dam is a "dry dam" that only impounds water during flood events. In order to address problematic seepage, which negatively affects the structural stability of the dam, the U.S. Army Corps of Engineers (USACE) designed a partial-depth and partial-length Seepage Barrier Wall (SBW) through the upstream slope of the dam. For this project, TREVIICOS constructed a 36 in (914 mm) positive SBW by plastic concrete panel wall method and excavated using hydromill and clamshell equipment. The plastic concrete was manufactured on-site, which allowed for enhanced quality control and prompt service. This required the installation, commissioning and operation of a concrete batch plant on-site. This paper presents the challenging aspects of the design, manufacture and placement of the plastic concrete for the SBW in order to meet contract strength and permeability requirements as well as workability requirements necessary for tremie concrete placement.

PROJECT GENERAL INFORMATION

Bolivar Dam is located in Northeast Ohio (Fig. 1). It is one of a system of sixteen dam projects designed to provide flood control and water conservation in the Muskingum Basin, which covers 20 percent of the State of Ohio (Fig. 2). Bolivar Dam is on Sandy Creek, a tributary of the Tuscarawas River.



Fig. 1. Bolivar Dam Location

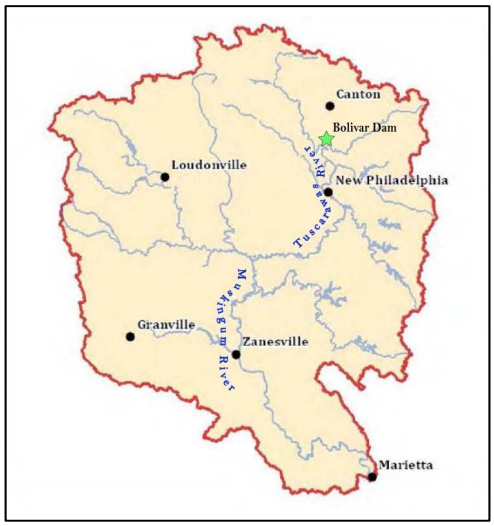


Fig. 2. Muskingum Watershed

The Dam has been constructed, operated and maintained by the USACE, for the Muskingum Watershed Conservancy District. Its construction was completed in 1937 and it has been in operation for flood control since 1939.

© 2016 Deep Foundations Institute

Bolivar Dam is a rolled earth fill dam, with a typical embankment section made of a central impervious core flanked upstream and downstream by a pervious outer shell with downstream rock toe. The embankment has a maximum height of 87 ft (26.5 m) and a crest length of 6,300 ft (1,920 m), and is comprised of the main embankment starting on the left abutment of about 1,500 ft (457 m) heading to the Northeast, followed by the terrace embankment of 4,800 ft (1,463 m) heading to the North. An upstream impervious blanket extends up to 500 ft (152 m) upstream of the dam along the majority of its length. The outlet works are to the left of the embankment, and consist of an intake structure with six gates, followed by two outlet conduits that finish in a stilling basin. To the left of the outlet works is the uncontrolled emergency spillway, which was doubled in width to 540 ft (165 m) in 1989, and excavated material utilized for construction of a stability berm against the upstream face of the main embankment. A concrete parapet wall was also installed along the upstream side of the crest during this work. In 1982, a toe drain, downstream filter berm and line of 35 relief wells were installed. The filter berm was recently extended an additional 200 ft (61.0 m) downstream and granular filters were placed over an approximate 2000 linear ft (610 m) reach of the lower terrace slope. (Fig. 3).

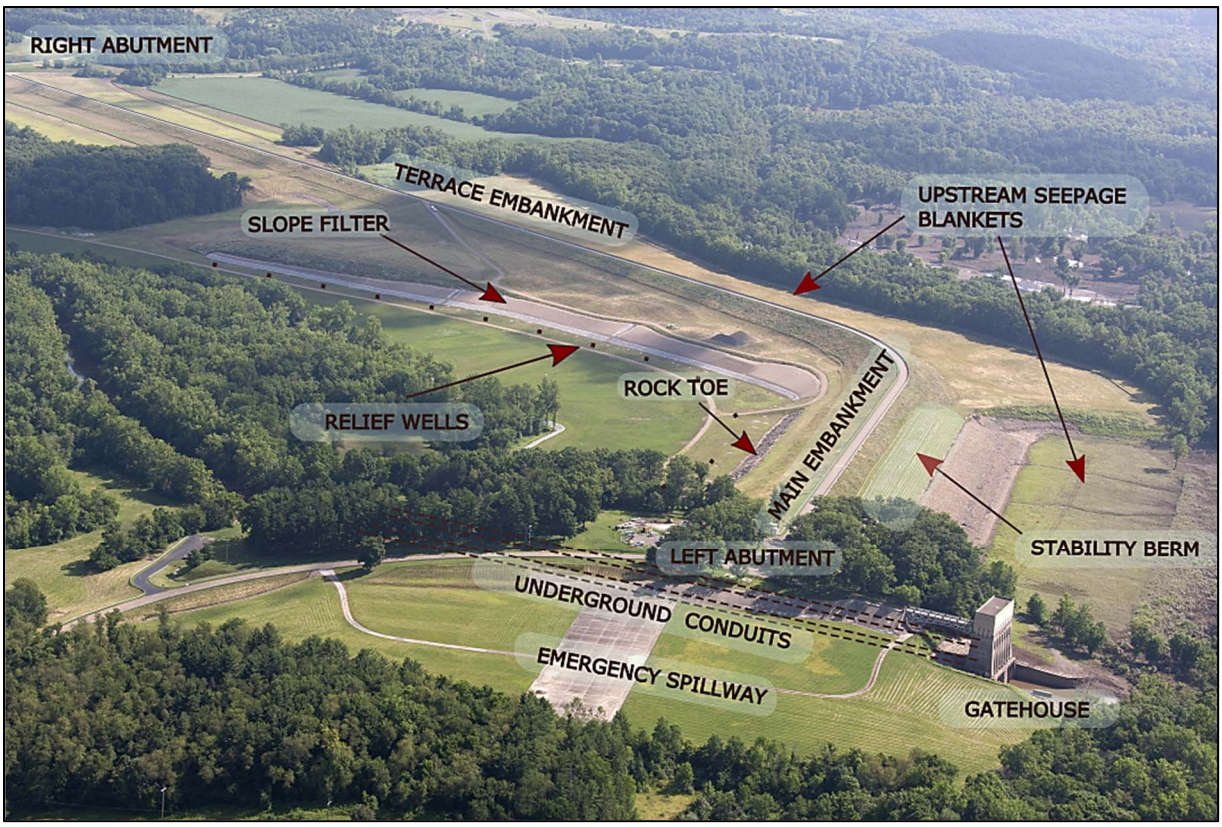


Fig. 3. Aerial picture showing location of main dam components

PROJECT PROBLEM AND TECHNICAL SOLUTION

Situated just below the glacial margin in northeast Ohio, Bolivar Dam spans a deep outwash filled valley of the Sandy Creek, confined by a bedrock trough of about a mile wide. The left abutment of the dam is the only portion founded directly on the bedrock ridge. The top of rock slopes down towards the center of the dam to a depth of 210 ft (64.0 m) below the embankment and rises near the right abutment to a depth close to 85 ft (25.9 m) below the Dam. The stratified soil deposits below the dam embankment predominantly consist of sand with variable amounts of silt, and several gravel strata with variable thicknesses and extents. The bedrock is predominantly composed by interbedded strata of claystone, limestone, shale, and sandstone. The top of rock is normally weathered or fractured. Competent rock

© 2016 Deep Foundations Institute

strength varies from hard to very hard, depending on the formation, with unconfined compressive strength ranging from about 5,000 to 30,000 psi (34.5 to 206.8 MPa).

The Bolivar Dam is considered a “dry dam” because most of the time, it has the controlled retention flood pool below the dam. When the flood pool is retained by the dam, the site geology leads to confined seepage through the dam’s pervious foundation and through discontinuities in limestone units of the left abutment (Fig. 4). The dam spillway elevation is 962 ft (293 m), and historically, pools as low as 920 ft (280 m) had produced flows that exited the downstream toe. Uncontrolled flow through the left abutment bedrock was documented in the vicinity of the outlets works discharge area for pools exceeding elevation 943 ft (287 m). During the record pool elevation of 952 ft (290 m) in 2005, excessive uncontrolled seepage was observed discharging from the lower terrace slope above the rock toe, in the curved section of the dam. Evidence of sand piping from the terrace slope and boiling downstream of the dam was also found. The relief wells were flowing at very high rates prior to being inundated by the tailwater as a result of a record pool at Dover Dam, located roughly 10 miles downstream of Bolivar Dam.

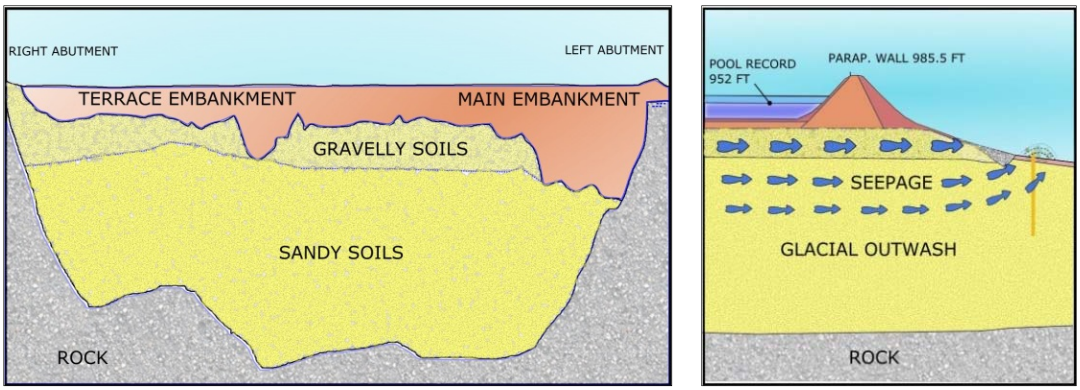


Fig. 4. Dam geology profile and cross section (not to scale)

In 2013 the USACE classified Bolivar Dam as a Dam Safety Action Class (DSAC) II, characterized by “failure initiation foreseen” or “very high incremental risk”. As a result of this classification, the USACE defined the project for the major rehabilitation of the Dam. This project consisted of the construction of a partial-depth SBW and a drilling and grouting (DG) curtain (Fig. 5). The SBW consisted of approximately 4,500 ft (1,372 m). in length, and a maximum depth of 144 ft (43.9 m) through the upstream slope of the dam embankment terminating in the left abutment. The DG started on the left abutment, connecting to the SBW, and ended at the emergency spillway. The DG has approximately 300 ft (91.4 m) in length and an average depth of 60 ft (18.3 m).

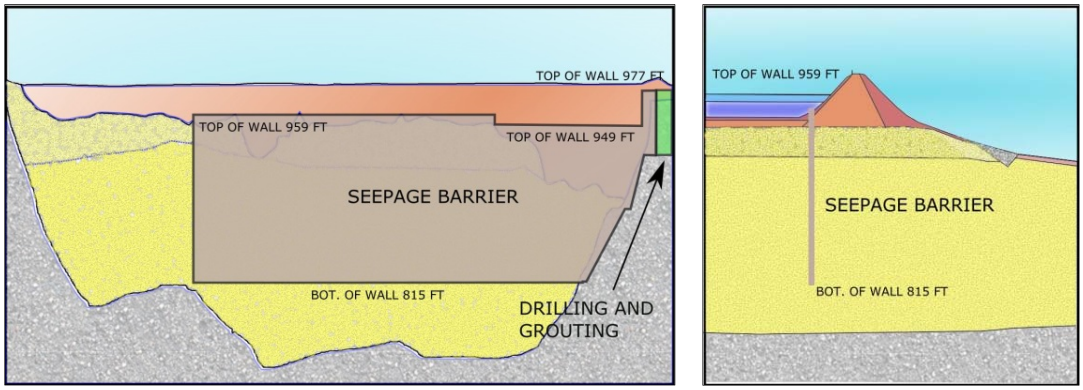


Fig. 5. SBW and D&G limits profile and cross section (not to scale)

© 2016 Deep Foundations Institute

SEEPAGE BARRIER WALL PERFORMANCE REQUIREMENTS

The Technical Specifications required, among other items, the following main performance criteria for the completed seepage barrier:

- Homogeneous backfill material
- Continuous SBW, meaning with no open joints, open seams, or open cracks which would have the potential to allow seepage and/or reduce the overall effective permeability of the barrier.
- Minimum width of 24 in (610 mm) through the entire length and depth of the SBW,
- In-place permeability of less than 1x10-6 cm/sec at 28 days.
- Minimum unconfined compressive strength (UCS) of 750 psi (5.2 MPa) at 28 days age (10-point moving average) on core samples, with no single UCS test results below 500 psi (3.4 MPa).

CONSTRUCTION METHOD

The construction method selected by TREVIICOS for the SBW consisted in the installation of overlapping primary and secondary panels, of 36 in (914 mm) wide, excavated with clamshell and hydromill equipment under bentonite slurry. The primary panel length was typically 30 ft (9.1 m), which was the maximum allowed length. The secondary panel length was 9 ft (2.7 m) and overlapped minimum 12 in (305 mm) with the two adjacent primary panels.

The SBW panels were backfilled with the tremie method. A series of 10 in (254 mm) diameter tremie pipes, typically in 10 ft (3 m) long section were connected together. Three tremie pipes were used during the placement of primary panels, and one tremie was used for secondary panels placement. Plastic concrete was delivered by ready mix trucks from the on-site batch plant to the panel.

PLASTIC CONCRETE

Based on the contract requirements and construction method, TREVIICOS selected plastic concrete as the backfill material for the SBW panels. In order to provide the best quality and production, it was elected to manufacture the plastic concrete on-site with a “wet batch” plant.

Plastic concrete consists of aggregates, cementitious materials, water, and bentonite, mixed at a high water to cement (W/C) ratio greater than one. It is characterized as ductile low strength concrete with permeability as low as $1x10^{-9}$ cm/s.

MIXING AND BATCHING PLANT

To optimize the quality with consistent and uniform properties, the plastic concrete was manufactured with a high efficiency “wet batch” concrete plant complemented with a bentonite slurry auxiliary plant. The plant was installed and operated by TREVIICOS. It was commissioned on-site after calibration and execution of trial batch tests.

The “wet batch” plant was fully automatic, and controlled by a PC based system (PCS Control), which stored and recorded the information for each batch. The plant included moisture probes for the fine aggregate as well as scales and gauges to measure the different backfill mix components. For each batch, the plant weighed and staged all the components of the plastic concrete above the mixer before the charging process started. Next, these components were thoroughly mixed in a dual-horizontal shaft, high shear mixer and the resulting homogeneous mix was then discharged into the ready mix trucks. This plant reached a production capacity of 90 cubic yards per hour.

© 2016 Deep Foundations Institute

Working in Ohio during Winter time required considerations for cold weather protection for the plant and materials. For cold weather protection, most of the components of the batching plant, the bentonite slurry auxiliary plant, and the ready mix truck parking areas, were covered with temporary heated enclosures (Fig. 6). The plant included a gas fueled boiler that was used to both regulate the temperature of the bentonite slurry and mixing water as well as to heat the aggregates using in-floor radiant heating pads. The mixing water was pumped from a relief well downstream of the dam and then transported to the plant through a buried pipeline. In addition, a significant benefit of having this plant on-site was the reduced exposure of the batched concrete in the mixer trucks during mixing and transportation from the plant to the placement location. The temperature control for the materials and the short transport time resulted in minimal variation of the placement temperature and provided consistent placement properties for the plastic concrete.



Fig. 6. Winterized batching plant and slurry preparation building

EVOLUTION OF PLASTIC CONCRETE MIX DESIGN

In order to achieve the appropriate mix, a comprehensive mix design program was developed by Mrs. Raffaella Granata, TREVI S.p.A. Technical Expert. This program started at the laboratory, where the mixes were prepared in small-scale quantities, followed by additional on-site testing, where the mixes were prepared utilizing the on-site batch plant, in larger quantities. This program continued to evolve with time based on the actual performance of the mixes.

The target of the mix design program was to produce mixes with the following properties:

- Slump: between 7-10 in (180-250 mm)
- UCS: between 1,300-1,600 psi (9-11 MPa) at 28 days
- Permeability: in the magnitude of 1x10-8 cm/s

Laboratory testing

Laboratory testing was performed in two phases. The initial phase started in the Spring of 2014 with the mixing of six different trial mixes. Each trial mix was prepared with different components and proportions. The components of the mixes were obtained from the same sources intended to be used during the construction of the project. Test cylinders from each trial mix were cast, cured, and tested for UCS and permeability at 7, 14, and 28 days.

© 2016 Deep Foundations Institute

Based on the test results obtained from the trial mixes, two mixes (Mix 2 and Mix 3) were selected for the final laboratory testing phase, which started in the Summer of 2014. Both mixes shared the same main components of water, bentonite, sand, and gravel. However, they differed in the composition of the cementitious material. Mix 2 was prepared with cement while Mix 3 was prepared with a mixture of cement and slag.

Following the preparation of the mixes, fresh concrete properties (i.e. slump, unit weight and air content) were tested and test cylinders from each mix were cast, cured, and tested for UCS and permeability at 7, 14, and 28 days.

Test results confirmed that both mixes achieved the desired properties in reference to consistency, workability, UCS, and permeability.

On-site testing

In the Winter of 2014, prior to the start of SBW construction, another series of mixing and testing for Mix 2 and Mix 3 were carried out using the on-site plant. This testing started with more focus towards Mix 2 since this mix was intended to be used during the Winter. The purpose of the on-site testing was to confirm the test results obtained in the laboratory, and to fine-tune the mixes for the effect of variables such as temperature and mixing procedures.

Unlike the small-scale batch size prepared in the laboratory, the on-site batches were prepared on a large-scale batch size. Fresh concrete slump tests indicated that adjustments were needed to improve the consistency and workability of Mix 2. These adjustments involved variations to the bentonite and cement proportions while maintaining a similar W/C ratio.

Three batches (Mix 2B, Mix 2C, and Mix 2D) were prepared with the varied bentonite and cement proportions. The batches fresh properties as well as UCS and permeability were tested. Mix 2D properties were deemed to be within the desired design criteria and therefore, Mix 2D was selected as the backfill for the SBW panels.

Admixtures

Although the original mix design did not include any admixtures, their use was anticipated. The need to introduce an admixture to the mixes became apparent after experiencing difficulties in plastic concrete flow towards the end of the placement of the first few primary panels, which required around 425 cubic yards each. These difficulties made it evident that enhancements were needed to extend the “Pot-life” of the mix in order to maintain its workability during the approximately 5.5 h of placement.

Several test batches were prepared on-site with different types of admixtures (i.e. hydration stabilizer and water reducer) and different admixture dosages. An admixture was selected and was used as panel placements proceeded. Additional studies continued to evaluate consistency and workability, and the effect of temperature. Slump loss was evaluated over time and ultimately the admixture dosage was optimized. Mixes that contained the admixture were denoted with the suffix “R” (i.e. Mix 2DR and Mix 3R).

Liquid dye

Another mix adjustment was needed when liquid dye was required to be added to the panel backfill mixes. Test batches were prepared to determine the required dosage of liquid dye and to examine the behavior of the mixes. Adjustment to the total water of the mixes was applied to compensate for the water

© 2016 Deep Foundations Institute

introduced to the mix by the liquid dye. The colored plastic concrete was eventually utilized for the backfilling of secondary panels to aid in the identification of panel joint locations during verification drilling.

QUALITY CONTROL

A stringent quality control program was established and implemented to ensure that the quality of the manufactured and placed plastic concrete was within acceptable parameters. This program involved controls on the plastic concrete materials, panel preparation prior to placement, during placement, and testing.

Quality control for plastic concrete materials

Since the quality and consistency of the materials used in manufacturing the plastic concrete is extremely important, the following materials controls were implemented:

- Verifying that delivered materials were from the approved sources
- Performing visual inspections of delivered materials
- Verifying aggregate test results (i.e. gradation, absorption, etc.) provided by the suppliers, on a monthly basis, are within the specified limits
- Performing additional quality control testing on aggregates and water used
- Performing moisture content testing on aggregate(s) prior to the start of batching
- Scales calibration

Quality control for panel preparation prior to placement

Prior to the start of placement, a desanding and bottom cleaning operation was performed to ensure that the properties of the excavation slurry were within the specified limits. Additionally, secondary panel's joints were cleaned by utilizing different type of brushes.

Quality control during tremie pipes setting and during placement

The following controls were implemented during tremie pipes setting:

- A tight connection between pipe sections was established by utilizing O-rings that sealed the couplers and a rubber sleeve was installed and wrapped with plastic wrap and duct tape at each joint to prevent infiltration of the slurry into the tremie pipes
- Slurry-saturated foam plugs were inserted into the tremie pipes prior to placing plastic concrete into the hoppers
- The rise of plastic concrete was measured at the end of each round (i.e. three trucks for primary panels and one truck for secondary panel) to ensure that tremie embedment was maintained at the 10 ft (3 m) minimum embedment and to identify the actual excavated volume backfilled during placement.
- Finish the placement after visually verifying that the quality of plastic concrete at the top is good and free of any slurry contamination

Testing and sampling

Properties of fresh plastic concrete were tested for slump and temperature. Typically, between four and eight loads were tested per placement. Tests were performed prior to discharging the load. The slump

© 2016 Deep Foundations Institute

requirement was between 7-10 in (180-250 mm), where the temperature requirement was between 40-90 ºF (4-32 ºC).

Additionally, twelve specimens were cast from each panel. Six cylindrical specimens 4 x 8 in (100 x 200 mm), were tested for UCS in accordance with ASTM D4832, and six cylindrical specimens, 3 x 6 in (75 x 150 mm) were tested for permeability in accordance with ASTM D5084 at 7, 14, and 28 days. Specimens were tank-cured and tested at the on-site laboratory. Test results were used to monitor the performance of the mixes and to detect any potential issues.

SEEPAGE BARRIER WALL VALIDATION AND ACCEPTANCE

The constructed SBW was validated and accepted only if the performance requirements were successfully met. The validation process involved performing verification drill holes where core samples were collected for UCS testing and in-situ permeability testing was performed.

Verification drilling

Verification drill holes were drilled along the centerline of the installed SBW with sampling at least every 50 ft (15 m) for the Demonstration Section and 200 ft (60 m) for the Production Section. Verification drill holes were drilled vertical or inclined. Vertical drill holes were located either within the body of the panels or at the joints. Inclined drill holes were drilled at 22 or 22.5 degrees from vertical where the drill hole intercepts multiple panels. Verification drill holes locations were selected by the USACE.

A rotary core drilling method was selected in order to provide the required samples for testing. Drilling was performed using a wireline system with a triple tube type core barrel. Drilling fluid used was water.

Continuity and homogeneity

Continuity and homogeneity of the in-place SBW were evaluated based on the quality of the recovered core samples (Fig. 7) and the quality of the core hole video image survey. The video image survey was performed by an Optical Geologger (OPTV) that recorded a 360-degree view of the borehole surface and projected the image as a “flat core” with description of depth and orientation (Fig. 8). This survey was performed after the borehole was flushed with a light bleach solution and filled with water. Note how the red dye in secondary panels assisted in the identification of the joints.



Fig. 7. Example of core sample at joint



Fig. 8. Example of OPTV image at joint

© 2016 Deep Foundations Institute

Compressive strength testing

Compressive strength for the in-place SBW was assessed by the UCS test results obtained from core samples. Five samples were select for UCS testing per boring at the following depth ranges: 10 to 25 ft (3 to 8 m), 25 to 50 ft (8 to 15 m), 50 to 75 ft (15 to 23 m), between 50 and 10 ft (15 and 3 m) from the bottom of the SBW, and within 10 ft (3 m) of the bottom of the SBW. UCS testing was performed at the on-site testing laboratory in accordance with ASTM C42 and D4832.

The acceptance criteria for the in-place SBW required a minimum UCS of 750 psi (5.2 MPa) at 28 days, as defined by a 10-point moving average over consecutive tests on core samples with no single UCS test results below 500 psi (3.4 MPa). A summary of 10-point moving average for UCS results for production section core samples tested at 28 days is shown in Fig. 9

Field Permeability Testing

Typically, field permeability testing was performed for each borehole at 28 days from the placement of the SBW. The permeability test was performed by means of a falling head test, meaning that the borehole was filled with water and the change in water level was recorded.

The change in water level was recorded for a period of 2 hours at 15 seconds intervals using an automated, electronic water level transducer. Manual measurements were recorded at 10 minute intervals utilizing a water level indicator.

The field data was then analyzed and reduced by the Hvorslev Method to obtain the permeability value. A summary of field permeability test results for the Production Section is shown in Fig. 10.

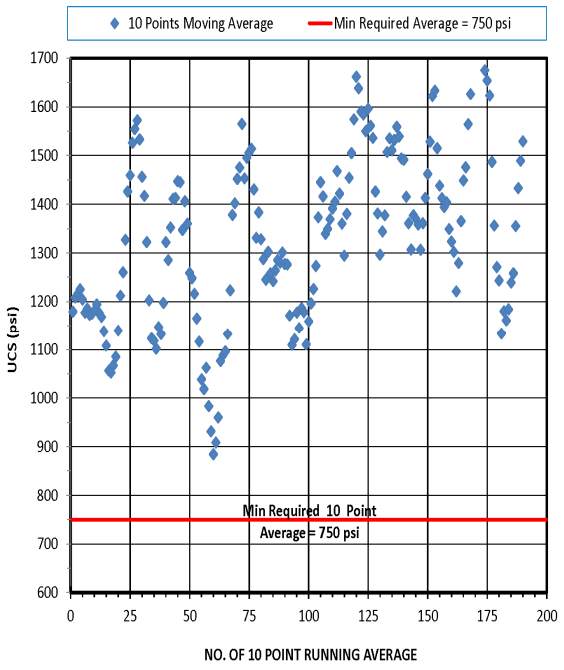


Fig. 9. Core samples UCS 10-point moving average

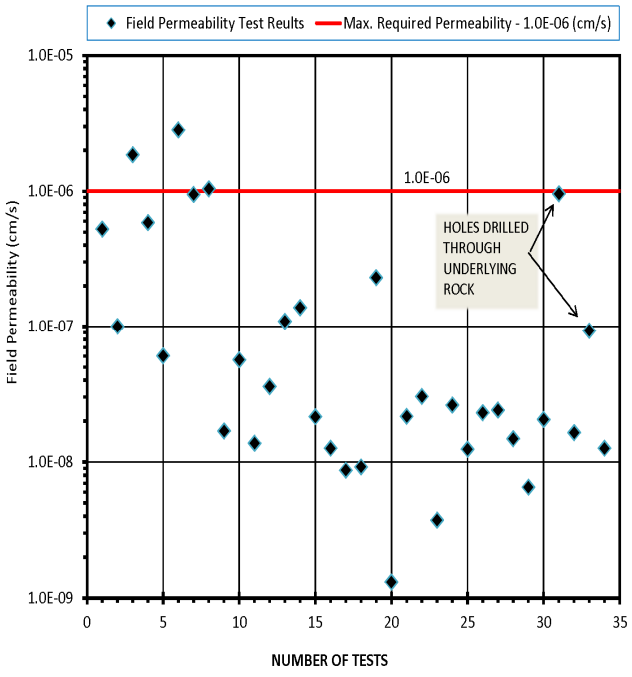


Fig. 10. Field permeability test result summary

© 2016 Deep Foundations Institute

SUMMARY CONCLUSION

Bolivar Dam Seepage Barrier was a challenging project that required immediate attention for the imminent risk related to its seepage problems. Thanks to the construction method implemented, the project was successfully completed earlier than anticipated by the USACE, and with an excellent final product quality, as revealed in the exposed portion of the seepage barrier wall (Fig. 11). The seepage barrier quality was validated qualitatively by extensive verification drilling in the elements and joints which allowed to verify the homogeneity and continuity of the seepage barrier; and quantitatively by in-situ permeability testing and Unconfined Compressive Strength testing that allowed to confirm the final hydraulic conductivity and strength of the seepage barrier fully complied with the design requirements.



Fig. 11. Exposed Seepage Barrier

The project's success is a result in part to the following main factors:

- Having an on-site batch plant, which provided the opportunity and flexibility to test, adapt and optimize the original mix design to the variable needs of the project along its construction.
- The winterized on-site batch plant and facilities, along with maintaining consistent material temperatures, allowed for the continuity of production, even on severe weather days during the Winter
- The manufacturing of the plastic concrete on-site permitted direct and continuous quality control measures on its production from receiving the mix components, followed by the batching of the mix, and quick cycle and delivery times
- The end result of the project proved that the selection to use plastic concrete as a backfill material for the Bolivar Dam produced a high quality seepage barrier that met all the performance requirements

REFERENCES

- USACE, Bolivar Dam bid documents and geotechnical data report

© 2016 Deep Foundations Institute

SEEPAGE BARRIER MIX DESIGN DEVELOPMENT, INSTALLATION, AND EVALUATION

George W. Little, Great Lakes Environmental & Infrastructure, Rocklin, CA, 916-462-6400, georgelittle@gleis.com

John Councilman, Great Lakes Environmental & Infrastructure, Rocklin, CA, 916-462-6400, johncouncilman@gleis.com

ABSTRACT

The selection, evaluation, and modification of contractor mix designs for self-hardening seepage barriers, completed by the slurry trench excavation method, can be one of the most critical and challenging tasks to ensure the cutoff wall meets the design performance criteria. Contractor mix designs should incorporate all available geotechnical data, including subsurface soil characteristics, groundwater information, and potential site contaminates. However, the pre-construction mix design can have limitations if subsurface data is limited or shows variability from initial investigations. Placement of the seepage barrier material needs to be evaluated during construction to identify in-situ performance. In one particular case study, the self-hardening cutoff wall mix design was evaluated over a one year period followed by a year and a half installation period, in which, a number of adjustments were made to the laboratory mix design during construction due to performance in the field. Installation for the selected mix was monitored on a real-time basis, using quality control data, as well as early strength and permeability results to determine if adjustments to the self-hardening mix were necessary. Evaluation of early results and relationships between strength and permeability were used to make adjustments prior to achieving the final test results for 28-day strength and permeability. Test results of the installed mix illustrated a direct correlation between strength and permeability. Achieving a minimum unconfined compressive strength was necessary to meet the minimum permeability of the seepage barrier. Other determinations conclude that the thorough development of a pre-construction mix design can have limitations even when all geotechnical data and site conditions are taken into account. Understanding early indicators and key strength and permeability relationships are necessary to determine seepage barrier performance and limitations.

INTRODUCTION

The evaluation of contractor mix designs for self-hardening seepage barriers was reviewed for one particular case study where a mix was developed using available geotechnical background information, including considerations for impacted groundwater. Following development of the laboratory mix design, the seepage barrier wall was installed over a one and a half year period, where the contractor mix design was evaluated and modified to accommodate field conditions affecting performance. The objective of the project was to cutoff and collect impacted groundwater and prevent the migration of contaminants to the creek running adjacent to the project. The self-hardening seepage barrier wall was installed using the slurry trench excavation method along 4700 ft of alignment and extended to depths ranging from 20 to 85 ft below ground surface, where it tied in with a low permeability stratum or competent bedrock.

© 2016 Deep Foundations Institute

SITE CONDITIONS

Subsurface soils, from top to bottom, generally consisted of fill material, alluvium, glacial sediments, glacial till, and bedrock. In general the layers above the glacial till and bedrock consisted of sands, gravels, cobbles, and boulders with low fines content and higher hydraulic conductivity. Significant constraints included: a remote job site with limited access and narrow working platform, excavation of materials such as cobble and boulders and tie-in to materials with Standard Penetration Test blow counts of >50, sequencing of excavation using a primary and secondary panel installation method, preventing self-hardening slurry from setting during multi-day excavation of panels, multiple crews working a day and night shift seven days a week, utilization of chisel and crane as well as pre-drilling the alignment with a down-the-hole hammer, and significant seasonal groundwater flows impacting self-hardening slurry properties.

CUTOFF WALL EQUIPMENT AND METHODOLOGY

The method used for installation of the seepage barrier was the slurry trench excavation method using specialized long reach excavators equipped with custom built boom and stick. For this project, three excavators were used: a PC 800LC-8 with custom long boom and stick, PC 1250LC-8 with custom long boom and stick, and Koehring 1466 with custom long boom and stick. In addition to the specialized long reach excavators, a stock PC 800LC-8, a Liebherr HS855SX crane and chisel, and HD 180 down-the-hole hammer drill rig were used to support slurry trench excavation. Due to difficulty in reaching the low permeability stratum with the specialized long reach excavators, pre-drilling on 3 ft centers along the alignment was performed in selected locations. Where boulders or hard subsurface layers were encountered that could not be removed using the excavator, the chisel was dropped through the slurry filled trench to fracture the layer so that the excavator could be used to continue excavation to the low permeability stratum.

The type of self-hardening seepage barrier selected for the project was a slag cement-bentonite slurry with a polymer added to control slurry viscosity and to delay set time. The performance criteria for the self-hardening barrier wall was a minimum unconfined compressive strength (UCS) of 50 psi and maximum of 300 psi and a maximum hydraulic conductivity of $5x10^{-7}$ cm/s at 28 days. Unconfined compressive strength testing was performed in accordance with ASTM D2166 and hydraulic conductivity was testing performed in a flexible wall permeameter in accordance with ASTM D5084. This type of seepage barrier wall was selected based on significant site features which limited the effectiveness of other types of seepage barrier walls. During the feasibility investigation phase of the project, a soil bentonite seepage barrier was an alternative option; however, due to slope stability concerns of tailing piles adjacent to the alignment of the barrier wall and the grain size distribution of the subsurface material, a soil bentonite seepage barrier was not viewed as an effective alternative. This option was initially considered since it is traditionally more economical than self-hardening walls. This would also allow for longer excavation times if boulders or obstructions were encountered without the risk of the bentonite slurry setting during excavation.

MIX DESIGN DEVELOPMENT

Some key consideration for development of contractor mix designs for self-hardening seepage barriers include: performance criteria specified for permeability and strength, subsurface soil properties, potential soil and groundwater contaminants, groundwater and batching water characteristics, groundwater flow characteristics, and availability of materials. In addition, contractor mix designs must also consider the

© 2016 Deep Foundations Institute

method and equipment used for installation and the time anticipated to reach the required depth of the panel or trench. Mix design development for this project involved considerations for groundwater dilution, effects of low pH groundwater, inclusion of site soils suspended during excavation, material set time, minimum slurry density, pumpability, and performance criteria. In-situ soils, site groundwater samples, and surface water were collected for use in the mix design. Reagents for the self-hardening slurry mixture included: Ground Granulated Blast-Furnace Slag, Grade 100 or higher, in accordance with ASTM C989, Portland Cement Type I-II, in accordance with ASTM C150, sodium cation base montmorillonite powder bentonite conforming to API Spec 13A, and a polymer admixture to control slurry viscosity and to delay set time. Previous analysis of site groundwater showed locations along the barrier wall alignment with sulfate concentrations up to 1710 mg/L, the presence of dissolved metals, such as aluminum, cadmium, copper, iron, lead and zinc, hardness up to 700 mg/L, and groundwater with a pH as low as 4.24. Mixes were developed and tested to evaluate performance and long term compatibility, particularly for acidic groundwater conditions which could dissolve cementitious material in the slag cement-bentonite matrix over time.

Due to the site groundwater characteristic, an extensive pre-construction mix design effort was conducted. A total of seven mixes, with various proportions of materials and water to cement ratios were tested during initial screening. Samples were mixed with ranges of slag to Portland cement ratios ranging from 20 to 40 (by weight), bentonite to water ranging from 3.9% to 6% (by weight), water to cementitious ratios ranging from 5.7 to 10.8 (by weight), and 0.008% (by weight) polymer admixture. Each trial mixture was prepared using water from the creek running adjacent to the site where water would be obtained for batching during construction. Analytical testing for the creek water showed a pH of 7.6 to 7.8 and hardness of 24 to 28 mg/l. Included in each mix was the addition of groundwater, obtained from the site, to simulate dilution of the mix while in the trench. The ranges of bentonite to water and water to cementitious shown account for addition of groundwater. Each mix was then cast into cylinders and allowed to cure for three days in a temperature and humidity controlled room. After three days, the cylinders we cut vertically four times to expose the samples. Exposed samples were then placed in a water bath of sodium sulfate to simulate the low pH groundwater environment (pH 4.2) that they could be exposed to on the site. Each mix was then tested for unconfined compressive strength at 14, 28, 60, and 91 days and permeability at 52, 73, 109, and 116 days after casting. Based on the test results, all mixes meet the minimum 28-day strength of 50 psi and the maximum permeability of $5x10^{-7}$ cm/s at 52 days. At 28 days unconfined compressive strengths ranged from 62 to 287 psi and 52-day permeability results ranged from $9.1x10^{-9}$ to $3.1x10^{-10}$ cm/s. 28-day permeability results were evaluated during final mix design development. Observations of test results showed that in general, compressive strengths did not increase significantly from 28 to 91 days and in the case of one sample, the compressive strength decreased. Permeability results at 116 days ranged from $6.9x10^{-9}$ to $1.5x10^{-10}$ cm/s. Additionally, it was noted that even when curing in the sodium sulfate water bath, permeability did not show an increasing trend with time.

In addition to evaluating mix designs under conditions anticipated to be similar to those on-site, initial mix screening also involved analyzing the effects of various bentonite to water proportions on the mix filtration and bleed. The addition of bentonite to a cement based slurry can help to control the stability of the mixture by reducing the amount of filtration water expelled from mixed slurry under the hydrostatic pressure encountered in the trench and control the amount of bleed water that occurs on the surface of the trench during initial set of the mix. Increasing the bentonite concentration has the effect of lowering filtration and bleed; however, too high of a bentonite concentration has the effect of lowering the unconfined compressive

© 2016 Deep Foundations Institute

strength of the slurry. In the slag cement-bentonite slurry, various proportions of bentonite to water, ranging from 3.5% to 8% (by weight), were analyzed for filtrate loss in a filter press in accordance with API 13B-1. For the range of mixes evaluated, bentonite to water mixtures between 5.0% and 6.5% (by weight) showed a reduction in filtration while meeting the compressive strength requirements. This range also allowed for some dilution with groundwater encountered during excavation without significantly affecting performance. Based on the initial mix screening results, a 6% bentonite to water (by weight) proportion was selected for the additional mix design screening.

Final mix design confirmation was performed to evaluate the selected reagents and proportioning, determined during initial screening, for performance under a range of curing water conditions as well as with and without the inclusion of site soils anticipated to be suspended in the slurry during excavation. The final phase of the mix design development also included testing for long term compatibility under conditions similar to those anticipated. Two mixes were prepared for the final mix design. Each mix include 7.5% site groundwater (by volume) to simulate dilution of the mix while in the trench. One mix also included 5.4% site soils (by volume), primarily consisting of sandy soils, which were anticipated to be suspended in the slurry during excavation. Each mix was cast into cylinders and cured in the same manner as performed during initial screening. During this iteration of the mix design, each mix was cured in four acidic bath solutions to evaluate the potential range of curing environments. The pH range measured for each curing bath were as follows: low impact, pH 5.8 to 7.0, moderate impact, pH 4.6 to 6.9, high impact, pH 3.8 to 4.9, and worst case impact, pH 3.3 to 4.7. Unconfined compressive strength results at 28 days ranged from 77 to 91 psi and the average permeability at 28 days was 3.4×10^{-8} cm/s, with no samples outside of project specifications.

Long term compatibility of the mix was evaluated by performing falling head permeability testing over a longer duration than typically performed. It should be noted that permeate water used for testing was the same as the corresponding curing bath listed above, that is, samples cured in low impact water also used low impact water for permeate and so forth. Due to the low rate of permeability of the self-hardening slag cement-bentonite matrix, these sample were cast into special molds that had a height of 1 in and diameter of 4 in. This created a greater area and shorter distance for the permeate water to travel, thus increasing the amount of fluid passing through the sample in a shorter duration than with standard sample sizes. In order to measure the amount of cementitious solids that could potentially leach out of the slag cement-bentonite matrix over time, the pH, electrical conductivity, and total dissolved solids were measured in both the influent and effluent permeate water during permeability testing. A trend was then developed to relate electrical conductivity to the total dissolved solids as measured in the influent and effluent permeate water. The laboratory trend developed allowed the use of electrical conductivity measurements in permeate water, which could easily be measured in the lab and could then be used to calculate the total dissolved solids representing leached cementitious material. It should be noted that analytical testing of the materials leached in the effluent permeate water was not performed. For each sample, electric conductivity of the influent and effluent permeate water and permeability rate measurements were recorded at set intervals. Over a period of approximately three months, measurements were recorded two times per week. For all samples a decrease in permeability was observed with time. For samples tested with low impact, moderate impact, and high impact water, each showed a trend towards decreasing electrical conductivity with increasing total effluent through the sample. The sample tested with worst case reactivity water showed a trend of increasing electrical conductivity with increasing total effluent through the sample; however, it was observed that after approximately three weeks of measurements electrical conductivity was relatively consistent for the

© 2016 Deep Foundations Institute

remainder of the readings. It should be noted that during preparation of the worst case permeate water, an error was made in the amount of sodium sulfate added to the mixture resulting in a concentration roughly 3.3 times higher than intended and is not anticipated to be representative of site conditions. Also, the high impact permeate and cure water represents conditions similar to the worst case measured on-site. Based on results of mix design testing, the mix determined for field implementation was a combination of the materials listed above with a water to cementitious ratio of approximately 7.4 (by weight), slag to Portland cement ratio of 20 (by weight), and bentonite to water of 6% (by weight).

Contractor mix design development, for the example discussed, included considerations for a number of factors. For some projects this is not always the case. Often times there may be a number of constraints limiting the amount of pre-construction mix design testing, including schedule and budget constraints; however, successful implementation of the mix design in the field should, at a minimum, consider the site specific geotechnical and groundwater characteristics and the performance criteria for the project. Due to the unique conditions presented for the project discussed, the contractor had the opportunity to conduct a thorough mix design, which had considerations for anticipated site conditions with an analysis of the mix performance for each. Even with considerations for anticipated site conditions addressed during pre-construction mix design studies, the contractor must also make allowances for variability of the known subsurface conditions. In the example discussed, the contractor and laboratory had to make some assumptions about the amount of groundwater and site soils that could dilute the mix during construction. The proportions off groundwater and soil included in the neat mix were based on assumed values determined by previous experience with this type of self-hardening slurry mixture.

SEEPAGE BARRIER INSTALLATION AND MIX EVALUATION

Installation of the self-hardening seepage barrier in the field involves the daily monitoring of slurry properties through on-site quality control testing, analysis of results, and approved laboratory testing to evaluate performance in accordance with contract specifications. During this phase of the seepage barrier mix evaluation, it is often the experienced contractor field personnel who are analyzing field testing and interpreting results to determine if the laboratory mix design is performing as intended to meet project requirements. Field personnel should also continue to work with the mix development team and testing laboratory during this phase to ensure successful implementation and seepage barrier construction. Coordination of field teams and mix development teams in the early stages of seepage barrier installation can help to mitigate future or potential ongoing issues with mix design performance in the field.

The goal of the mix design development phase is to establish materials and mix proportions that will meet the project requirements during installation in the field. For field personnel to control and confirm that the mix is consistent with what was designed for the project, the properties of the slurry are tested on minimum intervals during the shift to determine if the slurry is within a specific range of parameters. These tests are the daily quality control test carried out by the contractor. Understanding and interpreting these results can help the contractor ensure that slurry is being mixed and prepared in the proper proportions and can be an indication that conditions in the slurry trench are not adversely affecting the slurry properties and performance. Quality control testing for this project included measuring and testing the bentonite slurry for density, Marsh funnel viscosity, filtrate loss, and pH per the API RP 13B-1. Batch plant and in-trench slag cement-bentonite slurry testing included density, Marsh funnel viscosity, filtrate loss, pH, and sand content

© 2016 Deep Foundations Institute

per API 13B-1. The range quality control results for each test were not stated in the contract specification, with the exception of a minimum in-trench density of the slag cement-bentonite slurry of 69.6 lb/ft^3 to maintain trench stability. Quality control testing parameters were defined during the pre-construction mix design for the approved mix. As previously stated, laboratory testing for field mix performance included unconfined compressive strength and hydraulic conductivity at 7, 14, and 28 days after casting and curing of samples obtained from various depths in the slurry trench. For self-hardening seepage barriers, understanding quality control testing results are critical to contractor success, since typically, initial laboratory testing is not performed until at least 7 days after installation, with final testing not performed until 28 days after installation.

In the field, conditions can often vary significantly from the laboratory. During the initial stages of the project discussed, a number of quality control testing results and laboratory testing results were observed which necessitated adjustments to the laboratory mix design while still utilizing the approved materials from mix design testing. During excavation, it is anticipated that some amount of the finer grain sized material will be suspended in the slurry and some dilution of the slurry will occur due to inclusion of groundwater through soil moisture. When excavating the trench, the slag cement-bentonite slurry will form a filter cake on the trench walls which reduces groundwater infiltration. Slurry density, which is greater than that of groundwater, creates a hydrostatic pressure in the trench which helps to stabilize the trench walls during excavation. Even though the slurry has these effects, there were some locations along the alignment where higher porosity soils and high seasonal groundwater flows may have impacted the slurry matrix during initial excavation by diluting the slurry with additional groundwater.

Some of the testing used as an indicator during initial mix evaluation were comparing Marsh funnel viscosity, density, and filtrate loss before and after introducing slurry into the trench. It was observed that in some instances that Marsh funnel viscosity, measured in seconds, of the in-trench slurry was lower or the same as the sample from the batch plant, while the in-trench density increased and the in-trench filtrate loss increased. Interpreting these results must be done by experienced contractor personnel, as number of factors in the trench could be affecting them but may not necessarily affect the slurry performance. The increase in density in the trench was expected since some amount of sand and finer materials are suspended in the slurry matrix. The increase in filtrate loss can also be expected since the inclusion of soil and groundwater in the slurry matrix changes the properties from that produced at the batch plant. With this in mind, it is expected that with the inclusion of sand and finer materials in the trench slurry, the Marsh funnel seconds measured should be greater than the Marsh funnel seconds measured with slurry from the batch plant. In this case, this was interpreted that there may be greater than anticipated groundwater flows infiltrating and diluting the slurry since this would increase both the trench slurry Marsh funnel viscosity and filtrate loss. These results, however, were scattered for this instance and results of the early unconfined compressive testing and permeability testing were used to evaluate mix performance for the suspected conditions. After reviewing the 7 and 14 day compressive strength and permeability data for these samples, lower than anticipated compressive strength values and higher than expected permeability rates were observed. To ensure the seepage barrier met the performance criteria the mix was adjusted from a water to cementitious ratio of 7.4 to 6.5 (by weight), to allow for additional groundwater dilution of the mixture in the trench. After the water to cementitious ratio was lowered, sample performance was noticeably improved.

© 2016 Deep Foundations Institute

While installing the seepage barrier, another condition encountered that was not fully anticipated in the development of the mix design was the difficulty of excavation and the time required to reach the final barrier wall depth, or low permeability stratum. Due to the characteristics of the subsurface soil and the frequency of boulders encountered, excavation of each panel to depth was more difficult than expected and on average required two to four times longer than expected. This had adverse effects on both the constructability of the project and the characteristics of the slurry mixture. Effects of the longer durations for panel excavation were premature setting of the self-hardening slurry, increased Marsh funnel viscosity, increased slurry density, increased filtration loss, and increased slurry waste and material usage. Self-hardening walls installed using long reach excavators are usually installed with each panel or section completed during one working shift. Considering the unique nature of the site conditions, the contractor and the mix design laboratory consultant worked to make adjustments to the polymer admixture concentration in the slurry to delay set time and keep the slurry a workable mixture in the trench. When panel excavation proceeded past two working shifts, incremental amounts of polymer admixture were increased in the fresh slurry batches and introduced into the trench. Using the on-site quality control equipment enabled the contractor slurry specialist to define slurry characteristic which could be monitor during batching with the additional polymer admixture. Addition was monitored through a metered pump; however, the Marsh funnel viscosity of the batch plant slurry was a key indicator when determining if the regent addition fell within the desired range. As more of the admixture was included in the slurry, the Marsh funnel seconds decreased while the density stayed consistent. When using this method, it was imperative that contractor personnel understood quality control test results. If the dose of admixture was too high, panel set times were significantly delayed. In some instances, unconfined compressive strength did not exceed 10 psi until 21 days after placement. With the delay in set time, the permeability of the material usually did not decrease below $5x10^{-7}$ cm/s until significant unconfined compressive strength was achieved (usually around 50 psi). Close monitoring of the additional admixture amounts and the quality control data of the batch plant slurry allowed longer panel excavation while still meeting seepage barrier performance criteria. For self-hardening wall using this installation method, time of excavation should be considered when evaluating a mix design.

For the contractor to effectively evaluate and analyze mix performance in the field, both the quality control data and the laboratory data should be reviewed continuously. During installation of the seepage barrier, quality control data was consistently used as an indicator for deviations from the desired mix properties. Unconfined compressive strength and permeability results at 7, 14, and 21 days were also reviewed as performance indicators prior to receiving the 28 day test results. Though early strength and permeability results do not provide the contractor an option to adjust mixes already in place, they can indicate the need to adjust ongoing mixing ratios. It should also be noted that with the addition of the polymer admixture, unconfined compressive strength relationships from 7 to 14 and 7 to 28 days aren't observed to be as consistent as with a cement based grout mixture without retarding agents. For this reason, early results should be carefully reviewed with an understanding that lower than expected results may be observed during 7 and 14 day laboratory testing. Due to the long duration of the project, early unconfined strength and permeability relationships were developed for site conditions to help evaluate the mix performance during the initial stages of the project which provided guidelines for mix ratio adjustments. Figure 1 shows the relationship between strength and permeability during initial construction.

© 2016 Deep Foundations Institute

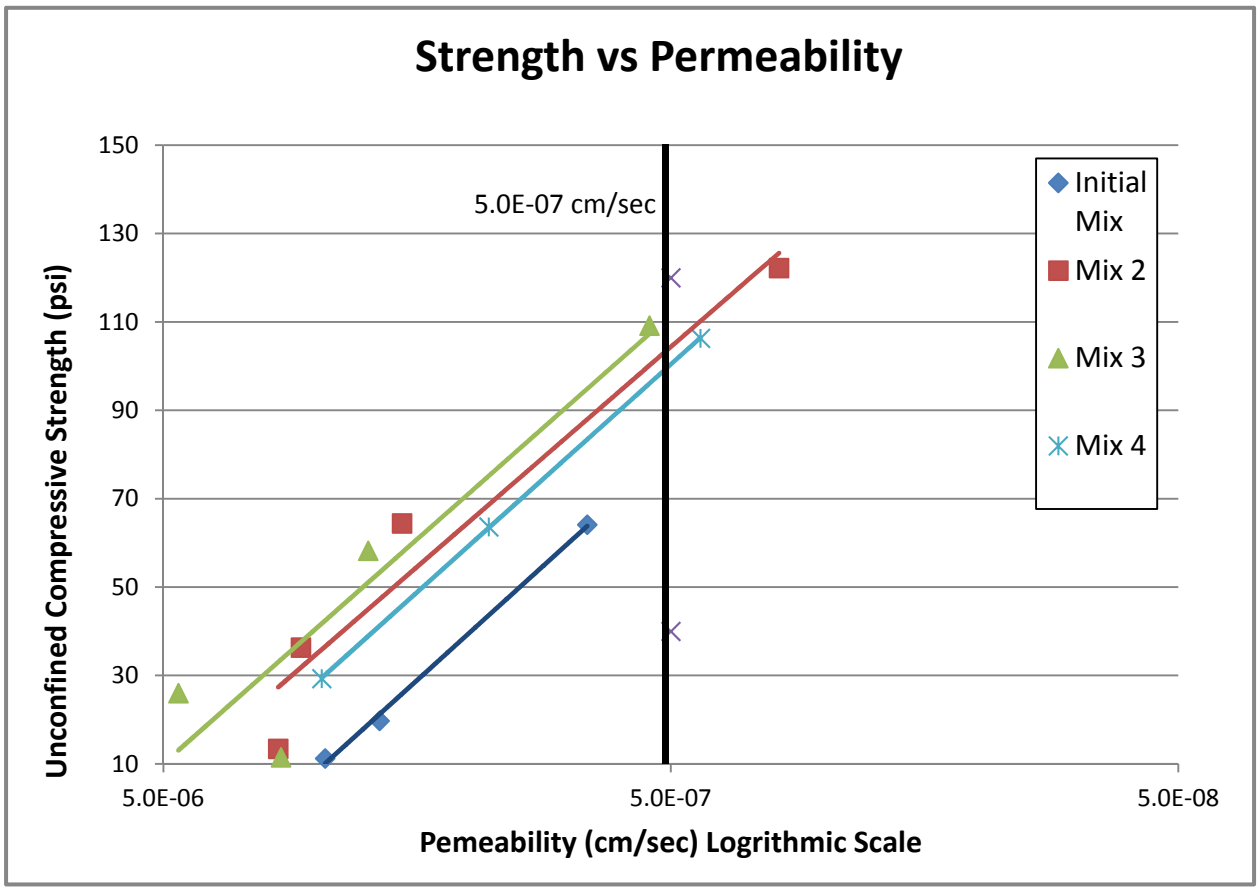


Fig. 1. Initial mix strength vs. permeability relationships showing average strength and permeability relationship for each mix.

As shown in the figure, for the initial mixes used for installation of the seepage barrier, when the strength was over approximately 100 psi, the permeability was less than $5x10^{-7}$ cm/s. As evaluations of the mix performance were made and the mix was adjusted to the final water to cementitious and material proportion ratios, significant improvements in performance were observed. Figure 2 shows the relationship between strength and permeability for the final mix proportions.

© 2016 Deep Foundations Institute

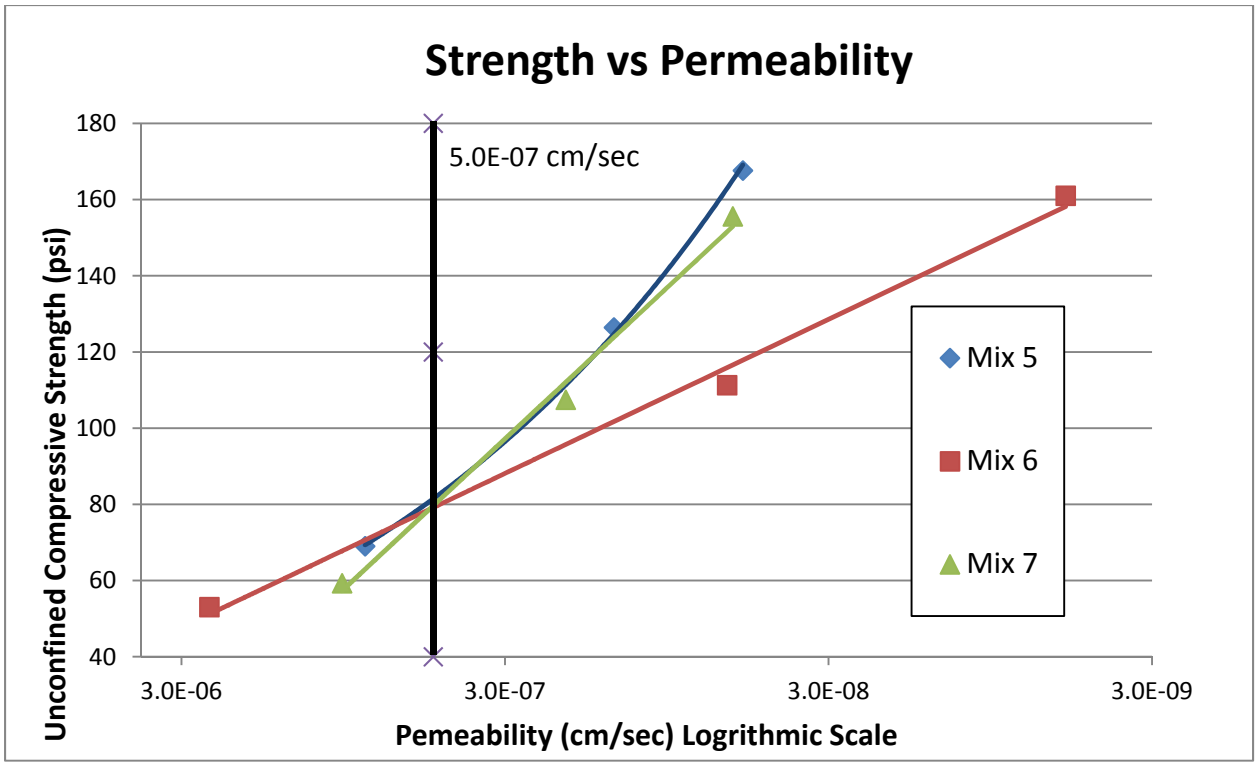


Fig. 2. Final mix strength vs. permeability relationships showing average strength and permeability relationship for each of the variations of the final field adjusted mix.

The final water to cementitious ratios used and the material proportions, shows a significant improvement in performance with curing time. With the final field adjusted mixture, when the strength was over approximately 70 psi, the permeability was less than $5x10^{-7}$ cm/s. It should be noted that data points of average strength and permeability for each mix are shown for 7, 14, 21, and 28 day results in fig. 1 and for 14, 21, and 28 day results in fig. 2. Utilizing some of the data shown above can be useful for early evaluations of performance. Once mix adjustments were made for the project, review of 7 or 14 test data could be used to reliably predict final 28 day results. For the final mix design, if a sample was not trending to or above 70 psi, it generally meant that permeability would likely not be less than $5x10^{-7}$ cm/s at 28 days after curing.

CONSIDERATIONS FOR SAMPLES NOT MEETING PERFORMANCE CRITERIA

Self-hardening seepage barriers are often evaluated based upon an unconfined compressive strength and permeability requirement. Often times field conditions can have some degree of variability along the horizontal alignment and vertically within the subsurface soil. Though a thorough contractor pre-construction mix design attempts to simulate these conditions and mitigate the results, there are occasion where field conditions can negatively affect mix performance and result in samples that fail to meet the performance criteria. One approach that is often recommend by the owner or the owner's representative is to replace the panel or section not meeting the required criteria. This approach can not only be costly for the contractor but can also negatively impact the project schedule; however, it does ensure that the performance criteria is meet. A common approach that was proposed and approved on the project referenced in this paper was to perform testing beyond the 28 day curing period. In most cases, it was observed that

© 2016 Deep Foundations Institute

the permeability of the material continue to decrease after 28 days of curing. Samples for the project were tested as far as 180 days after curing in one instance. For these cases it was often requested that additional testing was performed even after passing results were observed to ensure that permeability was continuing to decrease. This method was preferred since both the owner and contractor benefited by reducing overall contract schedule and cost. In some instances, the contract may allow for a small percentage of sample to be outside of the required range provided the majority of results demonstrate that they meet the performance criteria of the barrier. This can be a useful approach if the design of the project isn't overly sensitive to minor changes in performance. A consideration for this approach would be the intent of the overall project design and the purpose of the seepage barrier. For instance, a seepage barrier at a dam footing may not allow for any samples outside of the specified limits since the effects of a dam failure could be significant. Other approaches not mentioned may also be beneficial.

CONCLUSIONS

During pre-construction mix design development for self-hardening seepage barriers, it is important to have a good understanding of the site conditions, including the geotechnical and groundwater characteristics of the site and any potential contaminants. Thorough pre-construction mix design development can mitigate specific site characteristics which may adversely affect the performance of the seepage barrier. A good practice for the contractor mix design is to have a factor of safety when targeting project required performance criteria, regardless of how thorough the contractor mix design study is. Mix design development should also include considerations for the installation method, as this may require unique mix property characteristics. During the installation phase, the contractor should be familiar with the quality control test data as well as laboratory test data. Understanding relationships between test data allows the contractor to make adjustments to the mix if unaccounted for site conditions adversely affect the performance of the seepage barrier. For self-hardening seepage barriers, developing unconfined compressive strength and permeability relationships allow the contractor to recognize test data trends, prior to final 28 day results, which can be used to determine performance and limitations. When subsurface conditions vary from what is anticipated and the mix does not meet the performance criteria, it is beneficial to make considerations for samples outside of the specified requirement so that the project as a whole meets the intent of the design criteria.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the following individuals for their contribution:
Matt Marks, Tino Maestas, Louay Owaidat, and Chris Harris.

REFERENCES

Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter. Designation D5084 - 10, ASTM International, 2010.

Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. Designation D2166 – 06, ASTM International, 2006.

Specification for Drilling Fluid Materials. ANSI/API Specifications 13A, Seventh Edition, 2008.

Recommended Practice for Field Testing Water based Drilling Fluids. API Recommended Practice 13B-1, Third Edition, 2003.

© 2016 Deep Foundations Institute

USE OF ENHANCED PRE-MIXED SELF-HARDENING GROUT FOR AN EMERGENCY CUT-OFF WALL INTERVENTION IN WEST AFRICA

Bruno Vingiani, Trevi S.p.A., Cesena, Italy, +39 0547 319147, bvingiani@trevispa.com
Claudio Asioli, Trevi S.p.A., Cesena, Italy, +39 0547 319372, casioli@trevispa.com
Maurizio Siepi, Trevi S.p.A., Cesena, Italy, +39 0547 319318, msiepi@trevispa.com

ABSTRACT

The article describes an emergency rehabilitation intervention executed at Buyo Dam, in Ivory Coast, to intercept a significant water seepage occurring through the foundation of this zoned earthfill retaining structure constructed in the late Seventies. To solve the problem, a 800 mm thick, 400 m long impervious cut-off wall (COW) was constructed on the right side of the embankment, operating from the dam crest and with the reservoir almost at its maximum level. The use of an innovative pre-mixed self-hardening grout, with a target hydraulic conductivity coefficient (k) lower than 10^{-9} m/s, was considered the only way to match the tight working schedule and the strict requirements for the filling material of the COW.

Keywords: dam, rehabilitation, cut-off wall, COW, self-hardening grout

PRESENTATION OF THE HYDROELECTRIC SCHEME OF BUYO

The hydroelectric scheme of Buyo, located at 6°14'28" N 7°20'45 W on the Sassandra River, in Ivory Coast (Figure 1), was built in the late Seventies for agriculture and power generation. With a reservoir of 8.3 billion m^3 and an installed power capacity of 165 MW, enough to supply electricity to 111,000 homes, it represents an installation of great significance for this West African country.



Fig. 1. Location and general view of Buyo main dam and lake

As a whole, the hydroelectric scheme consists of the following key structures:

- main dam with a crest length of 3,615 m and a maximum height of 37 m;
- secondary dam with a crest length of 3,335 m and a maximum height of 20 m;
- discharge channel at the bottom of the secondary dam;
- 62 m wide spillway at the top of the main dam;

© 2016 Deep Foundations Institute

- water intake structure;
- control system consisting of a network of piezometers, leveling points and weirs to evaluate the quantity of drained water;
- power house.

The main dam consists in an earthfill embankment with an impervious core in clayey laterite enclosed in a semi impervious silty laterite material (Figure 2). A 2 m thick layer of compacted rockfill protects the upstream face of the embankment, while a 0.55 m thick stratum of compacted crushed stones covers its downstream surface. The drainage of the embankment is achieved by means of three layers of different filtering materials lying under the upstream rockfill and a 1.2 m thick single layer of washed sand placed against the downstream slope of the impervious core. Transversal sand drains located at the bottom of the embankment, together with a series of interconnected deep wells and a collector trench installed at the base of the downstream face, complete the drainage network.

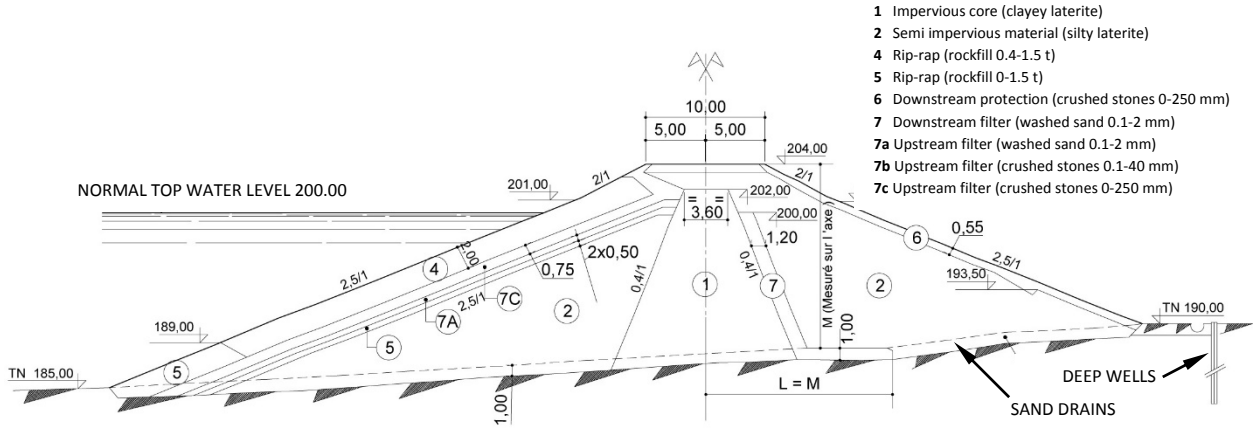


Fig. 2. Typical section of Buyo main dam

At the top of the dam, a 10 m wide paved road allows the circulation of light and heavy vehicles between the two sides of the embankment. The normal top water level is set at El. 200.00, while the minimum operational level is at El. 195.00. The hydroelectric scheme and its appurtenant works are managed by *Compagnie Ivoirienne d'Electricité* (briefly CIE), a private operator which supplies electric power in Ivory Coast under concession agreement with the government.

GEOLOGICAL FEATURES OF THE FOUNDATIONS

The main dam lies on a stratified formation of low permeability sedimentary soils. From the top to the bottom, the following strata are encountered (Figure 3):

2a – gravelly laterite (clay with pisolite);

2b – brownish lateritic clay;

3a – reddish micaceous clayey silt;

4a – highly weathered schists;

5a – weathered schists;

6a – sound schists.

© 2016 Deep Foundations Institute

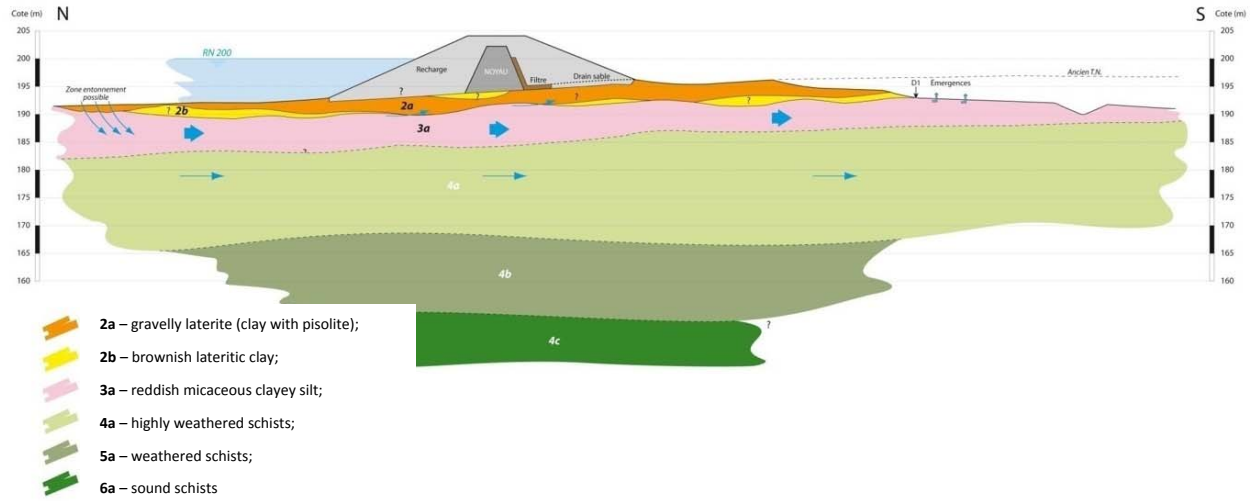


Fig. 3. Schematic geological section of the formations underlying the main dam

Relying on the low permeability of the above-mentioned soils, the implementation of an impervious cut-off wall to intercept the water flow crossing the foundation of the dam was not considered mandatory at the time of construction.

OCCURRENCE OF THE PROBLEMS AND DESIGNED SOLUTION

In 2007, some measuring weirs located at the foot of the dam's right bank started to show a sudden increase in the quantity of water passing through the embankment. Almost simultaneously, some localized ground depressions appeared in the same area, thus raising serious concerns about the overall stability of this section of the embankment. The occurrence of such problems urged the dam's Operator to lower the reservoir to El. 197.00 (i.e. 3 m below the normal top water level) and to entrust the International consulting company *Électricité de France* (briefly EDF) with a wide-ranging analysis of the occurred phenomena and with possible solutions.

As a whole, the study included a detailed assessment of the drainage system's current conditions, a specific soil investigation campaign and a series of in-situ and laboratory tests on soil samples. The results of this comprehensive study revealed that the micaceous clayey silt layer **3a** was the "weak point" in terms of permeability, mainly due to the high degree of erodibility of this formation.

After analyzing different options, the final solution to the problem was found in the implementation - operating from the dam crest - of an impervious cut-off wall (COW) deepened to such an extent as to cross the critical layer **3a** and safely penetrate for some meters into the underlying schists formation **4a** (Figure 4). In order to cover the entire area affected by the water seepage, the plan extension of the COW was set at 400 m (i.e. from Pk. 1350 to Pk. 1750). Moreover, considering an estimated maximum depth of 25 m, the design nominal thickness was 800 mm.

The filling material of the COW was designed to be a self-hardening grout, a cementitious bentonite slurry that behaves as a traditional stabilizing fluid during the excavation phase while, after hardening, it becomes an impervious barrier, "plastic" enough to endure deformations of a certain extent without fissuring.

© 2016 Deep Foundations Institute

In this specific case, the required long-term features of the filling material were specified as follows:

- coefficient of hydraulic conductivity (k) $\leq 10^{-8}$ m/s;
- unconfined compressive strength (UCS) on cylindrical samples in the range 200-2,000 kPa;
- modulus of elasticity (E_t) in the range 20-100 MPa.

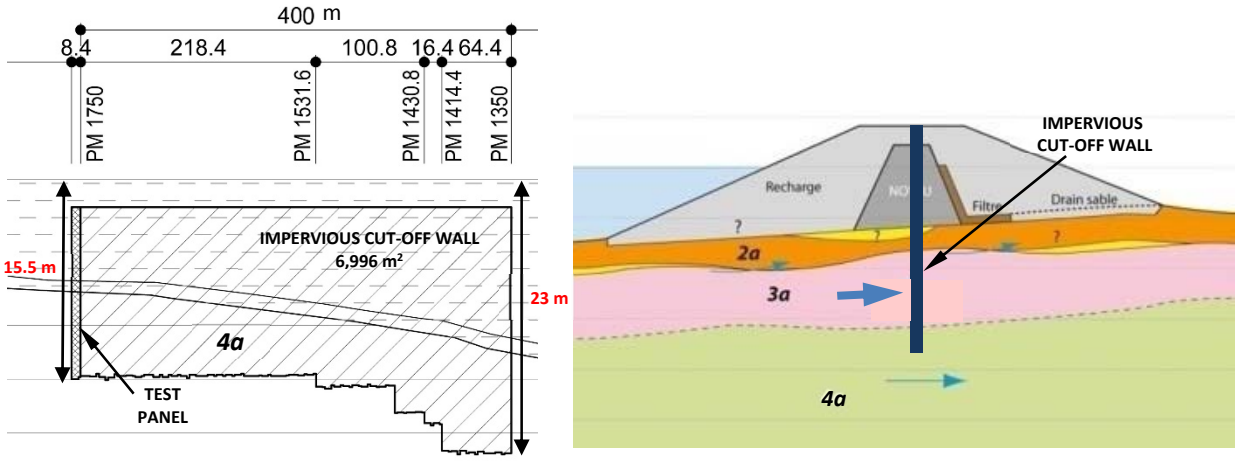


Fig. 4. Schematic configuration of the impervious COW

LOCAL CONSTRAINTS AND CHOICE TO ADOPT A READY-TO-USE FILLING MATERIAL

On July 14th, 2009 CIE assigned the execution of the COW to *Trevi S.p.A.*, under the supervision and quality control of the internationally renowned firm *Bureau Veritas* (briefly BV).

Unfortunately, soon after the first site visit and meetings with all the involved parties, the Contractor realized that some local constraints would clash with the extreme urgency of works. Indeed, the lack of a local bentonite producer and the limited experience of local laboratories to "deal" with a material never used before in the country, would greatly affect the tight work schedule.

For the above-stated reasons, after a comprehensive evaluation of different options, the Contractor decided to go for an innovative solution, thus importing into the country not only the missing raw material, but the entire blend of dry components which are needed for the production of a self-hardening grout. Consequently, based on experience gained in the application of similar solutions in Italy, the Contractor decided to involve in the project the cement company *Buzzi Unicem S.p.A.*, which produces such a pre-mixed material under the proprietary name of Solidur®.

Solidur® is a pre-blended powder mix of cementitious mineral binders, high-grade bentonite and specific admixtures produced according to a standardized industrial process under a tight quality control. The main advantage in using this material is that, once a sample of water to be used at the site is delivered to the Manufacturer (together with the project requirements), the design of the mix and the entire pre-qualification phase is carried out at the factory's laboratory under the full responsibility of the Manufacturer itself.

© 2016 Deep Foundations Institute

In few words, the Manufacturer assures the final performance of the self-hardening slurry, entrusting people at site with the only task of mixing in the right proportions Solidur® powder with water and systematically checking the conformity of the characteristics of the produced self-hardening grout with those specified by the producer (Figure 5). In this specific case, it is worth to mention that the pre-blended mix design and pre-qualification phase lasted only one month, so that the first supply of Solidur® could reach the site by sea freight in less than 1.5 months.



Fig. 5. Filling of the bags at the cement factory and production of the self-hardening grout at the site

EXECUTION OF WORKS

Thanks to the choice of using an imported pre-mixed material, COW construction works at site started on September 4th, 2009, less than two months after the date of the contract and soon after the completion of the necessary guide walls. To be noted that, in order to arrange a 3.65 m wide free lane for the local traffic's transit from both sides of the dam, the alignment of the COW was set 1 m downstream the dam axis (Figure 6).

As common practice for plastic COW, the construction sequence envisaged the alternate execution of primary and secondary panels, with the latter overlapping the edges of the first to an extent defined according to the maximum depth to be reached and the maximum deviation of the excavation tool. In the case of Buyo dam, 6.5 m long primary panels were alternate to 2.5 m long secondary panels (Figure 7).

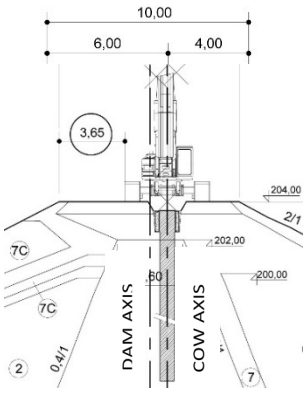


Fig. 6. Alignment of the COW

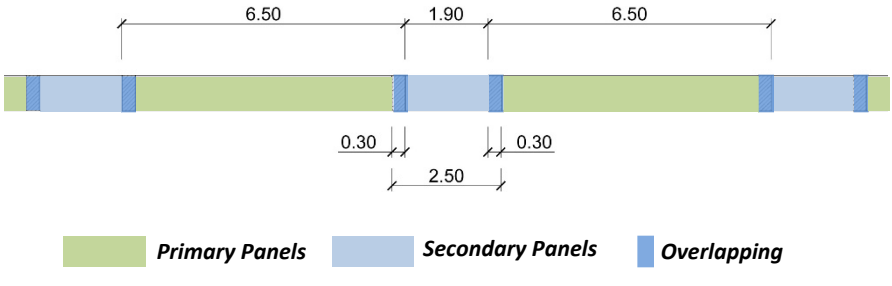


Fig. 7. Typical execution sequence and panel's overlapping

© 2016 Deep Foundations Institute

Taking into consideration the limited depth to be reached and an anticipated maximum deviation of the trenching tool in the order of 0.5% of the depth, an overlapping of 30 cm between the panels was considered sufficiently safe to ensure the necessary continuity of the COW.

For the excavation of the panels, a Liebherr 853 foundation crane and a 800 x 2,500 mm Soilmec BH-12 hydraulic grab were employed (Figure 8). Due to the logistical restrictions and the consequent impossibility to create a stocking area at the top of the dam, the grab discharged the excavated soils onto trucks which shuttled between a dedicated area at the downstream foot of the dam and the working area.

Despite the limited excavation depth, verticality and rotation of the trench under excavation were measured and recorded by means of a Jean Lutz Taraben biaxial inclinometer system fixed at the top of the grab. Measured deviations were always lower than the specified maximum tolerance (i.e. 0.5% of the final depth).

COW construction activities at site lasted 103 calendar days (including the execution of a test panel at one extremity of the impervious barrier) without facing major operational problems and without inducing any movement to the structure, which was kept under constant control during the execution of the works. The final average overconsumption of self-hardening grout compared to the theoretical excavated volume was 41% for primary panels and 37% for secondary panels.



Fig. 8. Execution of the COW from the crest of the dam

ROUTINE AND FINAL QUALITY CONTROLS

Routine quality controls on the self-hardening grout at site included systematic checks on the main rheological characteristics of the mix soon after its production and laboratory tests on hardened cylindrical samples.

The characteristic of the freshly produced mix kept under constant control were:

- specific weight;
- Marsh viscosity;
- bleeding at 30', 1 h, 2h and 4 h.

© 2016 Deep Foundations Institute

The samples to be tested at the laboratory were collected in cylindrical samples and cured at the site in a water basin under thermostatic controlled conditions. After a minimum curing time of 10 days, the samples were transferred to the independent laboratory in charge of the tests. It is worth noting that, in order to obtain reliable and truthful results, two different laboratories were involved in the execution of the tests, the first of which was a local laboratory in Abidjan, while the second was in Italy.

All self-hardening grout samples tested by the independent laboratories showed strength, deformability and permeability values within the range specified by the Designer. As far as permeability is concerned, the measured values of the hydraulic conductivity coefficient (k) ranged between 10^{-8} m/s and 10^{-10} m/s.

As a final check on COW's continuity and integrity, seven vertical cored holes were performed through it, in correspondence to the joints between primary and secondary panels (Figure 9).

To further assess the permeability of the wall, *Lefranc* type water tests were performed (at different depths) inside these holes while coring. Also these final tests confirmed that the permeability of the wall was in line with requirements.



Fig. 9. Execution of vertical cored holes through the COW in correspondence of the joints

CONCLUSIONS

Experience attained from this case history demonstrates that the use of a pre-blended self-hardening powder mix (type Solidur® or equivalent) may represent a valuable and effective solution in all cases where the timely execution of the COW is essential for the achievement of the intervention. This is even more evident when works are carried out in countries where even just one of the raw materials needed to produce the self-hardening grout must be anyway imported from abroad, due its unsatisfactory quality or unavailability in that country. As matter of fact, the certified quality of the raw materials, the pre-qualification of the mix directly carried out by the Manufacturer, the constancy of the powder mix guaranteed by an industrial fabrication process and, last but not least, the easiness of producing the final grout mix at the site can fairly compensate the import costs.

© 2016 Deep Foundations Institute

ACKNOWLEDGMENTS

The Authors wish to thank all companies and single persons that have in some way contributed to the realization of this safeguarding intervention. The truthful cooperation and support received from CIE, EDF, BV and all other Entities has been the true reason for the successful completion of the works.

© 2016 Deep Foundations Institute

WADI DAYQAH DAM CURTAIN

Frank Haehnig, Züblin Spezialtiefbau GmbH, Stuttgart, Germany, +49 711 7883 749, frank.haehnig@zueblin.de

Dr. Harald Krenn, Züblin Spezialtiefbau Ges.m.b.H., Vienna, Austria, +43 1 22422 2605, harald.krenn@zueblin.at

Harm-Christoph Claassen, Züblin Ground & Civil Engineering L.L.C., Dubai, U.A.E., +971 43344 324, harm-christoph.claassen@zueblin.ae

ABSTRACT

The Wadi Dayqah Dam Water Supply Scheme represents one of the most important projects implemented by the Sultanate of Oman to ensure sustainability of water resources. The principal components of the project comprise two dams to impound the Wadi Dayqah, to store and supply drinking water to the local area and to Muscat, the capital city of the Sultanate.

The Main Dam with a central spillway is a RCC Gravity Dam with a maximum height above foundations of 75m, crest length 400m; the Saddle Dam is a rockfill dam with central clay core, max height above foundations is 49m and crest length 360m. During construction of the Saddle Dam a key trench was excavated to 10m depth and filled with compacted clay connected to the central core of the embankment. Grouting utilizing cement grout was performed as a single line grout curtain prior to each filling operations. Both Dams are founded on soft Siltstone, Claystone, Sandstone and inter-bedded harder limestone.

After construction of the Saddle Dam, once water was in the reservoir it became evident in early 2010 that some seepage was occurring apparently from the right abutment. This seepage became more marked when the reservoir first filled to discharge over the spillway in June 2010. In order to reduce the seepage, additional sealing works by drilling and grouting of a three row curtain up to 80 meter target depth were initiated to supplement the existing single row curtain. Part of this project for the sealing works has also been to drill additional investigation holes to provide more information and confirm extend of the additional sealing works and its limits.

Züblin Ground Engineering was awarded the Additional Sealing Works Contract. Whilst Equipment was on Site, the opportunity was taken to carry out drilling and grouting on the Left Abutment of the Main Dam to reduce seepage through that abutment and to install additional standpipe piezometer for future monitoring of seepage patterns.

Work on this Contract started on site in January 2013 and was substantially completed in January 2015.

At the time of Tender Züblin Ground Engineering propose and Alternative Design in the form of a Secant PileCut-Off Wall, 85m long to a depth of 40m with additional drilling and grouting below the foot of the cut-off which was then included in the contractual scope.

© 2016 Deep Foundations Institute

KEYWORDS

Grout curtain, top down grouting, bottom up grouting, secant pile cut-off wall

INTRODUCTION

The site for the Additional Sealing Works lies between the site of the Wadi Dayqah Main and Saddle Dam construction site, approximately 80 km south east of the Sultanate of Omans capital Muscat; the Main Dam with the Spillway on the line of Wadi Dayqah and the Saddle Dam at a higher level about 300 m away on the opposite side of a large rock outcrop that divides the valley.

Additional Sealing Works were necessary to be carried out by drilling and pressure grouting in stages in order to reduce the observable seepage on the Right Abutment of the Saddle Dam (**Figure 1**) and also from the Gallery at the Left Abutment of the Main Dam. In Addition to the drilling and grouting works at the Saddle Dam, an Alternative Design included the construction of a Secant Pile Wall to a depth of 40 meter from the crest.



Figure 1: Wadi Dayqah Saddle Dam - Additional Sealing Works at Right Abutment

Prior to the secant piling and grouting works, fully cored site investigation holes were drilled in the Right Abutment of the Saddle Dam placed within the existing grout curtain to help to define more fully the extend of the sealing works to be done. Further investigation holes were drilled at the Left Abutment of the Main Dam and the Saddle Dam during the on-going sealing works. As part of the quality control of the additional sealing works the contract also included in-situ test such as water pressure tests (Lugeon) and onsite laboratory tests on the grout mixtures being used.

The installation of the additional grout curtain involved vertical and inclined grout holes in three rows to a general target depth up to 80m and pressure grouting in stages parallel to the center line of the crest and to

© 2016 Deep Foundations Institute

the original grout curtain. Locally depth of grout holes was necessary to extend to a maximum depth of 125 meter. The downstream row was grouted first, followed by the upstream row with central row being grouted, where required last. Successive half-hole spacing was adopted, i.e. an additional borehole was located mid-way between two previously grouted holes until the closure criteria were satisfied. The indicative closure criteria for this contract was a Luegon Value of less than 2 prior to grouting of tested hole or a grout take of less than 25kg of cement per meter.

Drilling at the Saddle Dam was performed using three-wing drilling bits when drilled through the dam core. For drilling in rock below the dam core a down-the-hole hammer system using water flushing media (Wassara-W80) was used, providing a hole size of 95mm. To prevent likely damages to the clay core caused by drilling operations, temporary casing were used in all holes to stabilize the hole during drilling and to avoid flush out of the clay core material when drilled and grouted below the dam core. Holes where core recovery was required were drilled using a wireline system providing a minimum hole diameter of 122mm. The drilling rigs used for the drilling with and without core recovery were capable of drilling up to depth more than 200m.

All Grout holes located at the Main Dam were drilled using the wireline coring technology in order to investigate the rock condition and the original grout curtain. In total, 24.000 m of boreholes were drilled, pressure grouted and water pressure tested.

It was agreed to follow a quasi GIN method for grouting. Utilizing Primary holes of the downstream row as trial grout holes, grouting pressures, volumes and GIN numbers were established and grout mix design were chosen. General grouting method adopted were bottom up grouting consisting of drilling to the full depth initially followed by grouting from the bottom of the hole upward in different depth or zones. However, at the interface zone between the clay core and the fractured rock where unstable conditions and high Lugeon Values as well as high grout intake was observed, stage grouting was adopted consisting of drilling the holes to a limited depth of 5 meter within a zone and grouting to that depth. After the injected grout attained its initial set, holes were drilled to a further depth and then grouting again in 5 meter long stages (top down grouting). Thus the holes were successively drilled and grouted until the required depth.

A continuous flow of grout to the inflatable packer inside borehole at pressures up to max. 12 bar and a minimum flow rate of 2ltr/min was provided by the HPI Grouting Unit. This Grouting Module is equipped with 4 double plunger pumps, 2 mixing tanks of 350 litres capacity and Injection Data Record software allowing to monitor and record the pressure and flow at real time.

The construction of the secant pile cut-off wall consisted of drilling from the dam crest up to 40m depth under a ready-made diaphragm wall mixture supplied from HeidelBergCement Germany. The execution of the wall only commenced on completion of the downstream row grouting in order to prevent any sudden loss of the wall mixture in untreated rock.

GEOLOGICAL DATA

In 2011 as result of the seepage observed after the first filling of the reservoir, an additional geotechnical investigation was carried out. Total 5 No fully cored investigation holes were drilled in the Right Abutment and Right Shoulder of the Saddle Dam to try to identify the main areas of seepage through or under the existing grout curtain. Geological conditions on site can be described as complex with alternately layers of soft Siltstone, Claystone, Sandstone and inter-bedded harder Limestone (**Figure 2**).

© 2016 Deep Foundations Institute

The rock strata are heavily folded and the dip of the rocks is variable. Some of the rocks are weakly cemented. From previously investigations the presence of Gypsum veins was known.

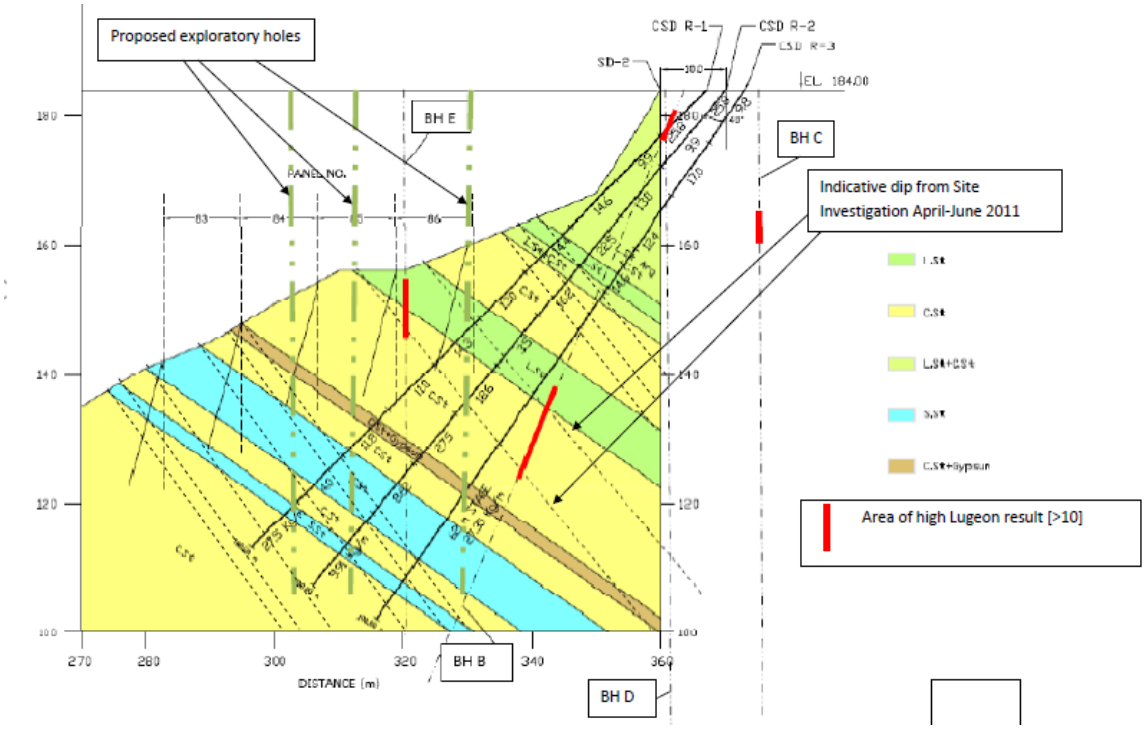


Figure 2: Geologic Section along Center Line of Saddle Dam, Right Abutment

According to the investigation report boreholes BH-C and BH-D carried out from the left shoulder of the saddle dam a layer of nodular limestone was encountered intersected by argillaceous and arenaceous limestone layers were down to the termination depth of 150m.

In BH- B a nodular limestone layer was encountered up to 16.00m. Below this layer moderately weak to strong calcareous claystone layers from 16.00m to 31.00m and argillaceous and arenaceous limestone layers from 31.00 were intersected down to the depth of 78.00m. Very weak to weak friable calcareous sandstone layers inter-bedded by claystone were observed from 78.00m up to a depth of 92.00m followed by a calcareous sand layer down to the depth of 102.00mm.

BH-E was drilled on the saddle dam through the clay core of the dam into the rock. From the Dam/rock interface down to 62.50m, arenaceous and argillaceous limestone layers with cement grout intercalations were penetrated. Silty clay and clayey silty sand layers were encountered from 62.50m to 73.80m depths. Towards the termination depth of 80.00m friable sandstone layer was penetrated.

Water Pressure Tests performed in the investigation boreholes to determine the permeability of foundation reflected that there are areas of high Lugeon values up to 135(l/min/m) starting approximate 55m below the dam crest and immediately below the dam foundation.

© 2016 Deep Foundations Institute

During the Additional Sealing Works Contract, further 15 No site investigation holes were drilled only at the Saddle Dam. 7 No were drilled on the Right Shoulder towards the center of the Saddle Dam, 5 No on the Left Shoulder, 1 No in the center of the Dam and 1 No on the Right Abutment. The majority of these investigation holes were drilled to a depth of 80m from the crest of the dam but some extended to 105m depth.

In all holes water pressure test were executed in up to 5 meter long stages. It became evident that almost 2 years after the last Site Investigation the condition at the Right Abutment of the Saddle Dam has changed in respect to the permeability at the interface area between Dam Clay Core and the bedrock. For example, where the previous Investigation did show acceptable permeability, the Investigation holes J, H1, F, G, K, N & L later did show high Lugeon Values in a range of 20 to locally 190 LU at the depth from 20m to 80m immediately starting at the dam/rock interface. Only BH O did show considerable low Lugeon Values of up to 4 LU.

As noted above, Lugeon tests were performed in each site investigation hole, however, at most locations the actual interface of Clay Core and bedrock was not tested as it was not advisable to apply pressure to this very critical area. A loss of flushing water while drilling in the interface area was taken to provide sufficient indication of a problem, although this may have resulted from applied fluid pressure from the drilling process.

At the Main Dam initially 7 No site investigation holes were drilled and tested on the Left Abutment. All holes except one gave low Lugeon values of less than 7 LU. In hole MD6 at the interface of the RCC and roch the Lugeon value was 78, so this hole was grouted in 5 meter stages. The stage close to the RCC/rock interface took 718 liters. Further investigation at the Main Dam was carried out as an integral part of the drilling and grouting exersice using data obtained from Lugeon testing and the grout-take records during the drilling and grouting from the gallery.

SECANT PILE WALL

At the time of Tender the Employer has advised the Contractor to submit an Alternative Offer if form of a deep positive cut-off. The minimum criteria for the cut-off was a depth up to 80 meter from the crest and a continues length of 85m (60 into the Saddle Dam and 25m into the Right Abutment. The minimum effective thickness of the wall was required to 500mm at any point and permeability not greater than 10-7 m/s. Provisions would still be required for supplementary drilling and grouting below the foot of the cut-off wall to a maximum depth of 125m.

As at that time of Tender Züblin Ground Engineering had no Equipment such as a Trench Cutter available for to reach required depth, an alternative Design for a Secant Pile Wall 85m long to a depth of 30m was made, in addition to the drilling and grouting. The proposed depth was limited to 30m to achieve sufficient overlapping at the toe of the wall considering deviation of piles during drilling. However, following negation, the depth of the secant pile wall was increased to 40m and the Secant Pile Wall with pile diameter of 1070mm was then included in the Contract (**Figure 3**).

© 2016 Deep Foundations Institute

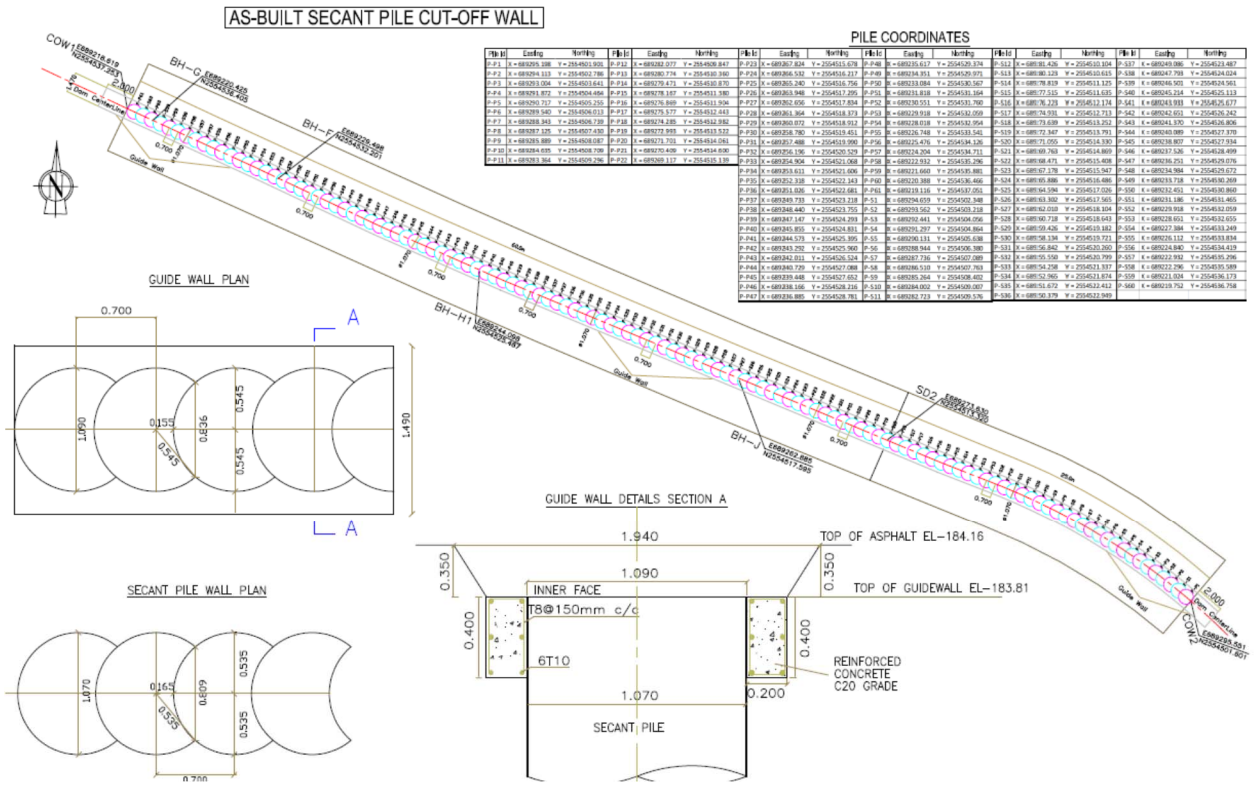


Figure 3: Plan View and Sections of Secant Pile Wall at Right Abutment of Saddle Dam

Prior to the installation of the Pile Wall a guide structure had to be constructed to ensure accurate plan position during drilling. Styrofoam forms with a diameter of 1090 mm were used as internal shutters. All piles have been drilled cased to a depth of 22 m. Individual piles are 1070mm dia at 700mm centres giving an intersection overlap of 370mm. In alternating succession male and female pile were drilled. While drilling cased, augers and core barrels were used as drilling tools. When drilled below the casing, the borehole was flooded first with a diaphragm wall building material supplied from HeidelBerg Cement called DiWa-mix 230. DiWa-mix diaphragm wall mixtures are high-quality ready-made product with a strong resistance to seepage waters. They are made from a special binding agent and select clay components and are therefore particularly environmentally safe. The product is simply mixes with water and is ready for immediate use. No swelling time is necessary.

After filling of the piles with DiWa-mix, drilling commenced then with core barrel and buckets to final depth. On completion of drilling the trajectory of each pile was measured using a Jean Lutz sensor and the deviation from vertical plotted allowing the horizontal cross section of the wall to be determined at any depth. In order to control verticality of the piles when drilled below casing, a 4 meter long core barrel was used. With the exception of a few areas of reduced thickness in the lower part of the wall, the installed wall lies within the design criterion of effective thickness of 500mm; the few areas of marginally reduced thickness were accepted on the recommendation of the Employers Panel of Experts.

The viscosity of the wall mixture was controlled by the Marsh cone. The aim was to achieve a Yield stress of approx. 20 N/m², the Filtrate loss was limited to less than 60ml and the sedimentation of the slurry after 24 hours was limited to less than 33% within 24 hours. The Uniaxial compressive strength after 28 days

© 2016 Deep Foundations Institute

shall not be less than 4N/mm² combined with a coefficient of permeability at 28 days less than $1x10^{-10}$ m/s.

As it was considered to be difficult to set the packer later on at the pile toe in the soft DiWa-mixture, the last 2 meter of the pile toe was grouted with a thick cement grout achieving a target strength of minimum 20MPa after 28 days and having a high density required to displace the fresh DiWa-mixture. Immediately after completion of the pile drilling a self-made injection nozzle system was lowered into the fresh DiWa mixture down to the toe of the pile and grout was injected. The displacement of DiWa mixture was monitored closely to ensure a success of the operation.

Also the core drilling executed later required on for the center line of the grout curtain did proof that the use of such system was successful as the interface between the DiWa-mix and cement grout was clearly visible.

THE GROUT CURTAIN

The Grouting Works at the Right Abutment of the Saddle Dam comprised three rows of grout holes drilled with and without core recovery to a general depth up to 80m and pressure grouting in stages to extend the existing grout curtain. Locally depths of drilling and grouting were instructed up to a maximum of 125m. The drilling for the grout curtain was executed through the overburden and the dam clay core at the Right Abutment of the Saddle Dam and within the Gallery on the Left Abutment of the Main Dam. The Saddle Dam Holes at the upstream and downstream line were drilled vertical, all remaining holes at the centreline inclined up to 45 Degree from vertical.

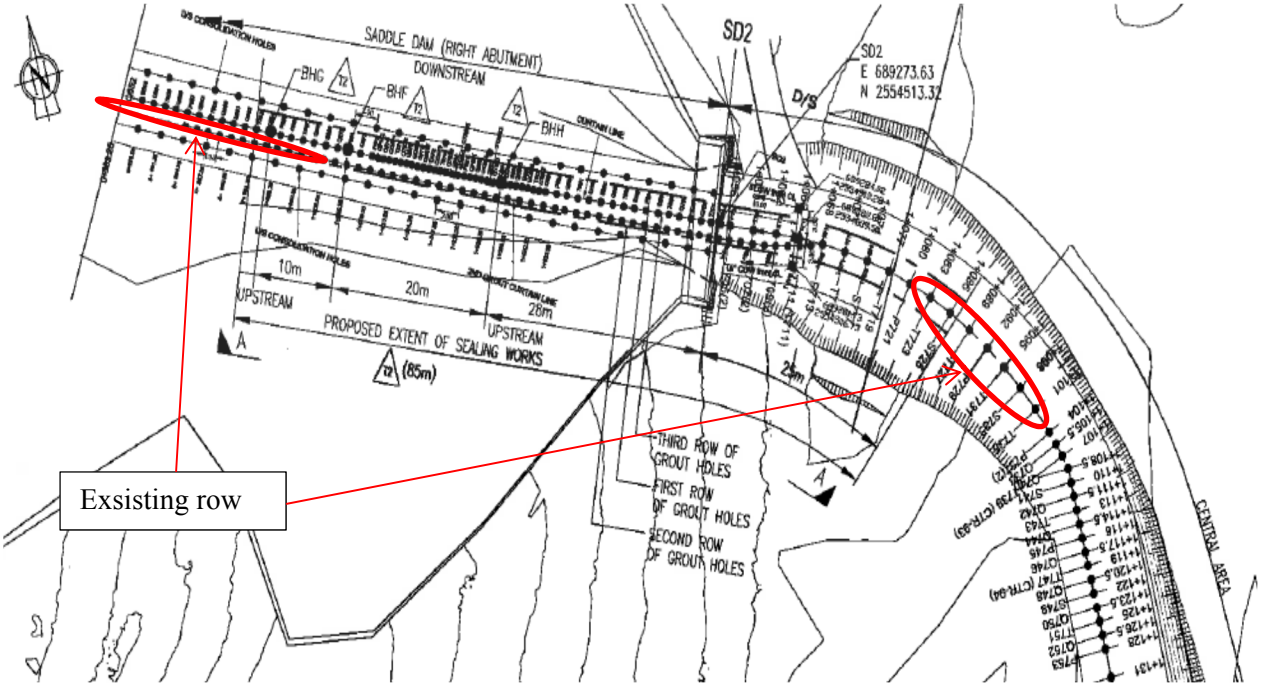


Figure 4: Plan View Grout Curtain holes Saddle Dam

© 2016 Deep Foundations Institute

The closure method of grouting was adopted wherever practical. In this method, successive half hole spacing is adopted (**Figure 4**), i.e. an additional borehole located mid-way between two previously grouted holes until the closure criteria is satisfied. The indicative closure criteria for this contract was a Lugeon Value of less than 2 after grouting, or a grout take of less than 25kg of cement per meter.

The grout curtain was carried out following standard method for cavity grouting and fissure grouting by the use of single or double packer grouting with stabilised cement- bentonite grout. It was agreed to follow the GIN method of grouting.

GIN numbers were selected initially after execution of trial grouting as follows:

GIN = 1500 for depth < 45m, GIN = 1800 for depth 45 to 55m and GIN = 2000 for depth > 55m.

Grouting pressures were selected as follows:

3 bar in 15-30m depths, 4 bar in 30-35m depths, 6 bar in 35-45m depths, 7 bar in 45-55m depths, 8 bar in 55m-65 depths, 10 bar in 65-75m depths, 12 bar in 75-90m depths.

In general grouting pressure should be as high as possible to increase the reach of the grout, but low enough to avoid hydro- fracturing. As above mentioned GIN numbers and pressures were found to low during execution of the works, GIN value was adjusted to GIN = 2500 and pressure in the upper stage (15-30m depth increased from 3 to 4 bar in December 2013. Where the grout curtain was installed immediately to the secant pile cut-off wall the drilling the grouting works was only executed after sufficient hardening of the pile cement toe.

GROUT MIXES AND TESTING

The grout mix composition for any part of the grouting operation consisted of cement, water and bentonite having a liquid limit of at least 400. In addition plasticizer was used to achieve a high viscosity. The grout mix composition was developed by trials were approved by the Engineer prior of commencement of the grouting works. Following grout mixes shown in Table 1 were proposed by the Zublin Ground Engineering.

Mix	W/C RATIO	WATER	BENTO-NITE	OPC CEMENT	PLASTICISER HaBe 28M	VISCOSITY	DENSITY	SETTLE-MENT	SETTING TIME
1	0,71	100 kg	1 kg	141 kg	1,7 ltr	35 - 40 sec	1,54 – 1,66 g/cm^3	2 -3% (2 hrs)	20 - 24 hours
2	0,68	105 kg	4 kg	150 kg	5,0 ltr	38 – 44 sec	1,62 – 1,68 g/cm^3	0 - 1% (2 hrs)	65 - 90 hours
3	0,73	100 kg	5 kg	140 kg	6,0 ltr	38 – 44 sec	1,55 – 1,62 g/cm^3	0 - 1% (2 hrs)	120 - 160 hours

Table 1: Grout Mix Proposal

The locally sourced cement was delivered to site in dry condition uninjured by exposure to the weather in pre-weighted 50 kg bags and directly loaded into the cement silos. Water for grouting was taken from the Dam Reservoir and was measured in a calibrated water tank to ensure the correct batch quantity. Bentonite was delivered in 1 ton Big Bags and Plasticiser in 1000ltr container. The Grout was mixed high speed colloidal mixers for a minimum of 5 minutes prior to being despatched into the agitating tank. The

© 2016 Deep Foundations Institute

temperature of the water was monitored at all time and was supposed to be in the range of 5° to 10°C immediately prior to the preparation of grout. The temperature of the water was maintained by cooling system where the water was circulated through air-cooled chillers. During the works grout testing was carried out on a daily basis. Following tests have been performed:

- Settling Test, measured as the ratio between the volume of water appearing above the grout and the total volume in a 1 litre, 6cm dia. test tube after 20, 40, 60, 80, 100 and 120 minutes and after setting.
- Setting Time Test (NFP 15-431)
- Marsh Cone Viscosity Test in accordance with American Petroleum Institute Procedures API RP 13B-2.1.2 to determinate the fluidity of the grout.
- Density Test in accordance with American Petroleum Institute Procedures API RP 13B-1 to determinate the specific gravity of the fresh grout mix
- Pressure Filtration Test.

DRILLING OF THE GROUT HOLES

As noted above the grouting works comprised three rows of grout holes. The downstream line was grouted first, followed by the upstream line with the central line being grouted, where required last.

Each row of the curtain consisted of primary holes drilled at 12m spacing, followed by secondary holes reducing the spacing to 6m, then tertiary at 3m, quaternary at 1,5m and where considered required, quintenary boreholes at 0,75m spacing. The first row of grout holes was located downstream 2m parallel from the center line of the Saddle Dam, the second row upstream also 2m from the center line of the Saddle Dam. The third row was drilled and grouted mid-way between 1st and 2nd line. For execution of the Primary and Secondary holes no drilling was be carried out within 30 m of a borehole being grouted or water tested until the pressure in that hole has reduced to zero. No drilling and grouting was allowed within 10m from a secant pile being drilled.

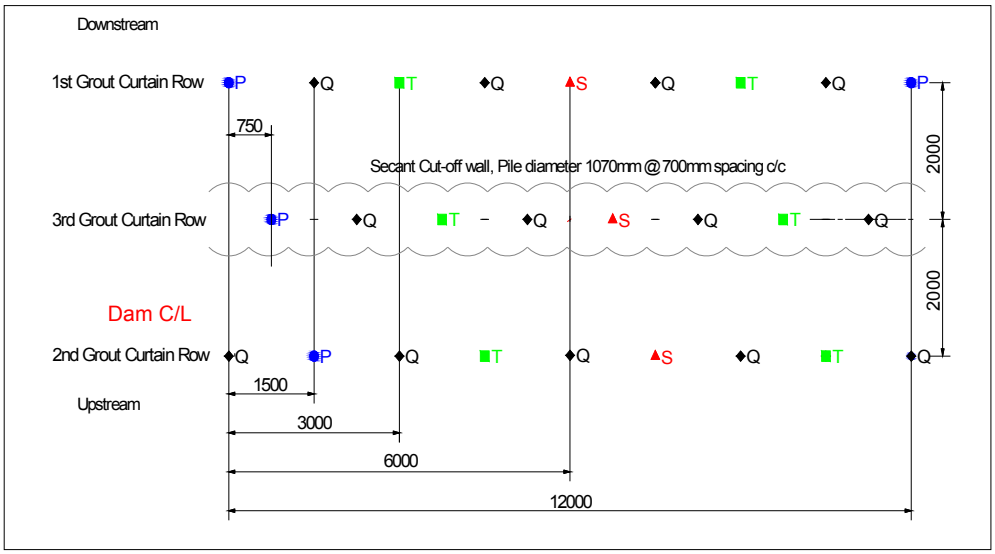


Figure 6: Split Spacing drilling sequence

© 2016 Deep Foundations Institute

Drilling through the dam core was performed using a rotary drilling method with a three-wing bit and casings. For drilling in the rock below the dam core a water flushed down the hole hammer system was utilised providing a hole diameter of 95 mm.

While drilling through the dam core the drilling fluid pressure was maintained sufficiently below the hydraulic fracturing pressure. A pressure gauge was installed at the rig for monitoring such that the injection pressure does not exceed 12 – 15H (KPa) where H is the depth of drilling. As the drilling in clay tends to put a lot of very fine particles into suspension in the slurry and adding weight to the drilling system. This can cause problems with cutting removal, over pressurization of the formation and excessive wear on tooling. If significant variations were observed, the drilling was stopped and the bore cleaned with water until the density has reached acceptable parameter before drilling continued.

Prior to water pressure testing or grouting works the holes were washed out with clean water to remove all debris, drill cuttings and dust which could clog the inner surface of the hole. The bore deviation was measured with vertical inclinometer.

GROUTING

For the grouting works a fully automated injection Module HPI with data recording, pump control and fully automatic mixing equipment was utilised. The software to record the injection parameters and control the injection is called IDE®/ IDR. It is a proprietary development by Zublin Ground Engineering for the record of the grouting data. The software is installed on a notebook and is connected with the grouting sensors applied on pumps and hoses. Getting the sensor data, the software records the actual grouting parameters and stores them in a file. Each injection results in a summary file which contains the location the point of time (datum, starting time, duration, end time) and the grouting data (ID of grout, injected volume, pressure) as well as the ID of the used pump.



Figure 7: Drill rig at the dam crest

© 2016 Deep Foundations Institute

On completion of drilling the hole down to its required depth or to the bottom of the stage and thoroughly cleaned by flushing to the bottom of the hole with clean water, the inflatable packer was set at the top of each stage to be grouted. The packer of 1 m length adjusted to the geological strata was used to seal off the section. When grouting any stage, care was taken to ensure that grout does not by-pass the packer. Once the grouting has started it had to be continued as specified until the refusal criteria according to GIN (Grouting Intensity Number) was obtained or when the grout take over a five minute period is not greater than 2,5 litres per meter length of the stage being injected. The application of the GIN method assured the grouting pressure does not cause displacement of the rock, hydro-fracturing nor excessive grout travel. A sample of the GIN trend curve is shown in **Figure 8**.

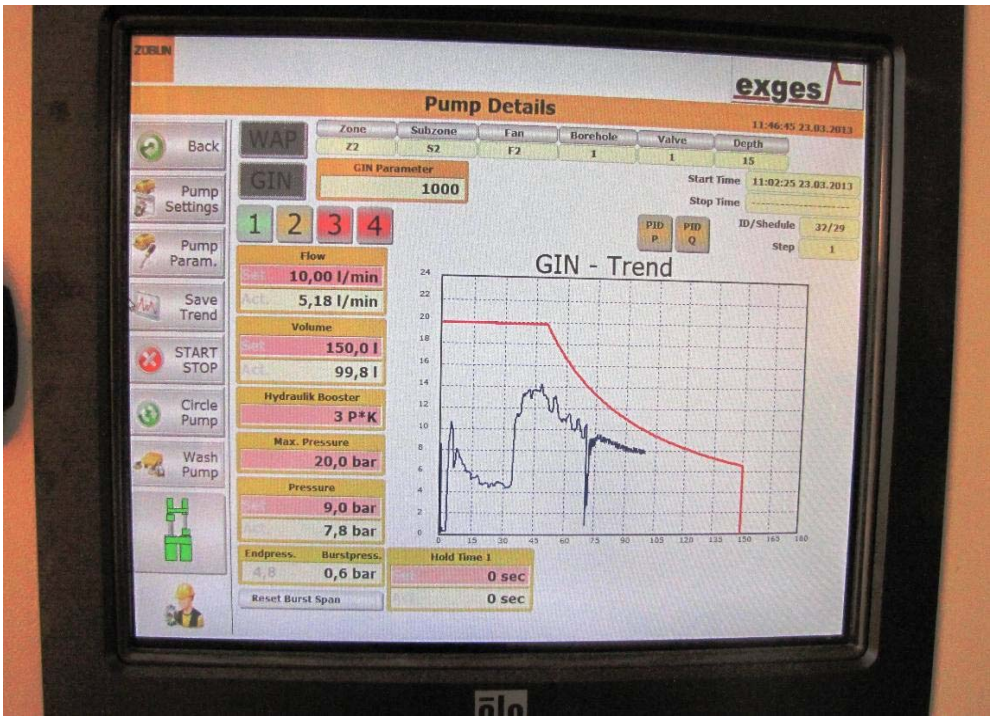


Figure 8: **Injection Data Record software – performing GIN grouting**

GROUTING RESULTS

The results of work at the downstream line of the Saddle Dam already confirmed the geology as being very challenging and variable. Even 0,75m difference in location in the grout wall could make a significant change. Where holes did show low grout takes (50 to 100 litres for 6 stages) another neighbouring hole only 1,5m away had a total take of 758 litres in just one stage, yet another hole located 0,75m from either above holes had grout takes in all but 2 stages ranging from 1220 to 4374 litres per stage. Due to earlier experience with such variable takes, quintenary holes were drilled and grouted where considered appropriate. The Sequence of drilling and grouting was worked out on a weekly base or if required on daily base depending on the progress.

© 2016 Deep Foundations Institute

The rock condition at the shoulder of the Saddle Dam was relatively good with small volumes of grout injected with few exceptions. For the Primary and Secondary holes located downstream row of the Saddle Dam Abutment a significant high grout take was observed immediately starting from the Interface of Dam Core to bedrock in depth up to 25 to 45m. Due to the challenging geology it was observed that stages could not be sealed properly and grout was bypassing through the high permeable rock strata causing collapse of the borehole above packer. The loss of packer was the result. To overcome this challenge, it was decided to grout the first 3 stages starting from the interface following the top-down method in 5 meter long stages. However, during further execution of grouting works it became evident that down stage grouting is also required for stages below 45 meter up to Tertiary and Quaternary holes. Same procedure was also applicable for the majority of holes located in the upstream row, only for grout holes executed in the center line the use of up-stage grouting was possible as the rock condition was stabilized by grout injected at downstream and upstream row.

The seepage flows in the chamber below the Saddle Dam have been recorded prior to and during the sealing works. Prior to the sealing works with the reservoir full a total Saddle Dam seepage of up to 25 litres/second was observed. In April 2014, following the near completion of the sealing works and with the reservoir full, the total Saddle Dam measure seepage was less than 2.0 litres/second.

A less visible result of sealing works is the lowering of the ground water table in the garded area below Saddle Dam; previously the water table was about 0,2m below ground and was in April 2014 around 2,0m below ground.

At the Main Dam the site investigation holes drilled on the Left Abutment did not identify any significant areas of high permeability. In order to try to find the egress points of visible leakages high on the Left Abutment, part of a retaining wall was removed. Based on the observation of the ground exposed by this excavation, 15 No holes from the top of the Main Dam were drilled and grouted and the effect of that grouting confirmed by angled hole. Grout from some of these emerged from seepage points in the area where the retaining wall had been removed. Further drilling and grouting was carried out from the drainage gallery within the dam, with holes generally extending 40 metres below the floor of the gallery. Lugeon tests were carried out during the drilling of selected holes before those were grouted. A few areas of higher permeability were identified and grouted.

The grouting at the Main Dam noticeably reduced the visible leakages from the downstream shoulder above the access to the gallery but had minimal effect on the seepage collected from the drain holes in the gallery.

© 2016 Deep Foundations Institute

ANALYSIS OF WATER LEAKAGE IN DOWNSTREAM DAM TOE OF KOMANI DAM

Përparim Alikaj, Geophysical expert at Dept. of Earth Sc., Polytechnic University of Tirana, Albania.
Email: alikajp88@gmail.com
Arjan Beqiraj, Hydrochemistry expert at Dept. of Earth Sc., Polytechnic University of Tirana, Albania.
Email: ae_beqiraj@yahoo.com
Arjan Jovani, Director of Dam Safety Department and member of ALBCOLD, Albania.
Email: arjanjovani@gmail.com
Altin Karriqi, Geophysical expert at Dept. of Earth Sc., Polytechnic University of Tirana, Albania.
Email: altin.karriqi@fgjm.edu.al

ABSTRACT

Koman dam is one of most important high dams in Drin River Cascade. Safety of these dam is very important not only in power production but also in social and environmental aspects connected with vital activity of local people living in downstream of Drini River and in economic development of the country. In December 2014, during the rehabilitation works on some protection measures at the downstream dam toe, a considerable amount of water leakage emerged from the dam toe. This was noticed even before, but its amount and physical-chemical properties were not evaluated. In order to study the origin and flow direction of this water leakage, KESH sh.a. (Albanian Power Corporation) realized Geophysical and Hydro chemical surveys, turbidity and flow measurements and respective analyses of them. Geophysical survey consist in Apparent Resistivity surveys and Self Potential survey. The survey was done in 3 lines using Resistivity "Real Section" technique. Results are plotted in Sections and plan maps of resistivity and SP parameter. There are defined possible leakage areas and the main flow directions are defined. Also, there is done a hydro chemical analyse. Nine water samples were taken from the different places as Komani lake, service galleries of dam, dam toe and two karst springs which were analysed for both physical-chemical parameters and isotopic composition of O-18 and D. The attained analytical data found that water from the dam toe leakages shows the same chemical signature and isotopic composition of O-18 and D as the water from the lake, thus confirming the hydraulic connection between them.

Also, There is done the analyse of water leakage turbidity and amount of this leakage

Key words: Koman dam, dam toe, water leakage, geophysical methods, Resistivity "Real Section", hydro chemical methods, water turbidity.

INTRODUCTION

Drini River is the longest river (285 km) in Albania, having a catchment area of 14,174 km^2. Five high dams have been constructed there creating the Drini River Cascade which is a unique cascade in Europe regarding the height, materials of dams, utilization and management of cascade, economic and human risks also hydrology and seismological risks.

Komani Dam is the second highest dam in Albania with a height of 115.5 m (Fig. 1). The length of dam crest is 275.0 m and the level of dam crest is 185.0 m.a.s.l. It is a concrete faced rock fill embankment dam that was constructed from 1980 to 1985. Its section is fairly symmetric. The surrounding area of the dam is composed of hard rocks, mostly represented by Triassic schists, Jurassic – Cretaceous limestone and Paleogene flysch.

© 2016 Deep Foundations Institute



Fig**.1. View of Komani Dam**

In December 2014, during the rehabilitation works on some protection measures at the downstream dam toe, a considerable amount of leakage water emerged from the dam toe (Fig. 2). This water leakage was noticed even before, but its amount and physical-chemical properties were not evaluated. In order to study the origin and flow direction of this water leakage, KESH sh.a (Albanian Power Corporation) realized some Geophysical and Hydochemical surveys, measurements and studies.



Fig**. 2. Water leakage in dam toe**

GEOPHYSICAL SURVEY

The geophysical survey consisted in using of Electrical Tomography method through known Apparent Resistivity "Real Section" technique as well as Self Potential method. Both these Electrical methods provide important information on location and flow direction of leakage waters in the area of downstream dam toe.

The methodology of Electrical Prospecting with "Real Section" technique is given in the paper of Langore, Alikaj & Gjovreku published in the journal of EAGE, Geophysical Prospecting in 1989. The "Real Section" is an original technique introduced by P.Alikaj in 1978, which provides a very good corroboration between the surveyed electrical parameters and the geological structure in a section. The "Real Section" is not a mathematical inversion, but through special algorithms it provides an accurate definition of electrical parameters observed at various depths in the form of so-called "Physical Inversion". This technique has been proved successful in Albania, Canada, USA, Latin America and in many other parts of the world, presenting very accurate results in mineral exploration, engineering geology studies, hydrogeological studies, environmental studies, etc. (Frashëri, Lubonja and Alikaj 1995, Roth 1997, Alikaj and Gordon 1999, Alikaj at al. 2012, Karriqi and Alikaj 2011, Karriqi and Alikaj 2012).

© 2016 Deep Foundations Institute

The analysis of Electrical surveys obtained with Apparent Resistivity "Real Section" technique and the Self Potential method are presented in the Figs. 3 - 8 below.

In Fig. 3 is given the topographic map of the area of downstream slope of the Koman dam with location of geophysical survey lines. Two lines were located in the lateral parts of the dam (near to east and west abutments), starting from the plunge pool to Komani Dam.

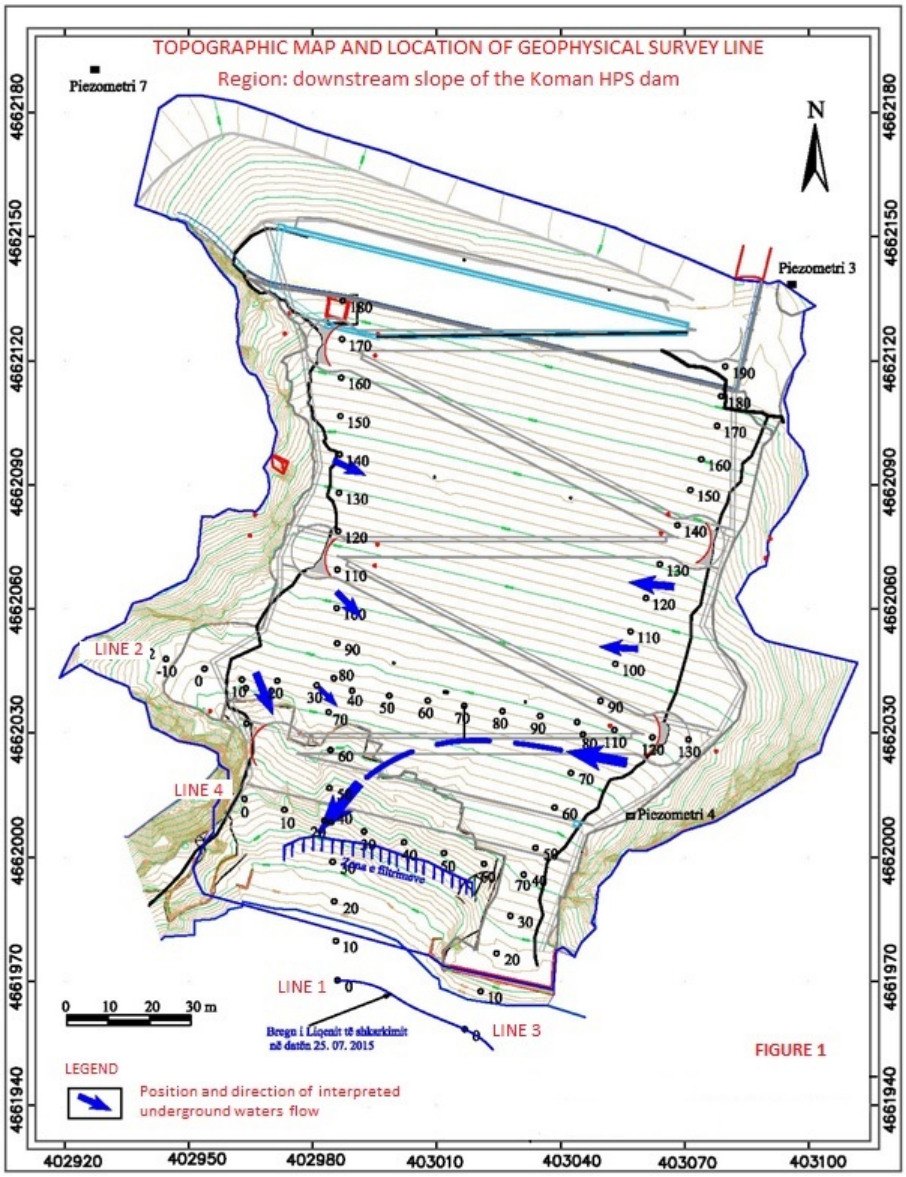


Fig.3. Topographic map and location of geophysical survey lines

In Fig.4 is presented the Physical Inversion of Apparent Resistivity (ρ_a) "Real Section" surveyed on Line 1. As it can be noticed, the lowest part of dam body (Stations 10 – 40) is characterized by low values of ρ_a, in the order of 90 – 200 Ohm.m. This is the part where the leakage waters come out under the dam (around St 33) and this represents the zone of lowest ρ_a in all surveyed lines. It can be connected not only with presence of underground waters but with an increased amount of clay material in the detritus rocky matrix in this sector.

© 2016 Deep Foundations Institute

Towards the dam center ρ_a parameter starts to gradually increase in the values 300 – 1,000 Ohm.m, up to stations 70 – 80 near the surface and up to station 90 at depth. This sector fits well with the level of underground water in the dam body, which is exposed at the maximum level of 80.5 m.

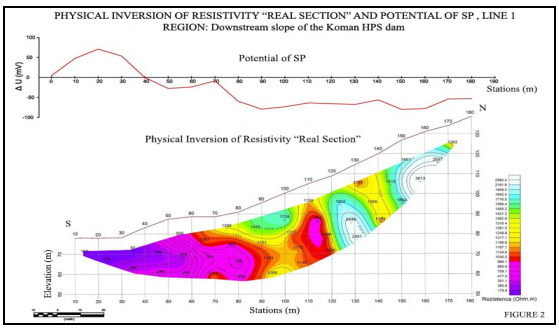


Fig.4. Physical Inversion of Apparent Resistivity (ρ_a) "Real Section" surveyed on Line 1

In sector between St 80 -105 is reflected an obvious increment in ρ_a up to 1,700 Ohm.m, near the surface, which is connected to the dry upper part of dam, consisted of carbonaceous rock pieces and slates of high resistivity.

On station 110 is noticed a decrease in ρ_a in the range of 860 – 1,000 Ohm.m, which shows for a weak karst zone within the limestones, located not more than 5 m from Line 1 in this sector. The water that enters the dam body is interpreted at depth 15 – 20 m. Based on Electrical survey is not possible to be defined its yield. Another less karst developed zone is observed in proximity to St 130, where the values of ρ_a are slightly decreased, compare to the previous ones of 2,491 Ohm.m. Further on, up to the end of the line the values of ρ_a remain high, up to 3,613 Ohm.m, which is related to very dry part of the dam body.

As regard to Self-Potential method surveyed in this line, it clearly indicates with positive values up to 72.6 mV the zone of leakage exit, between St 0 and 40. Based on our experience and on literature information (Minsley etc. 2011), the positive anomalies are characteristic for leakage outlet (discharge zone), while the negative ones are connected with recharge zone.

Previous similar studies (Minsley etc. 2011) notes that zones of leakage waters in dam are reflected with positive anomalies of Self Potential, with intensity 70 – 120 mV. In Line 1, the rest of the SP potential (Fig. 2) remains totally negative, up to the end of line (St 180). This can be explained with a nearly vertical downstream of the underground waters flowing in the contact between limestone rocks of the western shoulder (abutment) and dam body.

In Fig. 5 is represented the Physical Inversion of the Apparent Resistivity "Real Section" on Line 2, which runs orthogonal to Line 1 (west – east).

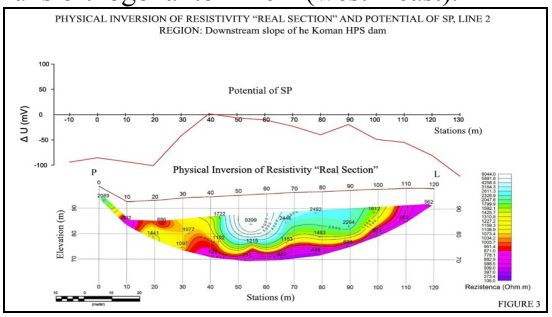


Fig.5. Physical Inversion of Apparent Resistivity (ρ_a) "Real Section" surveyed on Line 2

© 2016 Deep Foundations Institute

The Line runs nearly according to elevation contour at heights 93 – 98 m. In the sector between St 10 and 20 in western part, a decrease in ρd value (632 Ohm.m) is noticed, which is located near the surface, in proximity to limestone rock basement. Also, near the St 20 (7 m further north) is located a discharge tunnel, which brings a certain amount of water in the dam, so the surrounding environment is wet. It is noticed that the resistivity decrease is found in a broader zone than just the wet area near the tunnel. Around St 30 – 40, at level 80 m can be noticed as a resistivity decrease the merge of these waters with dam basement waters. On St 55, near the surface is noticed an extremely high value of ρd (9,399 Ohm.m), which can be explained with supper dry conditions of the near surface detritus rock matrix. At elevation 80 m and below, ρd is drastically decreased, down to value 711 Ohm.m. This can be clearly explained with maximum water level in the dam, which is observed in this line as well.

Through these surveys it is not possible to be accurately defined any deviation less than 3 m from the groundwater level (80 m) in this line. The sector between the St 100 and 120 reflects again low values of resistivity parameter, which can be explained with water leakage near the contact between limestone rocks of eastern shoulder and dam.

As regard the profile of the SP potential on Line 2, it is characterized by an increase in central stations with positive value up to 3.5 mV, while on both edges the values drop respectively to – 94 mV to west and –116 mV to east. These values support the idea of vertically down flow of leakage waters in contact between dam body and basement rocks of the shoulders. With geophysical methods cannot be defined if the water has a karst or lake origin.

In Fig. 6 is presented the Physical Inversion of Apparent Resistivity "Real Section" of Line 3, which runs almost parallel to Line 1 (north – south). The values of this parameter in this line have similarities with Line 1, because both are located in similar positions to each other, but in opposite flanks of the dam.

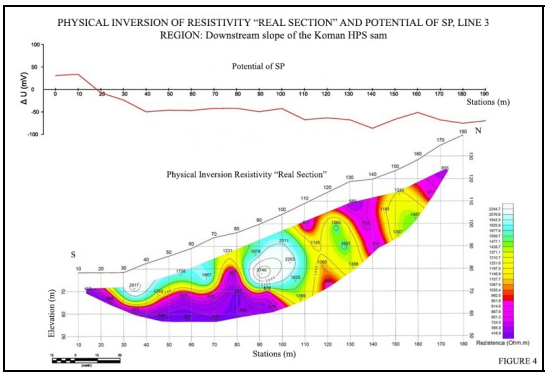


Fig.6. Physical Inversion of Apparent Resistivity (ρ_a) "Real Section" surveyed on Line 3

Here again, the bottom part of the dam is characterized by low values of ρd, in the range 250 – 580 Ohm.m. Such values are characteristic for dam body or very altered basement rocks saturated with underground water below the level 80 m. The sector near the surface, between St 25 and 70, is characterized by high ρd values (1,700 – 2,800 Ohm.m) caused by proximate limestone rocks of shoulder which are fairly compact.

It is of interest to point out a zone of relatively low resistivity between St 70 and 80, which can be interpreted as a zone of intensive fissures and porosity within limestone. This zone is likely to serve as a filtering route of leakage waters that enter the dam body by eastern shoulder. Further on, between St 80 and 100 the rocks become very compact again, due to high resistivity values in the range 2,000 – 3,700 Ohm.m. Respectively, on St 110 and 130 are noticed drops in resistivity values at 580 - 890 Ohm.m, which are interpreted as caused by near surface waters coming out of a grout tunnel in eastern shoulder of the dam, above the St 190. Such waters are not in considerable amount, at least on the surface.

© 2016 Deep Foundations Institute

As regard the measurements of SP on Line 3 their profile shape is similar to the one on Line 1. The zone of leakage water exit around St 0 – 20 is reflected with positive values up to 33 mV. The rest of Line is characterized by negative values, which vary in the range -26 to -86 mV. Such values are likely to be related with waters that come out from grout gallery and leak in the contact between shoulder and dam body.

In Fig. 7 is presented a map of Apparent Resistivity parameter for the Gradient array with dimensions AB = 140 m and MN = 10 m, compiled by survey data on three Lines. Even though the Lines 1 and 3 are far apart (70 m), combined with data of Line 2 they provide interesting information in this map (inclined according to dam plan), reflecting the water level of the dam down to depth 30 m from inclined surface. In the SE corner of the map (Line 3, St 35) is observed an increase of ρd due to proximity of the compact rock of dam shoulder. In this map, the sectors of pink colour in dam flanks indicate water leakage zones in dam body, which were explained above. In centre of dam, according to Line 2, is noticed a zone of dark pink colour, which reflects the deepest part (30 m) studied with Electrical Resistivity method in this Line.

This zone is connected with the broad zone obtained on Line 1 St 35 – 55, while on Line 3 it has a connection with the karst zone noticed near the station 75. This can be considered as a "preferential" axis of underground water in the dam which should be carefully considered in the future. It can be possibly caused by some increase in the porosity of dam fill material, as a result of a continuous water flow according to this direction. To better study this possible direction, a piezometer is recommended to be located on Line 2, St 75, as well as another one on Line 3 St 75.

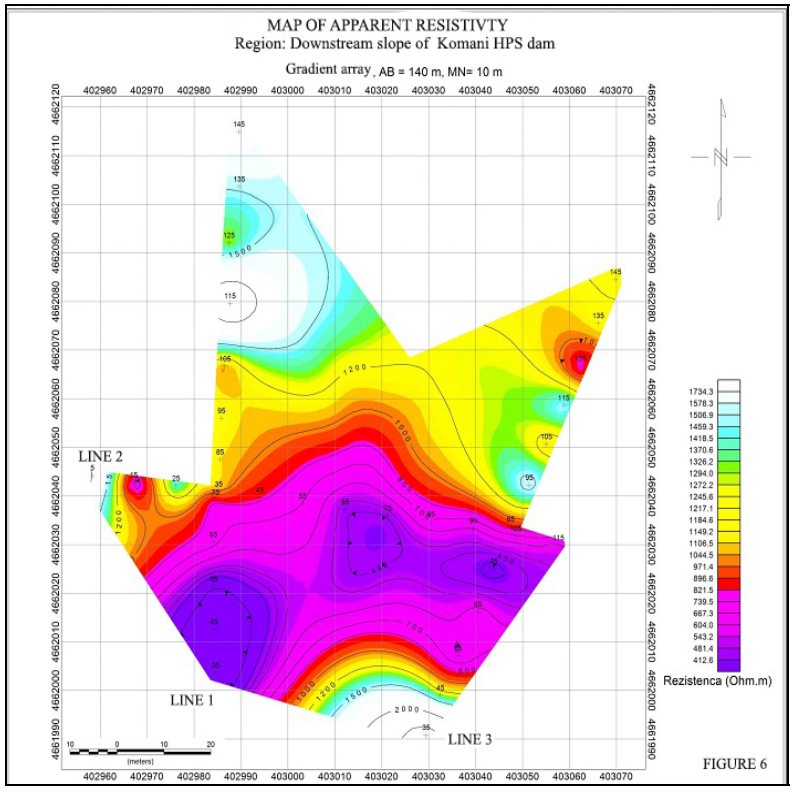


Fig.7. Map of Apparent Resistivity

To study the possible hydrostatic difference in north – south direction of the dam centre, another piezometer is recommended to be located about 50 – 60 m north of Line 2 St 75. Based on anomalous values of resistivity (pink) of Line 2 St 15 and Line 1 St 105, which were analyzed above, two other routes of leakage water intakes in dam body can be interpreted on the western part.

© 2016 Deep Foundations Institute

In Fig. 8 is presented the SP plan map obtained by measurements in four surveyed lines. This map clearly depicts the zone of leakage exit (near Line 4) through a broad positive anomaly with intensity up to 72 mV on Line 1 St 20. The width of anomaly in this sector indicates that the leakage water under the dam is not a local phenomenon.

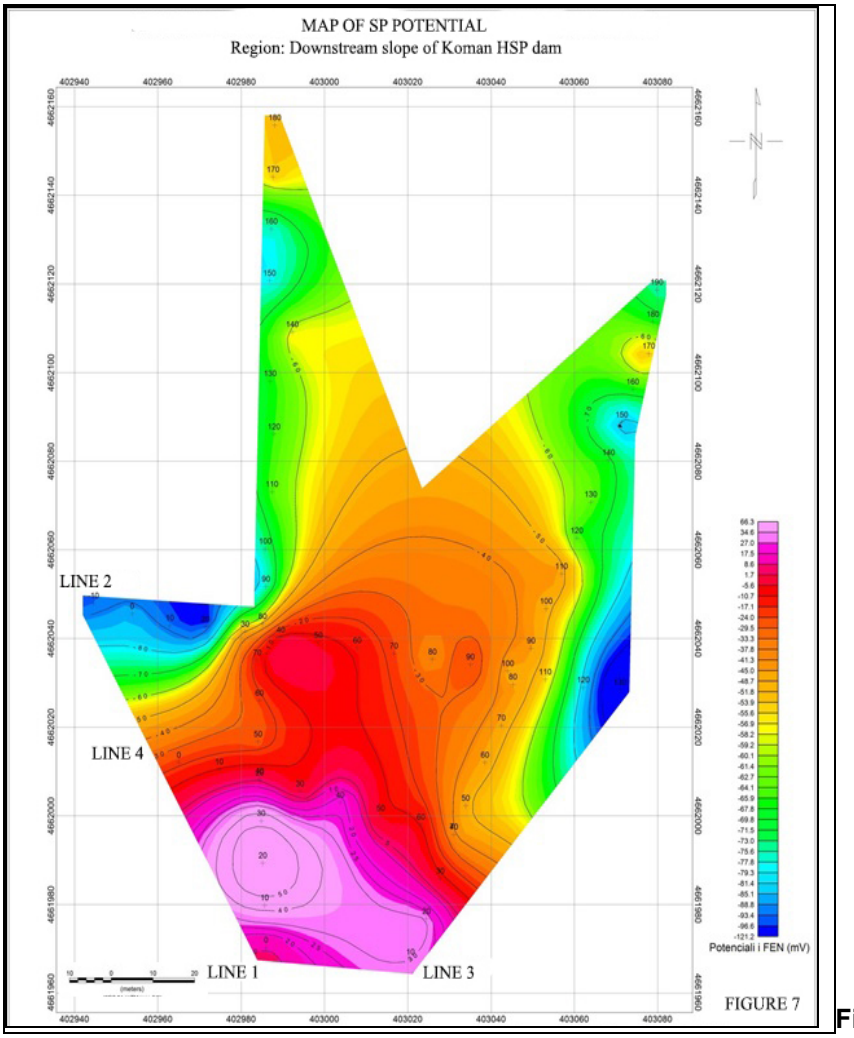


Fig.8. Map of SP potential

Based on all surveyed lines, in Fig. 1 are indicated with blue arrows the positions and directions of leakage waters, which are interpreted based on results of Apparent Resistivity "Real Sections" as discussed above. The bigger arrows indicate bigger interpreted amount of water flow.

© 2016 Deep Foundations Institute

HYDROCHEMICAL AND ISOTOPIC STUDY

In the frame of this study, nine water samples, for both hydro chemical and isotopic analysis, were taken from the following sites: sample no. K-1 and K-1/1 from dam toe water leakage, two samples from lake water at depth 30m and 50m, K-2 from turbine water, K-3 from a karst spring near to the dam, K-4 and K-5 from grout gallery at level 76m asl, K-6 and K-7 from the lake and K-8 from Bena karst spring located about 3 km north of dam. (Fig. 9)

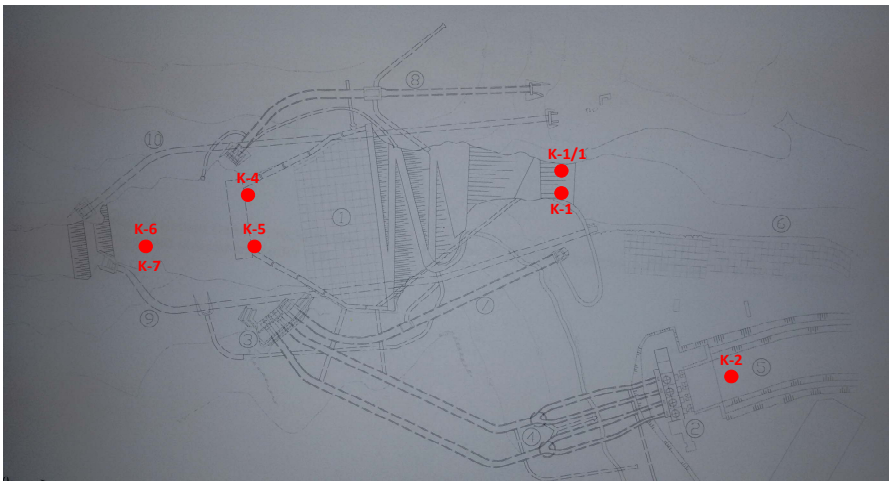


Fig. 9. Location of water samples in the area of Komani HPS

The above water samples were analyzed in the Laboratory of Water Chemical Analyses of Albanian Geological Survey for the following parameters: K, Na, Ca, Mg, NH4, HCO3, CO3, Cl, SO4, NO3, NO2, Total Dissolved Solids, Dry Residue, Hardness (°G), pH, Electric Conductivity (µS/cm). The results of analyses were processed in Aquachem software and Excel.

Based on chemical composition of water, two very different hydro chemical types have been clearly outlined: Ca-Mg-HCO3 hydro-chemical type, where the water from lake, dam toe leakage, and grout gallery are included and the Ca-HCO3 hydro-chemical type belonging to Bena karst spring.

According to their behaviour, the chemical parameters are divided in inert and reactive tracers and they may be used effectively to derive physical parameters such as recharge rates and mixing (Herczeg and Edmunds, 2000). Among the inert tracer's chloride is the most useful because it is highly mobile and is not involved in the common geochemical reactions that occur in groundwater. In addition, chloride is a conservative element with mobility in water very similar to the water molecules (Hill, 1984).

All chemical parameters and in particular chlorides (typical conservative parameter) indicate that the leakage waters in the downstream toe have the same chemical composition or very close to the lake water (Fig. 10), thus supporting the opinion that the downstream toe leakage water comes from the lake.

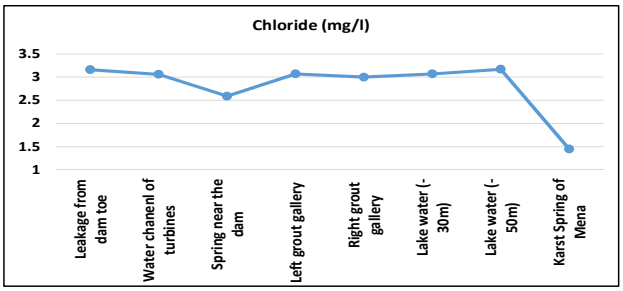


Fig. 8. Variation of chloride content in the waters

© 2016 Deep Foundations Institute

The vertical thermal stratification of lake water provides an extrapolated temperature of water on the lake bottom of 9°C, which is the same as the temperature measured at the water leakage of downstream dam toe (Fig. 11).

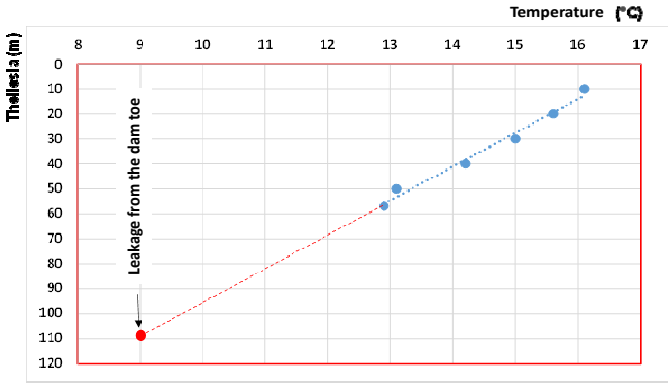


Fig. 9. Variation of water temperature

The isotopic analyses of O-18 and H-2 of the same water samples were also performed. Stable isotopes have been among the most used techniques for solving problems of age, origin especially of pathways of the water movement in a watershed (Michener and Lajtha, 2007).

Stable isotopes of water (hydrogen - ^{2}H or D for Deuterium) and oxygen (^{18}O) have been used since the pioneering work of (Craig,1961). Deuterium and oxygen-18), as an integral part of the water molecule, offer a unique opportunity for evaluation of leakages from dams and lakes.

In fact, isotopic composition of D and O-18 strictly confirmed the results taken from the hydro-chemical data. The water from lake, dam toe leakage and grout gallery show the same isotopic composition (Fig. 10) that, in turn, is isotopically lighter (more negative δ values) than karst water of Bena spring (K-8). Sample K-3 from the spring near the dam shows isotopic composition that falls between that of karst water and lake water (Fig. 10).

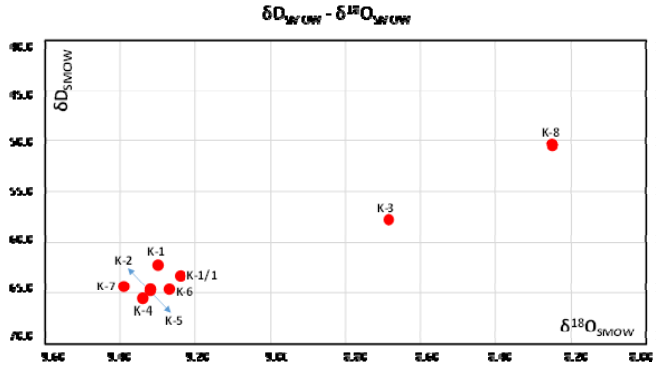


Fig. 10. Variation of $\delta^{2}D$ versus $\delta^{18}O$

© 2016 Deep Foundations Institute

MEASUREMENT OF WATER LEAKAGE AMOUNT AND TURBIDITY AND LEVELS

Every week, the water flow rate in leakage zone of downstream toe was measured and monitored. The amount of water leakage in the dam toe consist in 700 – 1000 l/sec. (Fig. 11)

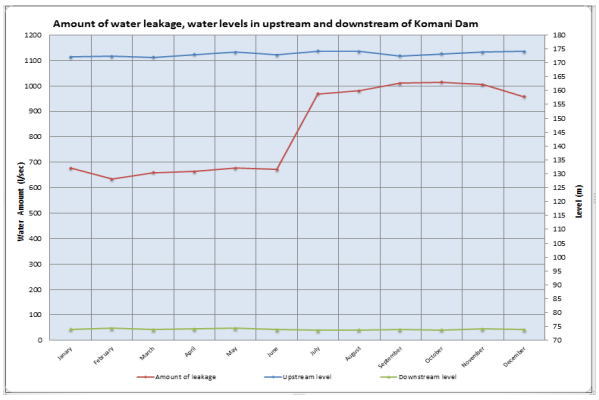


Fig. 11 – Amount of water leakage and water levels

Also, the water turbidity in leakage zone of downstream toe continues to be monitored. The values of the water turbidity in dam toe are very small. They are not depending from turbidity of surface lake water.

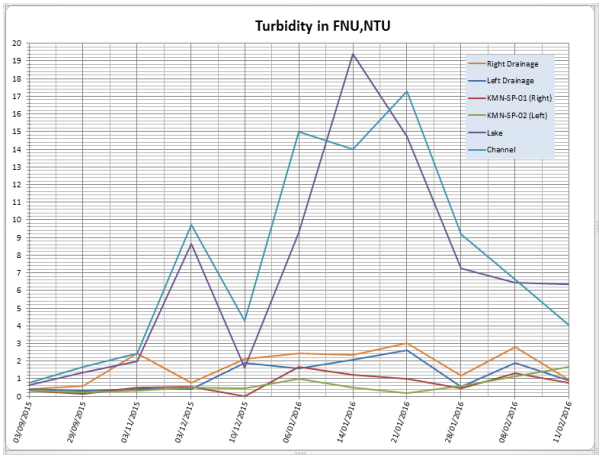


Fig. 12. Turbidity in FNU of water samples

In order to study the hydraulic gradient of water flow through the dam, at February 2016, KESH realized the installation of 3 piezometers along the downstream slope of dam and the measurements of water levels inside of them. The results and underground water levels are given in the Fig. 13.

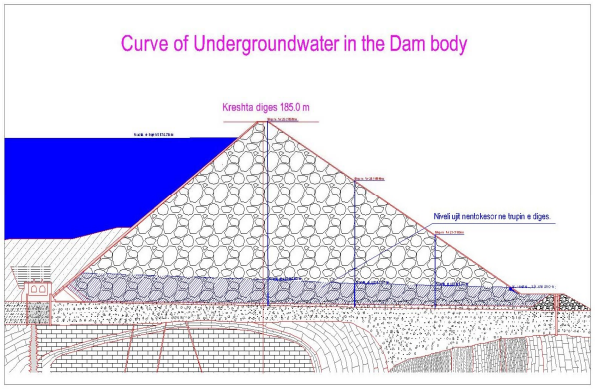


Fig. 13. Underground water inside Komani Dam

© 2016 Deep Foundations Institute

CONCLUSIONS AND RECOMMENDATIONS IN THE FUTURE

The Electrical surveys with Electrical Tomography according to Apparent Resistivity "Real Section" technique and Self Potential method have contributed in determination of location and direction of leakage water in the downstream slope of Koman Dam.

The Apparent Resistivity survey data indicate that leakage waters get in the dam body from both its shoulders (abutments) in various sectors. In Fig.1 are presented with blue arrows the locations of leakage water intakes in dam body as well as their approximate directions. It is noticed a preferential axis NE – SW, which has to be considered by KESH. With geophysical methods used in this study it is not possible to be evaluated the amount of water that enters the dam and its origin.

The data provided by Self Potential method indicate a broad positive anomaly in leakage water exit area, which fits with literature information on this purpose. The negative anomalies in both dam shoulders indicate the presence of leakage waters in the contact between abutments and the dam body.

Further geophysical studies are recommended in the future in Koman dam, especially in the reinforced concrete face, in order to study the possible penetration of the lake water through gaps between concrete plates. For this purpose, the Self Potential survey alongside the dam waterline is recommended.

Based on chemical composition of analysed waters, two hydro chemical types have been clearly outlined: hydro-chemical type Ca-Mg-HCO3, where is included the lake water and the leakage of downstream toe and the hydro-chemical type Ca-HCO3, where the karst spring types are included.

All chemical parameters and in particular the chlorides (typical conservative parameter) indicate that the leakage waters in the downstream toe have the same chemical composition or very close to the lake water, showing that leakage water of the dam toe comes from the lake. This fact was strongly supported by the isotopic composition of O-18 and D, as well.

The thermal stratification of lake water at depth provides an extrapolated temperature of 9°C, which is the same as the temperature measured at the water exit leakage of downstream toe. This supports the opinion that the leakage water emerges from the bottom of lake.

The monitoring of water leakage in dam toe should continue to check the quantity, temperature and turbidity of water leakage.



© 2016 Deep Foundations Institute

REFERENCES

- Alikaj P., Likaj N., Karriqi A., Collaku E. 2012 - Advancement in IP/Resistivity "Real Section" presentation. Bulletin of Geological Sciences, No.1, Tirana, Albania.
- Bublaku S, Beqiraj A.2015- Water Balance and Water Loss Implications in Badovc Lake, Kosovo". Journal of Geography and Earth Sciences.
- Craig H., 1961. Standards for reporting concentrations of deuterium and oxygen-18 in natural waters - Science, 133, 1833-1834.
- Karriqi A. and Alikaj P. 2012 - Electrical prospecting survey – an effective geophysical tool in seacoast environmental studies: two cases from Albanian western lowland", MarCoastEcos 2012, Tirana 25 -28 April 2012, Albania. Pg. 67, ISBN 978-9928-137-14-2.
- Karriqi A. and Alikaj P. 2011 – Combination of Resistivity "Real Section" with quantitative interpretation of Vertical Electrical Soundings. GeoAlb 2011 "Mineral Resources and their perspective", Mitrovice, Kosovo, 2011. Proceedings p.466 – 469, ISBN-978-9951-612-00-5.
- Herczeg A.L. and Edmunds W. M., 2000. Inorganic ions as tracers. In Environmental tracers in subsurface hydrology. Eds. Peter G. Cook and Andrew L. Herczeg. Springer Science + Business Media New York. 43-89.
- Hill D. 1984 - Diffusion coefficients of nitrate, chloride, sulphate and water in cracked and uncracked chalk. J. Soil. Sci. 35, 27-33.
- Minsley, B.J., B.L. Burton, S. Ikard & M.H. Powers 2011 - Hydro geophysical investigations at Hidden Dam, Raymond, California, Journal of Environmental and Engineering Geophysics, 16, 145-164.
- Leibundgut Ch, Maloszewski P, Külls C. 2009 - Tracers in Hydrology, Wiley-Blackwell, West Sussex, UK,415 p.
- Michener R and Lajtha K., 2007. Stable isotopes in ecology and environmental sciences. Blackwell Publishing LTD, USA. 566 pp.
- Iván A. Contreras, Samuel H. Hernández 2003 - Techniques for prevention and detection of leakage in dams and reservoirs.
- Plata, B.A. and Araguás, A.L.2002 – Detection and Prevention of Leaks from Dams. A.A. Balkema Publishers, p. 419.
- Alikaj P. and Gordon R. 1999 – A geophysical tool for Mexican geologic environments.

© 2016 Deep Foundations Institute

CANNONSVILLE DAM EMERGENCY BOREHOLE RESPONSE

Paul C. Schmall, P.E., Moretrench, Rockaway, New Jersey, USA, (973) 627-2100, pschmall@moretrench.com

John Vickers, P.E., NYC Department of Environmental Protection, Western Operations Division, New York USA, (607) 363-7000, JVickers@dep.nyc.gov

William H. Hover, P.E., GZA GeoEnvironmental Inc., Norwood, Massachusetts, USA, (781) 278-3816, william.hover@gza.com

Keywords: dam breach; artesian condition; pressure relief; deep wells; borehole closure grouting

ABSTRACT

Any sudden subsurface issue that threatens the stability of existing buildings or infrastructure is cause for immediate investigation and prompt remedial action. Nowhere is this more true than the nation's dams and levees. A breach of any kind, no matter how small initially, has the potential to be catastrophic and impact a wide downstream area. Addressing the problem quickly and effectively is therefore critical. When a turbid water discharge developed from a rock drainage embankment at the toe of the earthen dam impounding Cannonsville Reservoir, near Deposit, NY, the source of the problem was quickly identified as resulting from an artesian water release from an underlying permeable silty sand and gravel formation during drilling of three exploratory boreholes in preparation for construction of a hydroelectric plant at the downstream toe area of the dam facility. Work was halted, intense 24-hour monitoring was implemented, and as a risk reduction measure NYCDEP began to draw down the reservoir's total storage at a rate of a billion gallons per day. Input sought from contractors within the geotechnical community helped result in the design and development of a deep well system to control the artesian pressure. Once this primary objective was accomplished, modified compaction grouting was performed at the borehole locations to densify loosened soils, fill any voids created by the discharge activity, and seal off the permeable aquifer. This paper discusses the dewatering program, specialized drilling techniques required, monitoring and verification of pressures within the dam, and the grouting program.

BACKGROUND

Cannonsville Dam, owned and operated by the New York City Department of Environmental Protection (NYCDEP), is located in Deposit, New York on the West Branch of the Delaware River and impounds the Cannonsville Reservoir. Built between 1959 and 1964, the dam has an impervious central and upstream core zone keyed into the glacial till foundation, a semi-pervious downstream embankment zone, and a rockfill toe and is approximately 2,800 ft (853.4 m) long and 175 ft (53.3 m) high. The dam is formed by two embankment sections. The left section forms the main embankment blocking the main river channel, while the right section is constructed over a topographic saddle or low ridge. Reservoir level is actively influenced by withdrawals through the West Delaware Tunnel, based on water supply need. Under normal circumstances, water is continuously discharged downstream to the Delaware River.

In early July, 2015, three soil borings, designated FID-0, FID-1 and FID-8, were drilled at and near the toe of the dam under the auspices of NYCDEP as part of the final design for construction of a hydroelectric plant. The borings were advanced through the rockfill toe of the embankment using hollow stem augers, and drilled to depths of up to 62 ft (18.9 m) below ground surface by mud rotary methods. Drilling had been initiated in the powerhouse area on July 1, and resumed following the Independence Day 3-day holiday break. At this time, drillers noticed a turbid/muddy discharge in the tailwater.

© 2016 Deep Foundations Institute

Piezometer data indicated that two piezometers along the right side upper and intermediate downstream benches of this dam section and one near the downstream toe bench had shown a pronounced decrease in head of up to 13 to 14 ft (4.0 to 4.3 m). This was the first strong indicator of a change in conditions developing in the embankment. Slight to moderate reactions were also noted in other piezometers.

NYCDEP personnel immediately recognized the limited potential of eventual development of a piping condition, ceased any further drilling work, and as a risk reduction measure immediately began to draw down the reservoir's pool level at a rate of a billion gallons per day. Intense 24-hour monitoring was implemented, including increased piezometer reading from daily to hourly, around-the-clock physical monitoring of the turbidity, monitoring of discharge grain size, and settlement monitoring. Emergency Action Plan (EAP) orientation was also conducted for emergency management personnel in downstream communities. NYCDEP then retained a consulting engineering firm experienced with subsurface conditions at the dam to further evaluate the situation and work with NYCDEP, the specialty contractor and regulatory agencies to develop a mitigation plan.

GROUND CONDITIONS

Cannonsville Dam was built in a region plentiful in dense, impervious glacial till deposits and lake sediment deposits. According to the NYSGS Bedrock Geology map, these soils overlie the gray and green cross-bedded sandstones with alternating layers of red and green shales and rounded pebble quartz conglomerates of the West Falls Group (upper Walton Formation, formerly known as the Catskill Series).

The site chosen for the construction of the dam consisted of a narrow valley filled with glacial deposits overlying bedrock at depths up to 100 feet and beyond in some areas. According to the original contract drawings for Cannonsville Dam, overburden on the eastern valley walls was shallow to nonexistent, and overburden depths typically increased significantly on the western walls.

The impervious embankment was constructed with silt and clay fill. The semi-pervious downstream zone was constructed with medium to fine sand content as well. The foundation soils typically consisted of very dense glacial till deposits or hard to stiff silts and clays over dense glacial till. The glacial till deposits consisted typically of clayey silt with little sand and occasional cobbles and boulders. The bedrock beneath the concrete spillway and foundation soils underlying the earth embankment section consists of sandstone with occasional thin shale seams.

ROOT CAUSE OF THE PROBLEM

The drilling of the exploratory holes created a hydraulic connection between a deeper permeable stratum under artesian pressure and the overlying silt-clay confining later and stratified sands, silts gravels and clays immediately beneath permeable rockfill. During the three days that the borings were left open, the deeper artesian pressure induced groundwater to flow within the boreholes from the deeper artesian charged stratum up into the rockfill and in doing so erode the silt-clays and overlying natural fine sands and silts between the artesian zone and the rockfill (Fig. 1). The new seepage dissipated through the rockfill and ultimately through the embankment toe at or above El. 1000 ft (305 m). Creation of this shortened seepage path would be expected to locally increase seepage gradient and transport fine sediments. The drilling and borehole completion records indicated that the boreholes were not backfilled so as to create a seal between the pervious aquifer and the rockfill.

© 2016 Deep Foundations Institute

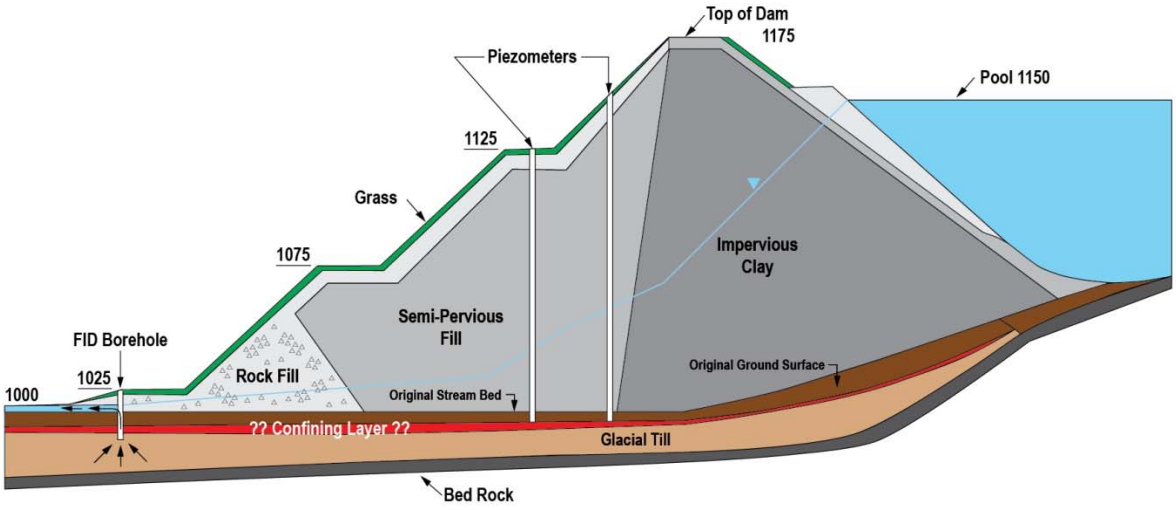


Figure 1. Dam profile illustrating the cause of the hydraulic connection.

The potential for dam instability resulting from the erosion of fine materials from the dam foundation was of primary concern for the long term, if erosion of the dam foundation were to occur. An on-site meeting was held with the engineering consultant, members of the NYCDEP response team, and several specialty geotechnical contractors to discuss potential remediation measures in order to provide a basis for rational selection of a highly qualified contractor with the resources to respond rapidly to the situation. Alternatives proposed included: large diameter over-drilling of the boreholes to physically cut off the boreholes from the seepage path; slurry (or "high mobility") grouting to seal seepage paths; and low mobility compaction grouting to fill voids and densify loosened granular soils. Relief wells were considered to be an effective intermediate step to control the artesian pressure prior to low mobility grouting of the three boreholes.

Ultimately, a relief well system to lower the hydrostatic head upstream of the three boreholes, within the pervious silt-sand-gravel artesian layer above the glacial till and below the confining silt-clay layer, followed by a grouting program to permanently seal off the artesian aquifer at the three boreholes, was the preferred approach. The specialty geotechnical contractor that accomplished the work was selected based on its rapid response capability and ability to not only develop and implement the pressure relief system but also to accomplish the grouting program, in a high quality and timely manner.

PRESSURE RELIEF DEEP WELL SYSTEM

Mobilization to the site by the specialty geotechnical contractor and initiation of well installation was accomplished within three days of notice to proceed.

The intent of the relief wells was to provide temporary pressure relief within the foundation area in the upstream vicinity of the three boreholes in advance of active measures to close the boreholes. The relief wells were designed to be installed in a drainage curtain configuration upstream of the exploratory borehole locations, as well as along the right abutment. The relief wells, combined with reduction of the reservoir level, were intended to reduce pressures and upward flow during the borehole closure process. These were intended as temporary relief wells which would include pumps to increase the amount of pressure relief, but consideration could be given later to convert the wells for long-term use in the event they are needed for future construction of the hydroelectric project and/or general pressure relief.

© 2016 Deep Foundations Institute

The initial planned configuration of the drainage curtain included up to 10 relief wells generally aligned along the upstream edge of the access road (Fig. 2). The well locations took into consideration the piezometers that indicated a pronounced pressure relief upon the turbid release. Relief Well No. 1 (RW-1) was established immediately across the road from FID-1, and was anticipated to intersect the high permeability aquifer based on review of boring logs, with subsequent wells spaced approximately at 40-ft (12.3 m) center to center along the road towards the low-level discharge chamber. During the installation process, wells 8, 9 and 10 were eliminated due to proximity to existing outlet works infrastructure. Ultimately, eight holes were drilled and seven relief wells installed.

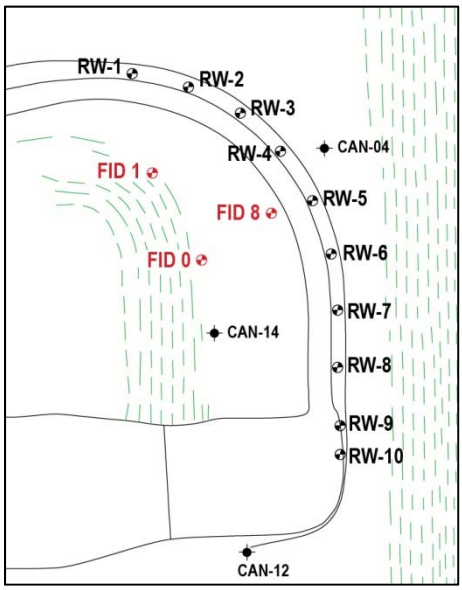


Figure 2. Plan view showing FID boring locations and initial relief well drainage curtain. During the installation process, RW-8, 9 and 10 were eliminated.

Well Installation

Sonic drilling was selected as the drilling method due to its ability to install a relatively large diameter casing through rockfill quickly and with little or no water or air flushing which could exacerbate the situation. Sonic drill rig availability and immediate availability of drill casing for permanent installation limited the size of boreholes (and thus screened diameters) that were possible. Ten-inch (254 mm) sonic drill casing accommodated an 8-inch (203-mm) steel casing grouted into the confining soil strata below the rockfill prior to advancing the boring into soils where elevated groundwater pressures might be present. Two rigs with different configurations were utilized so two different size boreholes were advanced beneath the 8-inch (203 mm) steel casing and completed wells with 3-inch (76.2) and 4-inch (101.6) diameter finished casings were designed and installed. A careful evaluation of the ground conditions was essential to provide a casing through the rockfill with adequate key-in and seal into the underlying confining layer, but not so deep as to encounter the artesian stratum. At the time of the drilling of the first relief well, the extent of the artesian zone was unknown.

The wells were constructed with slotted PVC screen and solid PVC riser. The slotted section was backfilled with filter sand on top of which a bentonite plug was installed. The remainder of the annular space was backfilled with cement bentonite. The wellscreens were set below the silt-clay confining layer so as to relieve the apparent artesian stratum (Fig. 3). The relief wells were designed to accommodate an electric submersible pump with a PVC discharge pipe. Each discharge pipe was designed to be fitted with a sampling port and flow meter and then connected to a common collection header. The collection header

© 2016 Deep Foundations Institute

discharged into a sedimentation tank which discharged into an existing catch basin that drained back to the river downstream of the turbid discharge location.

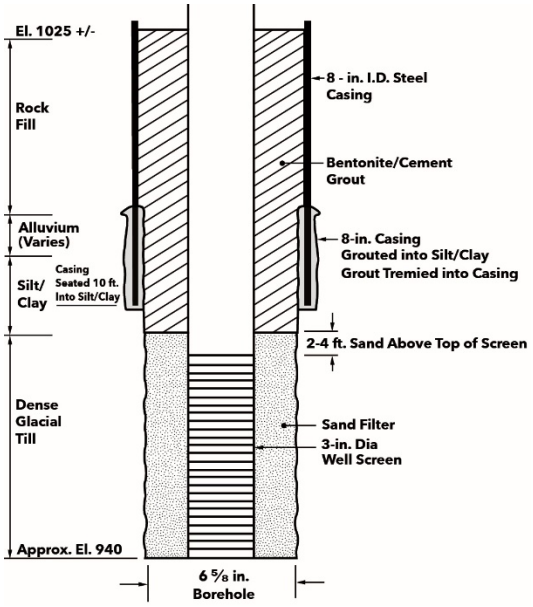


Figure 3. Relief well detail.

Results

Sonic drilling for relief well installation allowed continuous core sample retrieval. Soil cores were logged by the consultant and the geology was mapped. This allowed custom construction of screened zones for each relief well.

RW-1 was screened through a layer of very clean sands and gravels, clearly seen from the core samples, that was consistent with a feature believed to be associated with an ancient buried river bed. This feature was interpreted to be encountered deep within the glacial till foundation under the western core and downstream shell of the dam during a 2001 exploration program, and an old artesian water supply well located downstream of the dam. RW-2 and RW-3 were also screened in the same stratum. RW-4 through RW-7 did not encounter the coarser artesian stratum and were screened in the till or till-like material just above bedrock.

When RW-1, RW-2 and RW-3 came on line, with typical pumping rates of approximately 50 gpm (190 l/min), 25 gpm (94.6 l/min) and 30 gpm (113.5 l/min) respectively, the downstream toe was immediately depressurized and turbidity was no longer observed at the toe of the rockfill or in the river channel. RW-4 and RW-7, which had been pumping at approximate rates of 4 gpm (15.1 l/min) and 12 gpm (45.4 l/min), were deactivated once RW-1, RW-2 and RW-3 were in service. With all of the pumping from the deep artesian stratum, it was clear that it was the source of the erosion within the dam. The remaining two installed wells, RW-5 and RW-6, were therefore not used for production pumping. Further confirmation of the source of the erosive flow was provided when pumping was briefly stopped during electrical re-wiring operations and turbid seepage re-emerged. A historic photo of the construction of the dam (Fig. 4) indicated the old river channel that corresponded to the locations of the stratified sands, silts, gravels and clays east of the silt-clay confining layer and underlying pervious artesian zone, and the location of the higher yielding wells and the piezometers that were highly responsive to pumping.

© 2016 Deep Foundations Institute

As a result of the success of the relief well pumping, authorization for resumption of normal reservoir operations was given to the Department of Environmental Protection (DEP) by the Federal Energy Regulatory Commission (FERC).

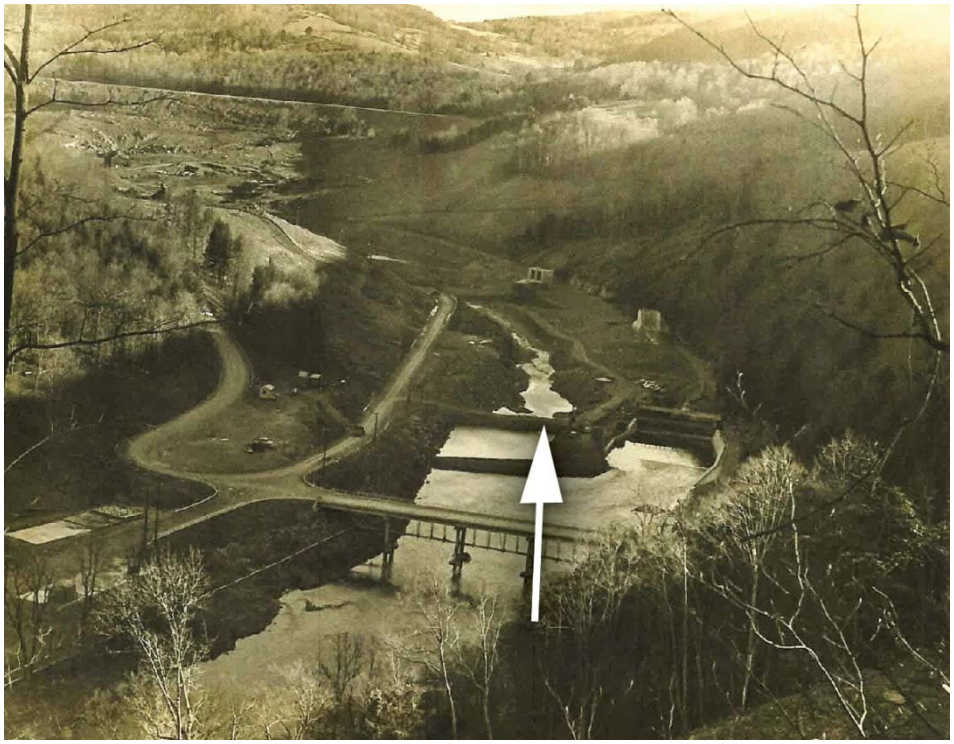


Figure 4. Historic photo indicating the presence of an old river channel.

BOREHOLE GROUTING PROGRAM

The objective of the grouting program was to restore pre-breach hydraulic conditions by sealing the leaking boreholes through the punctured confining layer, and replacing lost or eroded ground washed away with the moving groundwater. A significant challenge lay in that the precise location and verticality of the improperly grouted boreholes was unknown, therefore finding the boreholes was unlikely and the grouting would be conservatively required to modify the ground within a target area rather than at the precise borehole location. The determination of the success or failure of the grouting program would require external indicators such as piezometric levels and tailwater turbidity.

Low mobility grouting (LMG) was selected as the primary borehole closure methodology. LMG is a ground improvement method in which a low slump mortar-like grout is injected into the ground under pressure through an open-ended steel casing to displace and densify the surrounding soil. The grout does not permeate the soil but rather forms a bulbous mass. Strategically placed grout bulb "columns" within the target zone displace and densify the surrounding soil. The technique inherently results in ground displacement and movements.

Provision was made for an additional phase of permeation or jet grouting should the primary LMG grouting prove insufficient to achieve complete borehole closure. However, given the primary LMG grouting results, this was not required.

© 2016 Deep Foundations Institute

The Phase 1 LMG grouting plan included a first attempt to re-trace each of the FID exploratory holes, plus three holes surrounding each FID borehole location on a triangular pattern. The re-traced holes were intended to clean out and backfill the FID boreholes and the surrounding three holes were intended to squeeze the FID boreholes closed if they could not be found and re-traced.

The grouting plan included an alternate hole sequencing so that there was stagnant time at each grout location for dissipation of locally elevated pore-pressures. The LMG casing was advanced by internal flush duplex drilling from the ground surface to the target depth. Once the target depth was reached, the casing was retracted in 2-ft (0.6-m) lifts as the grout was injected. For each lift, injection continued until pressure refusal, a pre-determined target grout volume was reached, or surface heave was observed.

Although anticipated to be unlikely, if the boreholes could be re-traced, cleaned out, and grouted, this would be the most reliable closure that could be expected. Special measures were taken to give this approach the greatest success. Relatively flexible, specially selected 2.5 inch (63 mm) drill casing with a lost point tri-cone 3.5-inch (90-mm) diameter bit was advanced first with direct water positive flush so as to follow the existing FID borehole, as closely as possible. The approached worked very well at each of the three FID borehole locations. Minimal resistance to the advancement of the drill string and an abrupt increase in apparent density at the termination depth of each of the exploratory holes was a fairly conclusive indicator that the boreholes were re-traced full depth.

At each of the three boreholes, the lost-point bit was dropped at the bottom of the borehole and the hole flushed and grouted with a stiff mortar-like material. In general, the high pressure grouting began 3 to 5 feet below the bottom of an individual FID borehole. This assisted in attempting to densify any soils which might have been lifted or loosened at the bottom of the FID boring as a result of upflow. Higher grout pressures, up to 500 psi (34.5 Bar), were utilized beneath the confining stratum to replace eroded ground from the cleaner soils and compact the loosened soil zone. Lower pressure grouting was performed with an LMG grout of greater slump [4 to 6 inches (100 to 150 mm)] in the “erodible zone” of soils comprised of the stratified sands, silts, gravels, and clays below the rockfill and the foundation interface at the bottom of the rockfill. This erodible zone basically was the confining stratum. Pressures were reduced to minimize the potential for damaging the integrity of the confining layer due to the high forces that can accompany LMG grouting.

Following what was perceived to be successful sealing of the FID boreholes, grouting was performed at the three surrounding points on a triangular pattern around each borehole. This was elected to be done based on the volumes of grout which were realized and the concern about remaining voids or loosened zones beneath the confining layer. Figure 5 shows the grout volumes injected vs depth at each of the FID boreholes. The highest grout volume was observed at FID-1, which was always suspected by the project team to be the most likely problem based on the responses of the piezometers upon drilling.

Results

Observations during drilling of the FID boreholes provided the team with great confidence that the boreholes had indeed been retraced, despite earlier apprehension that this might not be possible. Observations during grouting indicated that the ground had been tightened up. After a reasonable grout cure time, the relief wells were deactivated one at a time. Immediate response in the piezometers (Fig. 6) suggested that pre-breach conditions had been restored (Fig. 7). It was generally agreed that the grouting had been as successful as could have been anticipated, but long-term verification of recharge of the piezometer levels as the pool returned to historic levels has been the conclusive factor. The wells have since been winterized for monitoring and possible dewatering for future construction of the powerhouse.

© 2016 Deep Foundations Institute

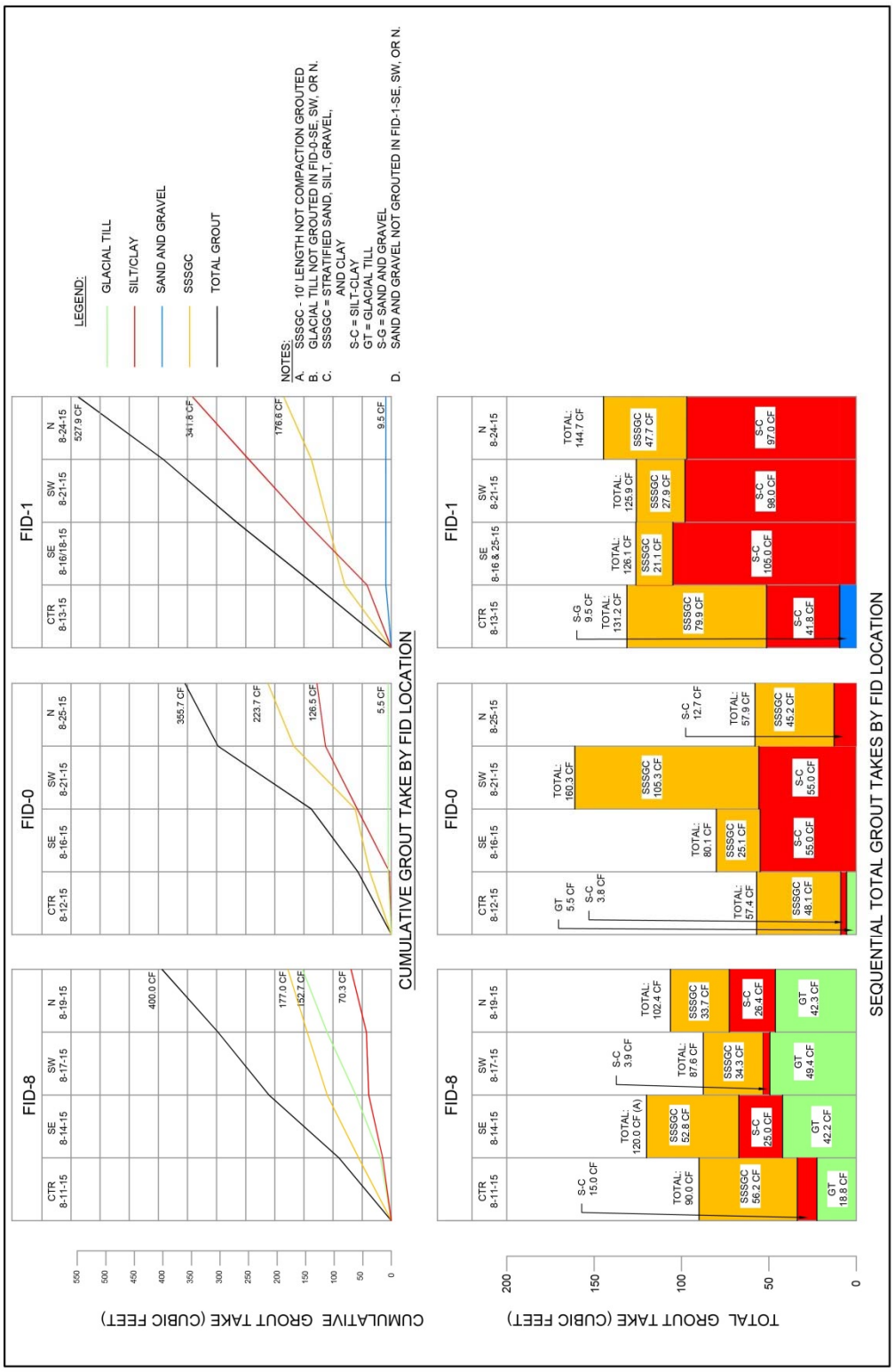


Figure 5. Cumulative grout take by FID location.

© 2016 Deep Foundations Institute

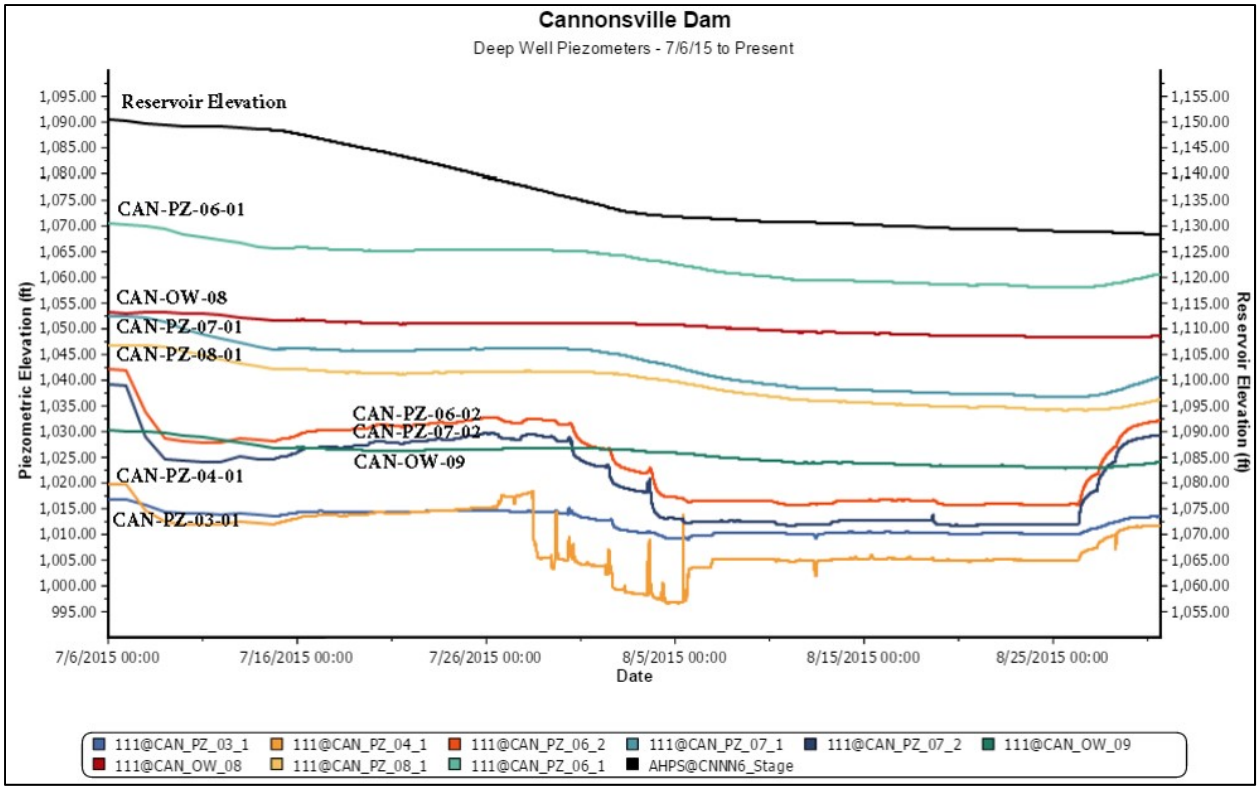


Figure 6. The piezometric responses observed upon breaching the confining layer, activation of the pressure relief wells, and sealing of the confining layer with compaction grout.

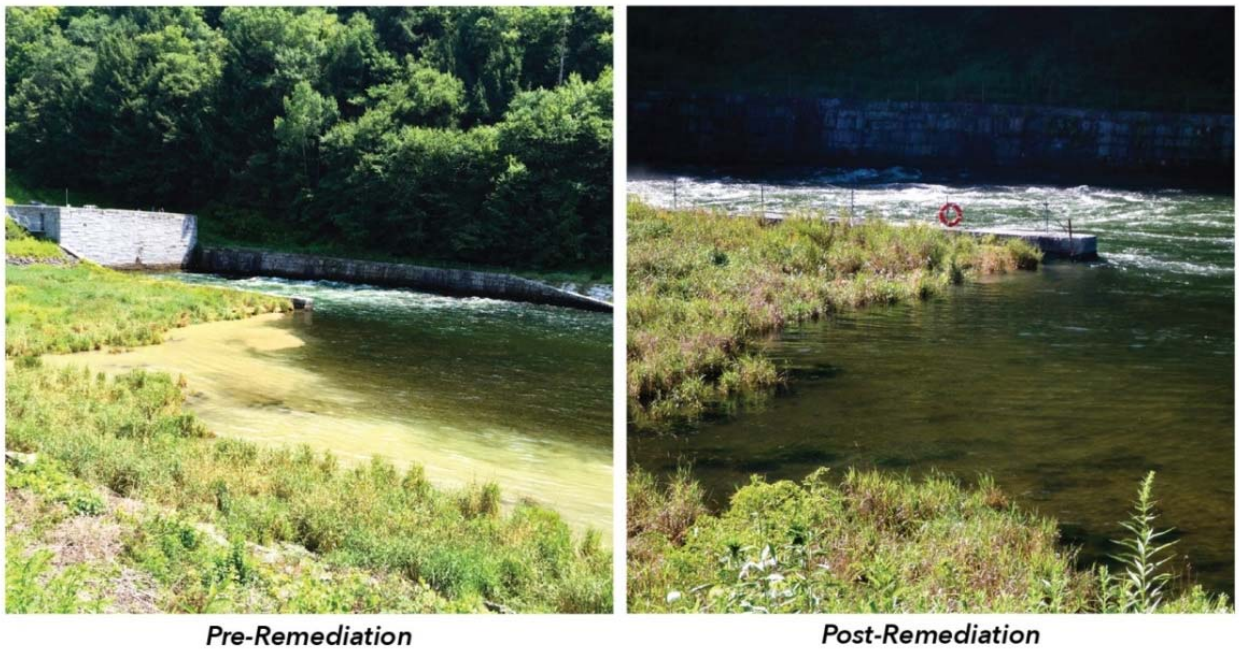


Figure 7. Photographs taken before and after the remediation program clearly show that the dam had been successfully restored to pre-breach conditions.

© 2016 Deep Foundations Institute

CONCLUSIONS

Key to the expedient resolution of the turbid discharge was the ability to respond rapidly in developing and implementing the pressure relief and grouting programs with close cooperation between project team members throughout the process. The work was an excellent example of the observational process. Appropriate design and placement of the relief wells minimized the number needed and expedited their effectiveness. The success of the primary grouting program eliminated any need for supplemental grouting. Effectiveness of borehole closure to date continues to be verified by observation of water levels at various positions within the dam.

© 2016 Deep Foundations Institute

GOWANUS PUMPING STATION; JET GROUT BOTTOM SEAL AND DEEP SUPPORT COLUMNS

Paul J. Lewis, P.E., Vice President, Gannett Fleming, Inc., Camp Hill, Pennsylvania, USA;
phone: 717-763-7211 email: plewis@gfnet.com
Paul C. Schmall, PhD, P.E. Vice President, Chief Engineer, Moretrench, Rockaway, New Jersey, USA;
phone: 973-627-2100 email: pschmall@moretrench.com
John M. Stanbury, Grouting Field Operations Manager, Moretrench, Rockaway, New Jersey, USA;
phone: 973-627-2100 email: jstanbury@moretrench.com
Kevin Clarke, Portfolio Manager, BEDC NYC Environmental Protection, New York, New York, USA;
email: kcl@dep.nyc.gov

The $200M rehabilitation of the more than 100-year-old Gowanus Flushing Tunnel and pumping station located on the Gowanus Canal in Brooklyn, NY required deep excavations in an extremely congested urban environment. The environment posed significant challenges, including: support of excavation for protection of existing structures, excavation through man-made fill while eliminating groundwater inflow due to potentially contaminated groundwater chemistry; discharge and disposal impacts; and potential settlement of surrounding structures. This paper will present the design and construction challenges associated with the construction of a new combined sewer overflow (CSO) chamberwhich included a jet grout bottom seal, to mitigate groundwater inflow during excavation. The foundation for the finished structure also included a unique system incorporating the jet grout bottom seal elements with deep jet grout columns (piles) acting as the foundation system for the final structure. Design and construction challenges included highly variable soil conditions such as man-made fill, obstructions from previous construction, presence of an organic stratum, and conflicts with existing timber foundation piles. The jet grout bottom seal was constructed in loose sands with the deeper jet grout foundation elements extending to bear on a dense sand stratum. Challenges associated with the bottom seal included grid spacing and construction sequencing. Wet in wet sequencing was used to mitigate potential windows in the bottom seal further complicated by shadowing from existing timber foundation piles.

Keywords: jet grouting, ground modification, seepage control, foundation support

INTRODUCTION AND BACKGROUND

Modernization of the Gowanus Tunnel Pumping System required deep excavations in the extremely congested urban environment of Brooklyn, NY. The project location is shown on **Fig. 1 and 2**. The focus of this paper is to present the design and construction challenges associated with the support of excavation, control of groundwater, and ultimate foundation support for the combined sewer overflow (CSO) bypass chamber extension. The project site was tightly constrained by adjacent, active roadways and underground utilities, as well as residential and industrial structures. Therefore, excavation support was required to limit ground movement and maintain the integrity of the surrounding facilities. Excavation depths to as much as 16 ft below ground level, 8 ft below groundwater, required that seepage and groundwater be controlled so as to minimize pumping and disposal of groundwater and maintain the surrounding groundwater levels to avoid potential settlement of adjacent roadways, utilities, and structures. In addition, the subsurface conditions required that the proposed CSO bypass chamber extension be supported on a deep foundation to avoid differential and total settlements that may be detrimental to the existing and expanded structure. A plan view of the existing CSO bypass chamber and the proposed extension is shown on **Fig. 3**.

Based upon a review of as-built record drawings, the CSO bypass chamber extension was originally planned to be constructed upon an existing reinforced concrete slab. The 2 ft thick reinforced concrete slab supported

© 2016 Deep Foundations Institute



Fig. 1 - Location

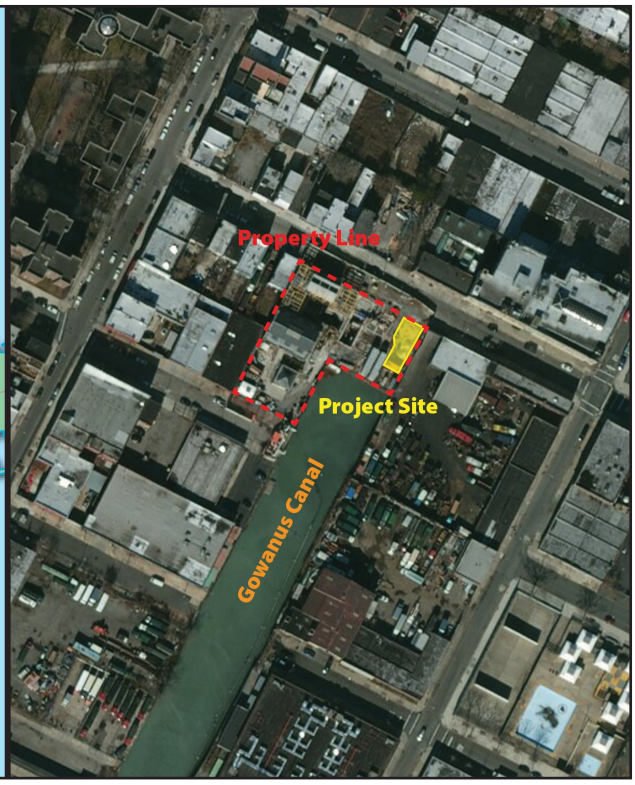


Fig. 2 - Aerial Location

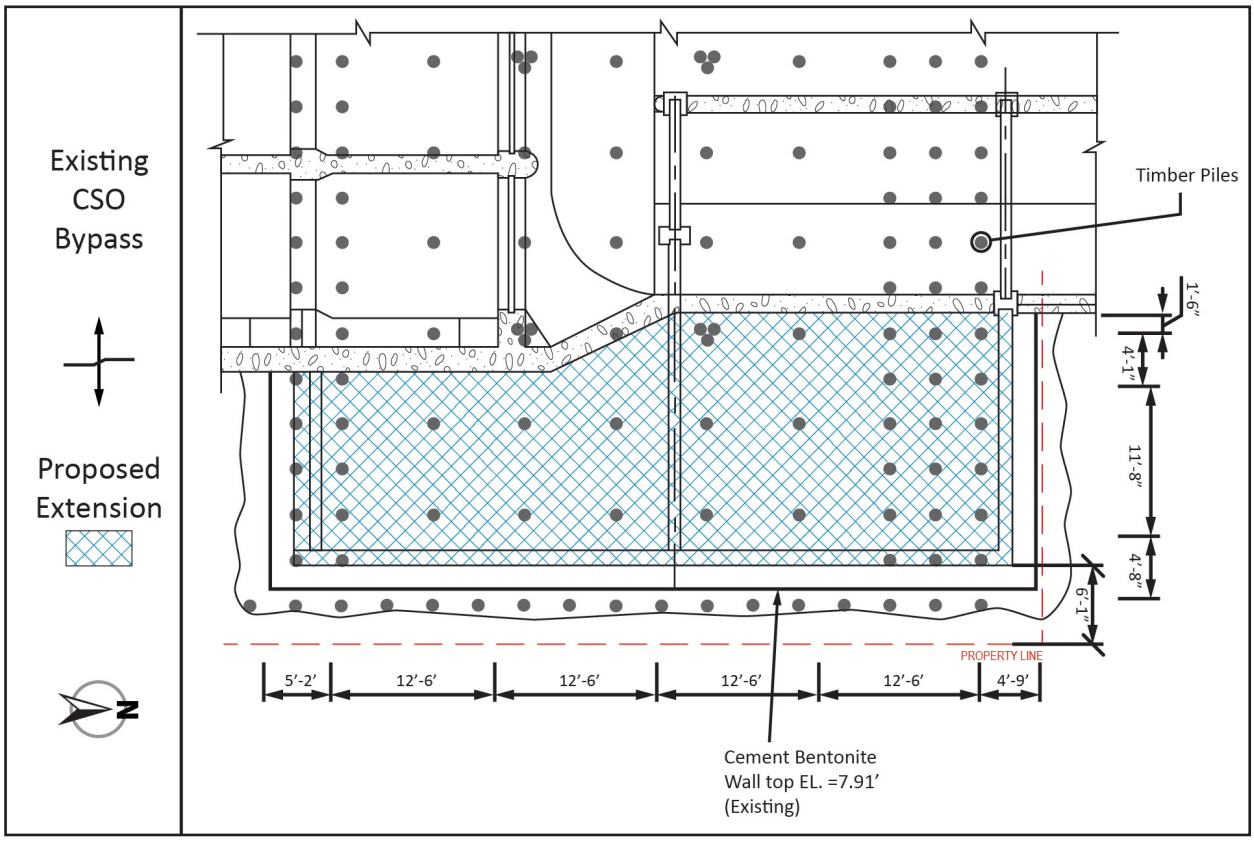


Fig. 3 - Plan View of Existing and Proposed Structure

© 2016 Deep Foundations Institute

on timber piles was the remaining portion of a previously demolished structure and was planned to be re-used as the foundation support system for the proposed CSO bypass chamber extension. **Figure 4** shows the planned re-use of the existing structure foundation as support for the proposed CSO bypass chamber extension.

A support of excavation wall consisting of a reinforced cement bentonite wall with soldier beams pinned to the existing concrete slab was designed and installed by the on-site construction contractor Northeast Remsco and their subcontractor, Nicholson Construction Company, with design assistance from Brierly Associates. A photo of the excavation support system under construction is included as **Fig. 5**.

Unfortunately, upon coring to embed the toe pins, the condition of the concrete slab was discovered to be extremely poor, and in some places non-existent. Subsequent further investigations of the condition of the concrete foundation slab revealed that the system could not be relied upon to limit seepage and groundwater inflow or provide foundation support for the proposed CSO bypass chamber extension and the need for an alternative solution became urgent.

SUBSURFACE CONDITIONS

Test borings on the site indicated that the subsurface conditions underlying the site consisted of approximately 20 ft of predominately granular fill with an occasional intermittent layer of cohesive soil. Beneath the fill is loose sand with N-values less than 10, which varies in USCS classification from SM to SP, extending to a depth of 45 ft. At this depth the sand becomes dense with N-values above 40. Groundwater was encountered within the fill stratum at approximately elevation 0 ft. The typical soil profile underlying the CSO bypass chamber extension is shown in **Fig. 6**.

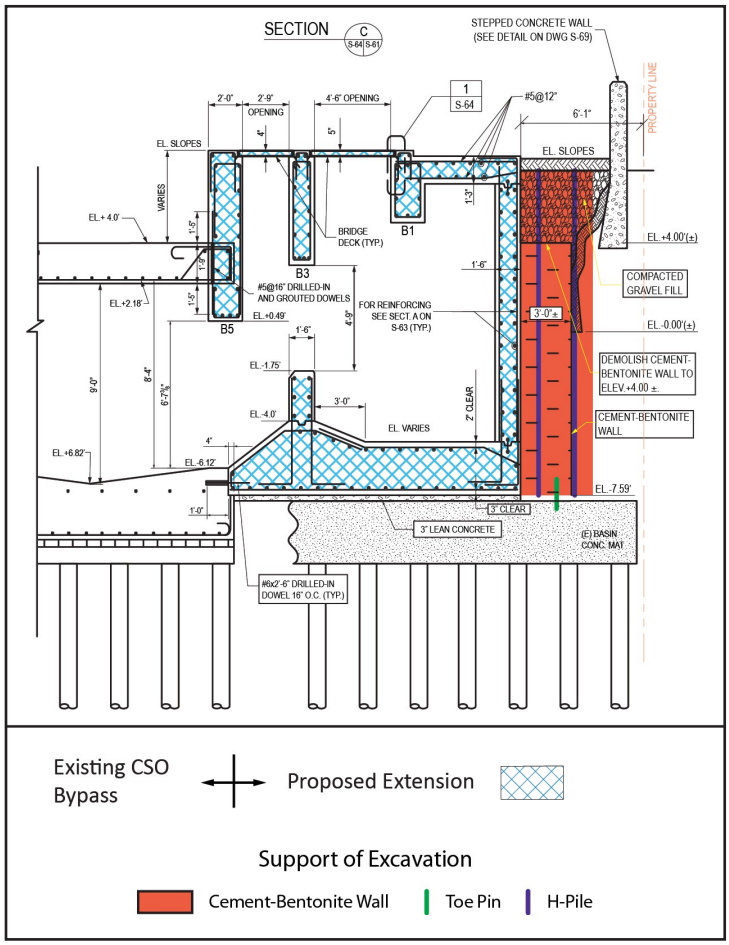


Fig. 4 - Section View of Existing and Proposed Structure



Fig. 5 - Photo of Cement-Bentonite Wall

© 2016 Deep Foundations Institute

ALTERNATIVE SOLUTION

Based on the described subsurface conditions, Gannett Fleming selected and designed a jet grout bottom seal and foundation support system as the most appropriate solution to minimize seepage and groundwater inflow/pumping during construction as well as to provide a permanent foundation support system for the finished structure.

In addition to the nearly ideal soil type for a jet grouting ground improvement method, the jet grouting process also provides a means for incorporating the already constructed cement-bentonite (CB) excavation support wall into the system by jetting a perimeter seal that would conform to the base of the CB wall as well as sealing in and around the grid of timber foundation piling that was present under the former concrete foundation slab. Refer to **Fig. 3** for the theoretical location of existing timber piles within the footprint of the jet grout bottom seal.

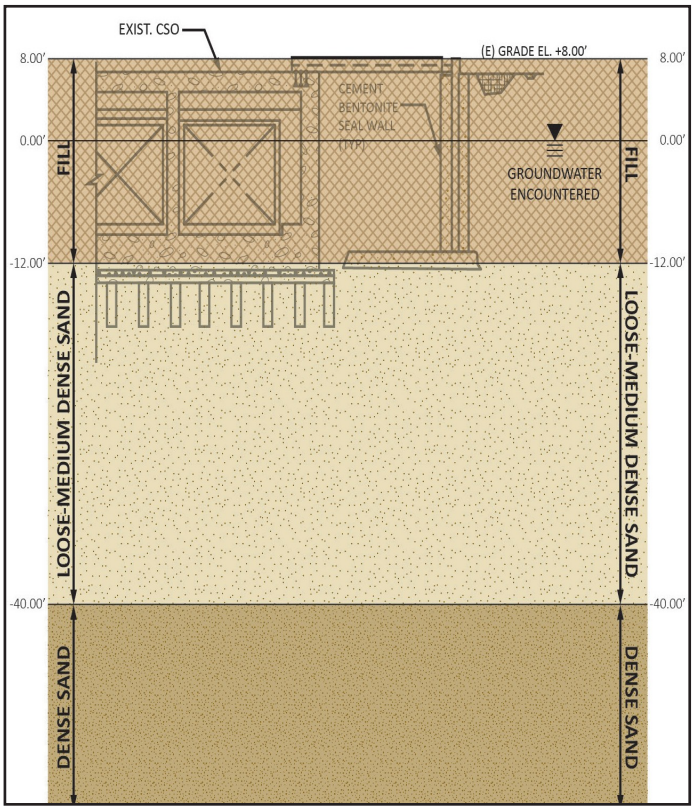


Fig. 6 - Typical Soil Profile

Specific jet grout elements in the planned construction pattern could also be extended to bear on the dense sand stratum at depths of about 45 ft, thereby providing positive end bearing support elements for the proposed CSO bypass chamber extension. Since the as built drawings were, at this point, deemed to be unreliable, it was concluded extending individual jet grout columns to act as foundation elements represented a low-cost risk-reduction measure, considering the potential impacts of differential settlement.

The jet grout bottom seal and deep foundation support elements were designed and specified with performance requirements including 6 ft diameter columns with a minimum compressive strength of 400 psi. The planned thickness of the bottom mat was a minimum of 15 ft below the planned excavation bottom to provide a minimum safety factor of 1.2 against uplift during construction considering only unit weight of the jet grout mass. Side friction was conservatively ignored.

The minimum number of deep jet grout columns needed for foundation support was determined to be 10 with a clear span between columns of 10 ft. Based on this spacing and number, the actual bearing pressure on each column was determined to be 11.4 kips/ft^2 which is less than the maximum allowable bearing pressure; established under New York City's building code of 12 kips/ft^2 for dense granular soils.

Plans, cross-sections, and installation notes were developed for the jet grout bottom seal and foundation support elements based on the following criteria:

- The entire area under the CSO bypass chamber extension should be completely filled with shallow jet grout columns to form a continuous mass of jet grout for seepage control and resistance to uplift.
- The bottom plug columns should begin at an elevation of approximately -7.6 ft and extend to -22.6 ft to form a 15 ft thick bottom plug. This would incorporate the existing slab into the jet grout mass.

© 2016 Deep Foundations Institute

- A double row of jet grout columns should be provided to seal and incorporate, as well as provide toe restraint for, the existing cement-bentonite wall along the perimeter of the CSO extension (north, east, and south walls). These columns should begin at approximately elevation -5.6 ft and extend to -22.6 ft. The two rows would provide jetting from both sides of the existing wall.
- A minimum of 10 deep jet grout columns, 6 ft in diameter should extend to an elevation of -45 ft and bear in the dense sand stratum.
- All jet grout columns, should maintain a center to center spacing of 4 ft, 10 in, at a maximum, to provide sufficient overlapping of columns.

Plan and section views of the jet grout bottom seal, perimeter seal, and deep foundation elements are shown in **Fig. 7 and 8**.

WORK PLAN

Given the site constraints inherent in this extremely congested urban environment and the criticality of the effectiveness of the bottom plug and perimeter seal, a thorough planning process, and documented work plan and quality control procedures were essential to the success of the project. Some of the critical elements of the work plan are described below.

Jet Grout Column Location and Sequencing - The jet grout column locations were carefully laid out and sequenced. Layout and sequencing was performed in consideration of:

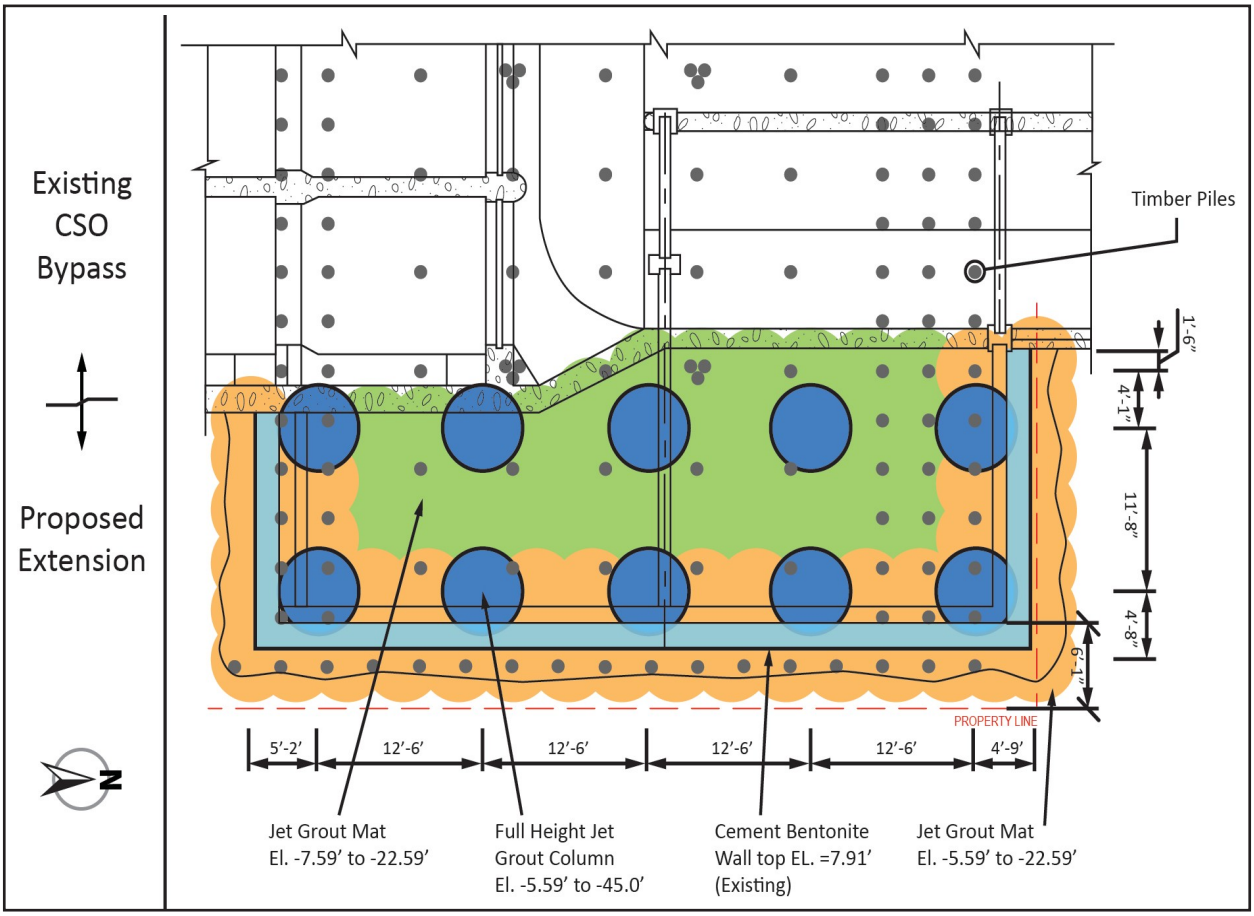


Fig. 7 - Plan View of Jet Grout Columns

© 2016 Deep Foundations Institute

- Theoretical location of existing timber piles;
- Minimum overlap;
- Mitigation of potential “windows” from shadowing by existing piles;
- Angled holes required due to interference with existing electrical duct bank;
- Primary through quaternary holes to provide wet in wet sequencing;
- Toe restraint for the 3-sided perimeter CB cutoff wall; and
- Long columns for foundation support.

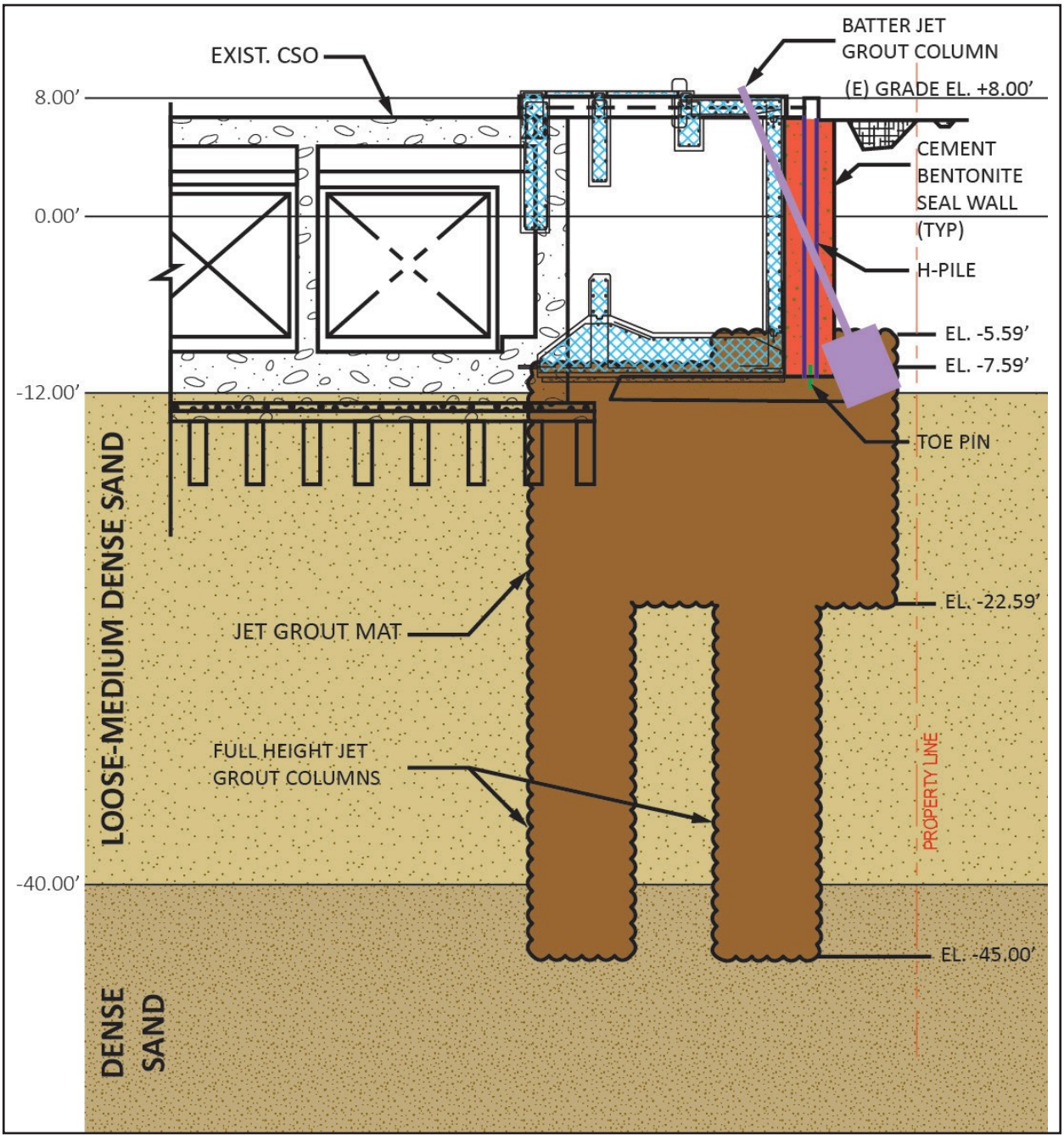


Fig. 8 - Section View of Jet Grout Columns

© 2016 Deep Foundations Institute

A total of 84 jet grout columns were required. Thirteen battered columns were required in the area of an existing duct bank and 10 deep columns were required for foundation support. **Figure 9** provides a plan view of the jet grout column layout and sequencing.

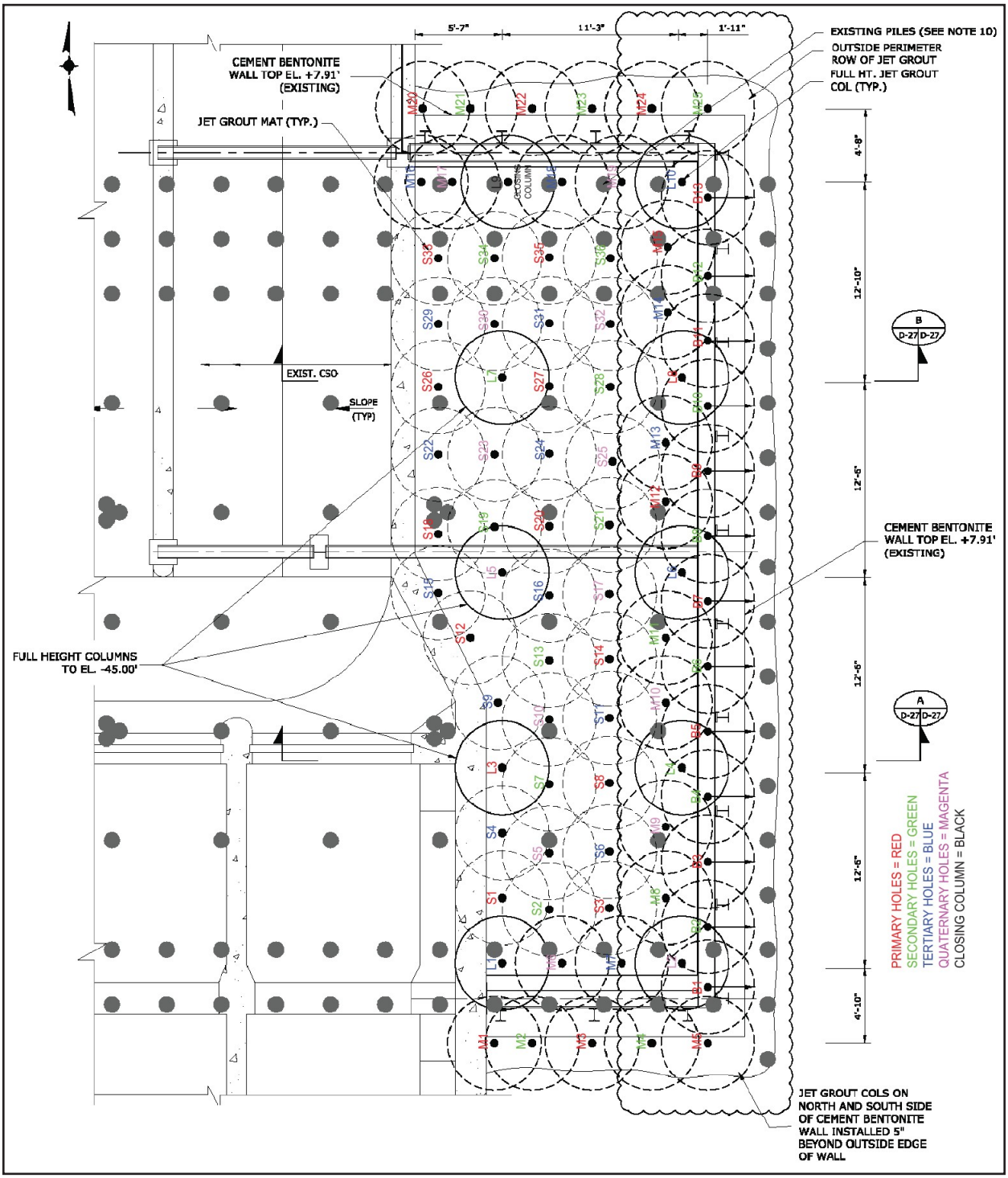


Fig. 9 - Plan View of Jet Grout Column Layout and Sequencing

© 2016 Deep Foundations Institute

Materials and Target Grouting Parameters — Materials and target grouting parameters were based on successful trials and experience with jet grouting operations at other structure locations on the site.

Spoils Handling and Disposal — Wastewater and jet grout spoils were collected and pumped via hoses directly to on site containers. A sacrificial mud mat was placed as a working platform to assist with containment, control and collection of waste spoils. Whenever possible, water was recirculated to minimize waste disposal requirements. Ultimately, wastewater and spoils were removed by vac truck and disposed off-site. A photograph of the spoils containment, handling and disposal operation is provided in **Fig. 10**.

Quality Assurance and Control — Performance measurements, observations and records were diligently maintained to ensure conformance with the target parameters by both Moretrench and the on-site Construction Manager, Hazen and Sawyer and their sub-consultant Mueser Rutledge Consulting Engineers. The following measurements, observations and records were maintained throughout the jet grouting operations:

- Subsurface conditions encountered during drilling
- Jetting parameters including pull rate, rod rotation, grout flow and pressure, and air pressure
- Grout mix specific gravity at the batching location
- Spoils flow return and specific gravity of the jet grout spoil
- Unconfined compressive strength of the jet grout spoil at a frequency of one set per column

In addition, a representative number of core holes were performed in completed jet grout columns to verify the consistency and continuity of the columns. **Figure 11** provides a photograph typical of the core recovered from a jet grout column.

Finally, before commencing full scale excavation, test pits were excavated to observe groundwater inflow conditions, which proved to be minimal.



Fig. 10 - Spoils Containment, Handling, and Disposal



Fig. 11 - Typical Core Removed from a Jet Grout Column

© 2016 Deep Foundations Institute

CONSTRUCTION CHALLENGES

There were a number of unique construction challenges. The jet grout pattern was carefully laid out to provide the best overall coverage of the area. Although the exact locations of the piles were unknown, the jet grout column spacing was determined to provide jetting at each pile location from several directions to minimize the potential for shadowing. It was, at one point, questioned if the timber piles were indeed in place. Significant size chunks of the piles were retrieved in the jet grout return as proof that they were there.

A carefully planned grid spacing layout and wet in wet sequencing were employed to assure that the pre-existing structures were incorporated into the jet grout mass and the mass was continuous and watertight. Columns were also laid out to effectively grout the base of the CB wall. The exterior columns along the long side of the excavation had to be drilled on an angle through the cut-off wall because of the interference of an electrical duct bank immediately outside of the structure. Jetting was performed up through the existing slab so as to incorporate it completely within the jet grout mass.

Contaminated ground conditions necessitated the installation of PVC casings from ground surface to the top of the jet grout columns so as not to carry contamination deeper in the formation. The casings were grouted in place. A diverter was connected to the casings so that the jet grout spoils could be vacuumed into containment vessels and trucked off site as the work proceeded. A sacrificial mud mat was poured at the ground surface for containment of any stray spoil and construction water. All personnel were HAZWOPER trained.

CONCLUSIONS

The jet grout bottom seal and deep jet grout columns proved to be a successful solution to preventing groundwater inflow into the deep excavation and providing foundation support for the proposed extension to the existing CSO bypass chamber. Thorough planning and diligent execution were required to overcome the challenges associated with this extremely congested urban environment with adjacent and conflicting surface and subsurface obstructions including man-made fill, buried foundation slab, timber piles, and underground utilities. An effective bottom and perimeter seal was accomplished as evidenced by the dry conditions of the finished excavation as shown in **Fig. 12 and 13**.

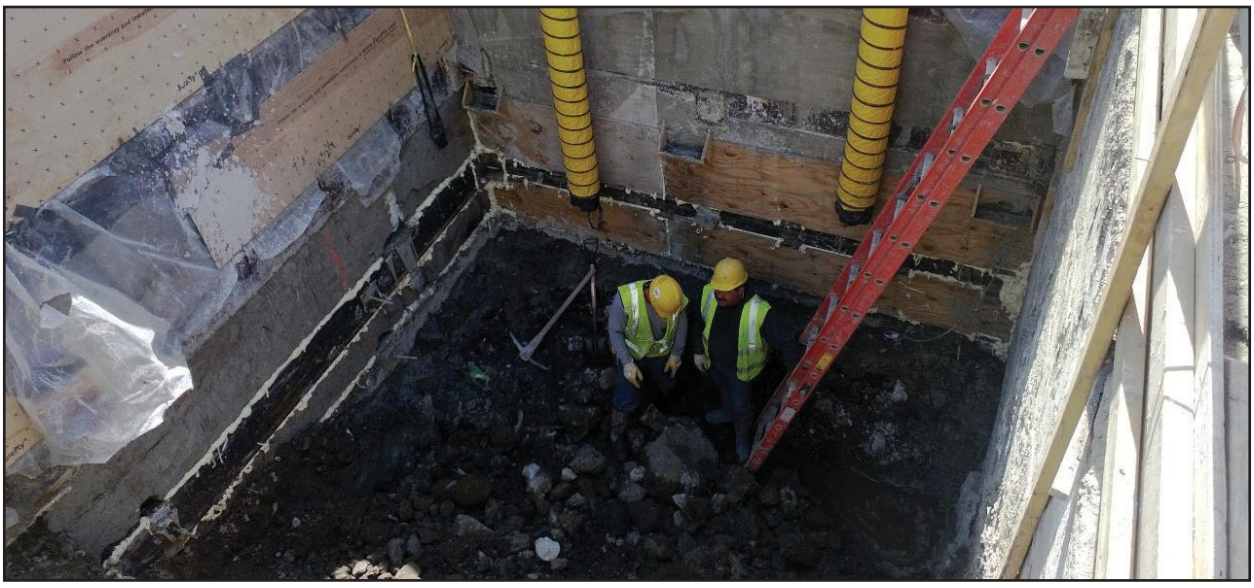


Fig. 12 - Finished Excavation with Effective Seal

© 2016 Deep Foundations Institute



Fig. 13 - Dry Conditions within the Finished Excavation

ACKNOWLEDGMENTS

The authors of this paper would like to acknowledge the NYCDEP for their permission to publish this paper. We would also like to recognize other project participants including Dvirka and Bartilucci, prime consultant; Northeast Remsco, prime contractor; Nicholson Construction, specialty subcontractor for the cement-bentonite wall; Brierly Associates, cement-bentonite wall designer; Hazen and Sawyer, construction manager; Mueser Rutledge, jet grout field quality assurance.

© 2016 Deep Foundations Institute

IMPROVED DRAINAGE BY DRILLED FILTER DRAINS, DAM SAFETY UPGRADE, CASE STUDY

Åke B. Nilsson, M. Sc (Civ. Eng.), Principal Engineer, WSP, Dam Safety, Stockholm-Globen, Sweden, phone +46702818230, e-mail: ake.nilsson@wspgroup.se

Carl-Oscar Nilsson, M. Sc (Civ. Eng.), Dam Engineering Specialist, Uniper, Dams and dam safety, Sweden, phone +46705186525, e-mail carl-oscar.nilsson@uniper.energy

ABSTRACT

The embankment dam at Storfinnforsen is 23 m high and was completed in 1954. The dam has an impervious zone with timber sheet piles and a support fill of glacial till. The pore water pressure in the downstream shoulder has during recent years increased due to deterioration in the sealing function of the timber sheet piling. There is a concern that the high pore water pressure in the glacial till will cause internal erosion initiating at a thin coarse transition layer between the glacial till and the rock fill along the downstream toe. As a remedial action in order to improvement of the leakage resistance a supporting berm of coarse rockfill against the downstream slope of the dam was constructed in year 2013. In addition a new impervious seal in the centre of the dam e.g. a diaphragm wall was considered. However, after further studies and a trial stretch of 40 drains each 20 m long and diameter 0.3 m an alternative with vertical drilled drains in the downstream support was selected. A contract has now started up to drill in a similar way to treat the full length of the dam with drains. The work is in the planning to be completed in 2016.

Keywords: dam safety, upgrade, internal erosion, drains

DAM CONSTRUCTION

Storfinnforsen Embankment Dam was put into operation in 1954. The dam is 330 m long and maximum height is approximately 23 m. The dam is constructed with a central core zone and supporting zones of earth and rockfill on the upstream and downstream sides giving an almost symmetrical cross-section about the dam axis, see Fig. 1. The upstream slope is 2H:1V down to el. 270 and 2.5H:1V below. On the downstream side the slope is 1.75H:1V down to +269 and 2H:1V below. The core zone consists of a timber sheet pile wall with a Zone 5 of glacial till, approximately 2 m wide, adjacent to the wall on the upstream side.

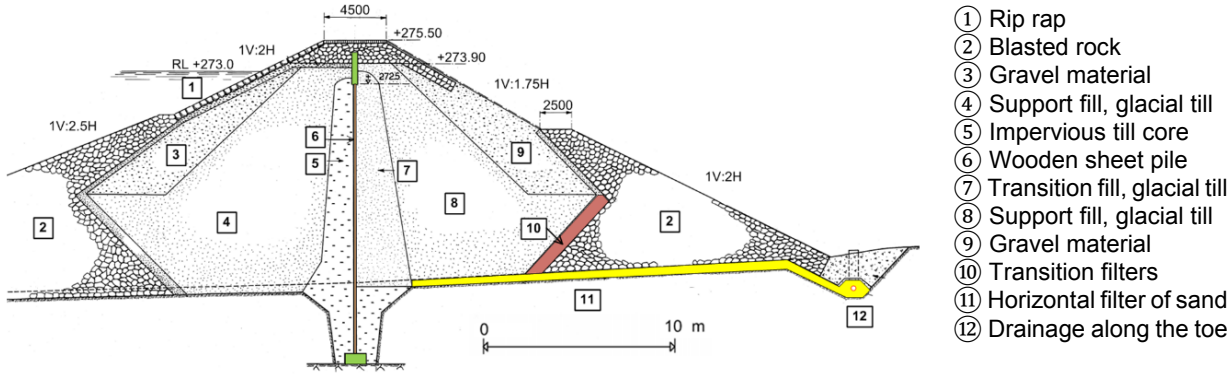


Fig. 1. Construction section of the embankment dam with a timber sheet pile wall sealing

© 2016 Deep Foundations Institute

At the dam´s highest section the timber sheet piling consists of two layers of planks down to 5 m below retention level and three layers below that. The planks are none impregnated timber, each 75 x 150 mm. The upper edge of the planks is at the same elevation as the minimum reservoir retaining level and 0.5 m below the retention level. The dam construction and the early investigations are comprehensively described by Nilsson et al. (2015).

MONITORING OF DAM PERFORMANCE

When the dam was built, a Thomson weir was installed at the downstream toe on the right hand side for seepage/leakage measurements. In addition, four open standpipes were installed through the downstream support fill down to the horizontal filter drain or into the overburden foundation. The total seepage/leakage that has been measured over the years has generally been small approximately 2 l/s. The readings from the four standpipes have generally indicated constant head until 2007, when a slight increase, some 9 mm/year) became evident as shown in Fig. 2. Based on these result, in connection to potential deterioration of timber sheet pile and further investigations started. Work to improve the instrumentation system also started in 2013, and also to be able to measure turbidity more reliably.

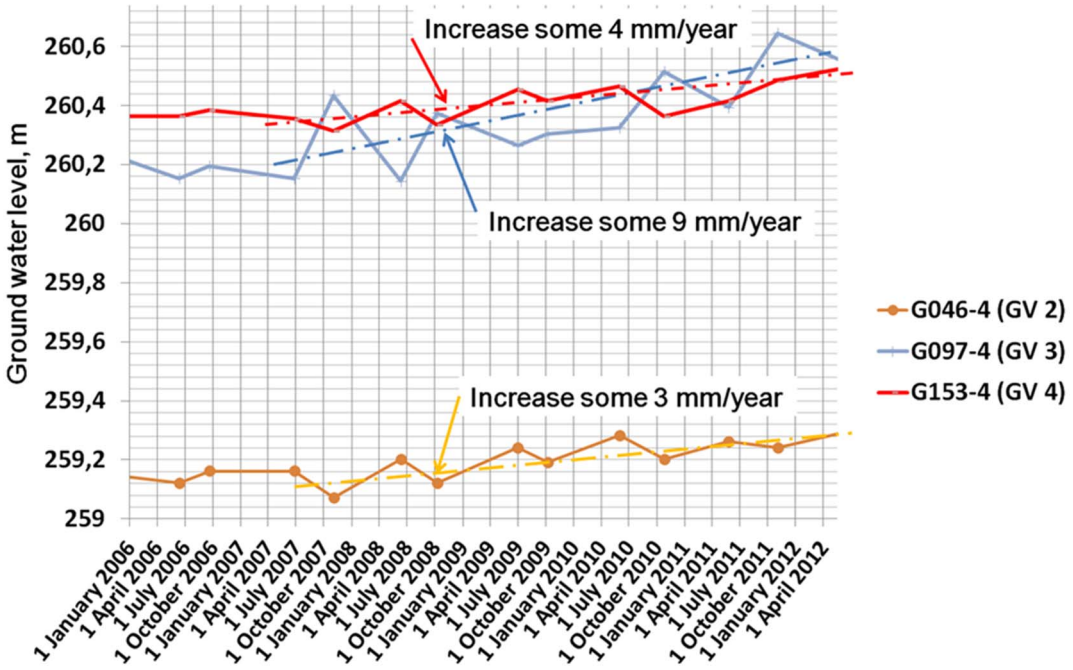


Fig. 2. Measurements between 2006 and 2012 in four standpipes installed in the subgrade or in the horizontal filter layer during construction in the 1950

FURTHER INVESTIGATIONS AND INSTRUMENTATIONS

Geotechnical field investigations with sampling were performed in June 2012 and from December 2012 to January 2013. The result showed generally good agreement with expected conditions based on the information from the construction. However, signs of contamination of fines from upstream till material were found in the transition filters. In addition to geotechnical investigation, the instrumentation was supplemented with a number of piezometers in the downstream support fill, where the material is made up of impervious material of glacial till type. However, layers of a more pervious character (clean gravel and sand) were encountered during the investigations. The measurement frequency of manual measurements was therefore increased to once a week. Both hydrostatic pressure and temperature were measured at a number of points.

© 2016 Deep Foundations Institute

The standpipes which record the ground water head in the subgrade and in the horizontal filter layer below the downstream support fill indicate head within the horizontal filter layer. However, directly above the horizontal filter layer the readings indicated ground water pressure in the order of 4 to 5 meters of water as shown in Fig. 3. The readings show a yearly variation probably due to variation of precipitation and snow melt.

The gradient in the area just above the horizontal filter layer is high. The filter layer is a sand layer and the filter functions to the glacial till above the filter are satisfactory and the risk of backward erosion is judged to be small. However, there are also fairly high pore water pressure adjacent to the downstream transition layer. This material is a coarse gravel (8-64 mm) susceptible for continues erosion of the upstream glacial till with a content of fines (0.063mm) of up to 30%, see Foster and Fell 2001.

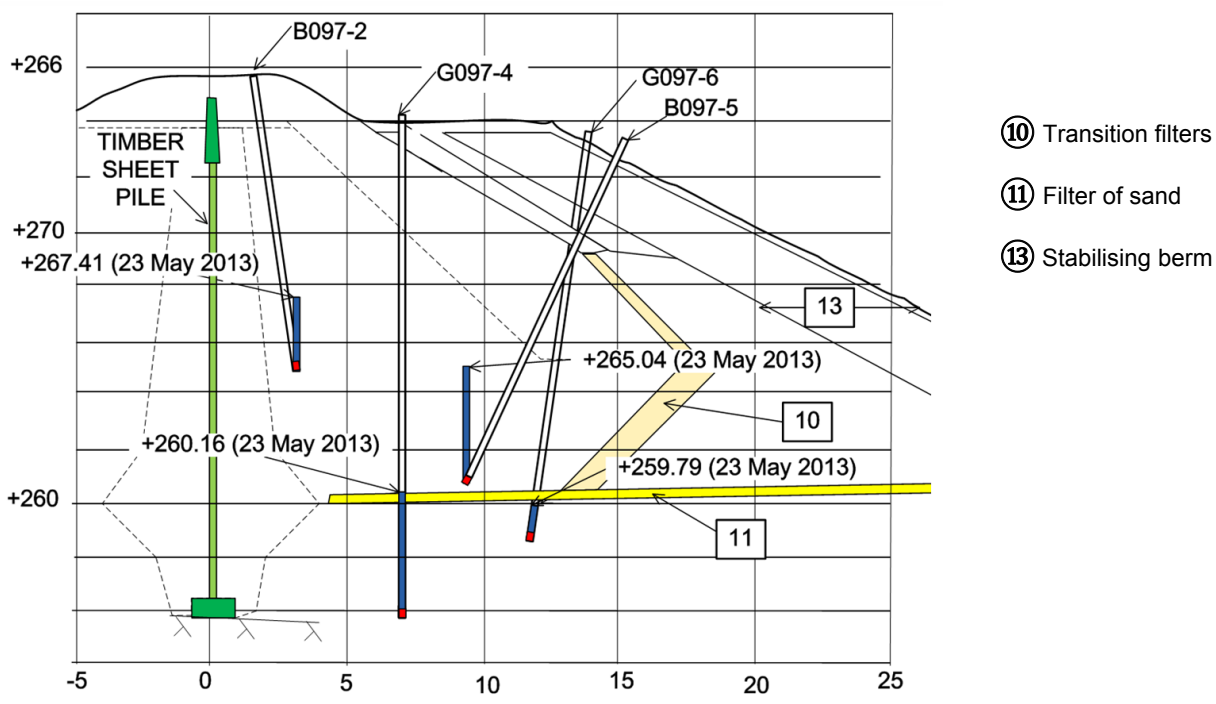


Fig. 3.
Pressure head at the piezometer gauge in support fill and in standpipes in the foundation

The measurements were performed manually up until March 2013, when automatic measurements were started. An increase in pore water head has been noted during 2013. Based on these result it was decided to improve the dam.

In Table 1 the set of hydraulic conductivities judged to be the best fit to the measured pore water heads is presented. The downstream support fill is selected to have 100 times higher conductivity in the horizontal direction compared to the vertical direction. This difference is assumed to be justified due to the permeable horizontal layers that were encountered in the glacial till during investigations and also because the material was spread and compacted in layers during construction. Both the instrumentation of a typical section and the flow calculations results shown in Fig. 4 indicate a risk of pore water pressure in the area for the transition layer upstream the rockfill zone.

© 2016 Deep Foundations Institute

Table 1. Hydraulic conductivities used in the presented flow calculations

Zone		Material	k (m/s)
A	Support fill	Gravel	1·10-4
B	Upstream support fill	Gravelly glacial till	1·10-7
C	Impervious fill	Clayey glacial till	1·10-8
D	Transition layer	Glacial till	1·10-7
E	Downstream support fill	Gravelly glacial till	1·10-7
E	Downstream support fill	Gravelly glacial till	1·10-5
F	Horizontal filter	Gravelly sand and sandy gravel	1·10-6
G	Downstream toe	Rock fill	1·10-2
H	Subgrade	Glacial till/gravel	1·10-7
I	Upstream toe	Rock fill	1·10-2
J	Subgrade, rock	Rock	1·10-11

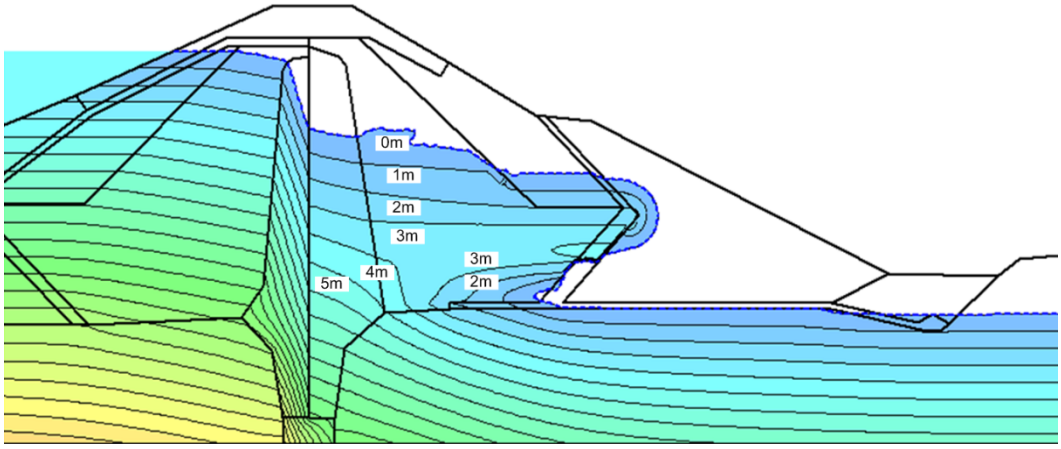


Fig. 4.
Pore water head from SEEP/w calculation using hydraulic conductivity according to Table 1

DAM SAFETY RISKS

The concern from dam safety point of view is a possible high pore water pressure in the downstream support fill adjacent to the transition layer to the coarse rock fill at the toe. The transition layer is too coarse to protect against backward erosion in the downstream till. The wooden sheet pile wall has probably disintegrated and increased the pore water pressure. At present the wooden sheet pile is not expected to assist as a sealing element.

The difference in gradation of the blended filter and the glacial till support fill constitutes a weakness which may result in internal erosion being initiated if water flow occurs between the glacial till and the filter. An event tree where internal erosion is initiated in the filter transition in the downstream edge of the support fill of glacial till can be described in accordance with Fig. 5. There are currently no clear indications of internal erosion in the filter interface to the downstream coarse filter. However, a degree of turbidity has been reported in the leakage water in the Thomson weir. This information indicates that the current situation is between box 2 and box 3 in Fig. 5.

© 2016 Deep Foundations Institute

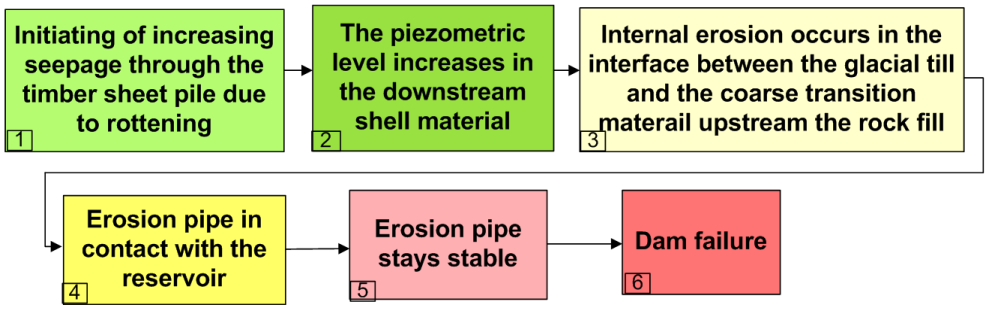


Fig. 5.
Event tree for a failure mode

REMEDIAL ACTIONS

New drainage and supporting berm to avoid catastrophic failure

The old drainage system was in bad condition and was filled with gravel in order to avoid movements in the fill if the drainage pipes in the long term would collapse due to the additional load of the supporting berm. A new drainage system, consisting of two parallel concrete pipes diameter 225 mm with open joints, was installed about two meters lower than the existing drainage system. The new measuring weir was constructed to collect the seepage from the dam. The ground water from the downstream side was collected separately and discharged downstream of the measuring weir, see Fig. 6.

The measures taken comprise a supporting berm to avoid a catastrophic failure if internal erosion is initiated. The philosophy of strengthening of the dam by adding the supporting berm is that the berm increases the time available for self-healing, and gives the operator time to perform strengthening works, lowering of the reservoir, or for emergency actions, if these are called for.

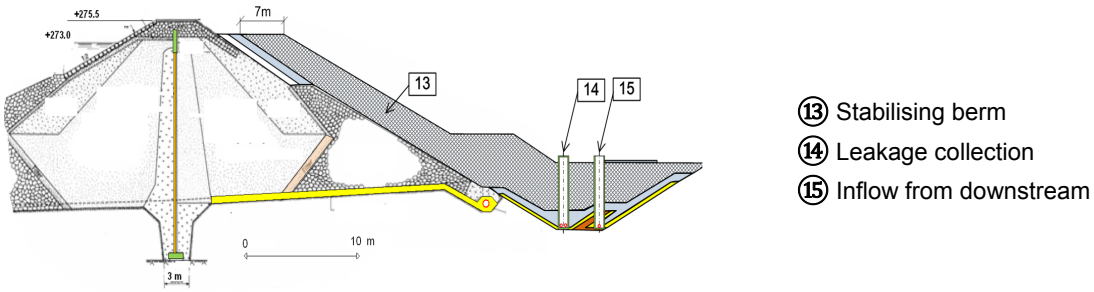


Fig. 6.
New drainage and supporting berm of rockfill

Measures to avoid initiation of internal erosion

Even if the supporting berm was judged to give time to avoid a catastrophic failure, further actions were considered in order to avoid an initiation of internal erosion. Following remedial actions were considered;

1. No further enhancements are made (called the zero option).
2. A new impervious seal is built either downstream or upstream of the centrally located existing seal consisting of timber sheet pile walling and glacial till.
3. Improved drainage by constructing a chimney drain of drilled holes filled with filter material.

© 2016 Deep Foundations Institute

The advantages and disadvantages of the above listed alternative options were evaluated. In order to provide a somewhat subjective comparison between the various alternative options (zero option, sealing or drainage), a number of factors which may indicate for or against the options are listed below. Each factor has been subjectively evaluated as positive (+) or negative (-), see Table 2. The degree of advantage or disadvantage is corresponded by a colour according to legend in the lower part of the table. It must be realised that the importance of the listed factors varies greatly depending on the option which means that it is not possible to sum up the number of positive and negative factors to get a rating of the alternative options.

Table 2 Subjective evaluation of the three options: "zero option", "sealing" and "drainage"

Factor	0-option	Sealing	Drainage
Dam safety risks – long term	- -	+	+
Costs	N	-	-
Dam safety risks – during construction	N	-	-
Require sealing adjacent to existing concrete structures	N	-	N
Require sealing to rock in the subgrade	N	-	N
Maintenance and surveillance	-	-	-
Knowledge of Swedish consultants and contractors	N	-	N
Risk of increased costs	-	-	-

LEGEND: Degree of advantage/disadvantage

Very positive (+)	Neutral (N)	Very negative (-)
Positive (+)		Negative (-)
Slightly positive (+)		Slightly negative (-)

On the basis of the evaluation, "sealing" appears to be the most advantageous option from a long term dam safety perspective, but the option also has several disadvantages of which cost is the biggest, but also dam safety risks and design/execution difficulties can be foreseen during construction. The "0-option" is judged not to be acceptable since the long term dam safety risks are assessed to be "negative" or possible "very negative".

"Drainage" appears to be a good option from a long term dam safety perspective, but a few negative factors can be seen e.g. maintenance and surveillance. The drains are expected to prevent high piezometric head in the interface to the existing rock fill toe and significantly reduce the risk of internal erosion. The long term safety is secured by surveillance by piezometers and leakage measurements and possible preventive actions can be taken if these are called for. The advantages of the drainage method are:

- Project planning and design as well as award of a contractor available in Sweden and implementation of the contract can be undertaken relatively quickly once a preliminary geotechnical investigation has been completed.
- The cost is comparatively low.
- Uncertainties relating to construction are judged to be relatively small (no connection is needed to the concrete dam or the underlying rock).
- Monitoring can be easily undertaken by measuring the water level in the strainer pipes, and additional installed standpipes and piezometers. A degree of maintenance in the form of infiltration of water is thought a possibility if the effect of the drains were to be reduced over the years. Thresholds for the instrumentation measurements are possible to be implemented in advance in the surveillance to improve the evaluation in the future.
- The option with a new sealing in the future is not ruled out and the vertical drainage arrangements are designed in such a way that i.e. a diaphragm wall is still possible to be constructed.

© 2016 Deep Foundations Institute

The disadvantage of the drainage method is mainly:

- It was not clear as to how successful the method will be and whether the existing horizontal filter and drainage in the subgrade is sufficient enough to drain the water collected in the vertical drains. An initial trial strip in the field was at that stage considered necessary.

Trial strip with vertical drains in the downstream supporting fill

In addition, in order to avoid the initiation of internal erosion, vertical sand drains were decided to be tested in a trial strip. A test section 50 m long with 53 drains were installed in 2015, partly with 2 m spacing and partly with 1 m spacing as shown in a plan in Fig. 7 and section in Fig. 8. The borings for the installation of drains were performed with casings of diameter 273 mm. In the casing a strainer pipe 50 mm diameter and slits 0.5 mm was centred and the pipe surrounded by filter sand (gradation 0.5 – 0.7 mm) and the casing recovered in steps as shown in Fig. 9. The hydraulic pressure head before and after installation of the drains are shown in Fig. 10.

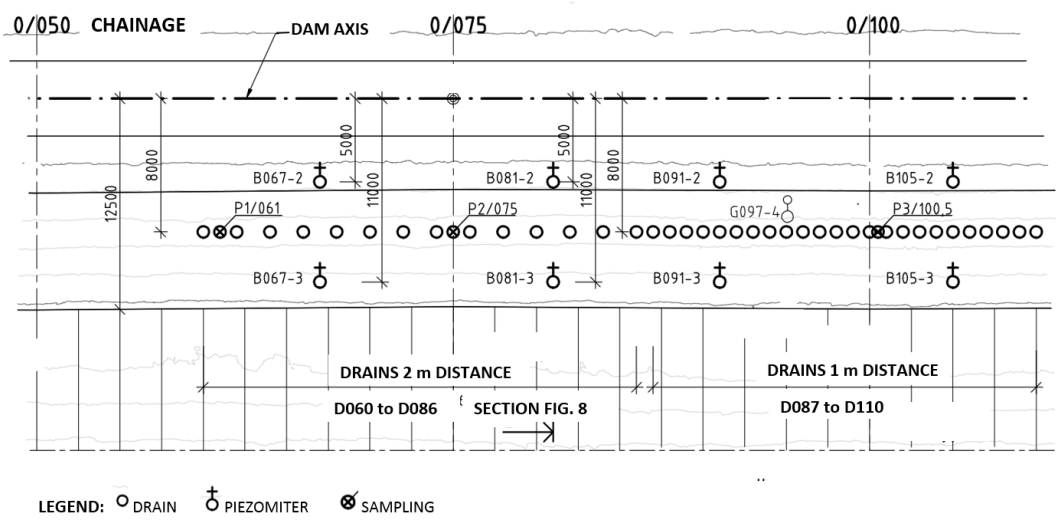


Fig. 7.
Plan – Trial strip with drains 2 m and 1 m spacing and piezometers located 3 m upstream and downstream of the drains

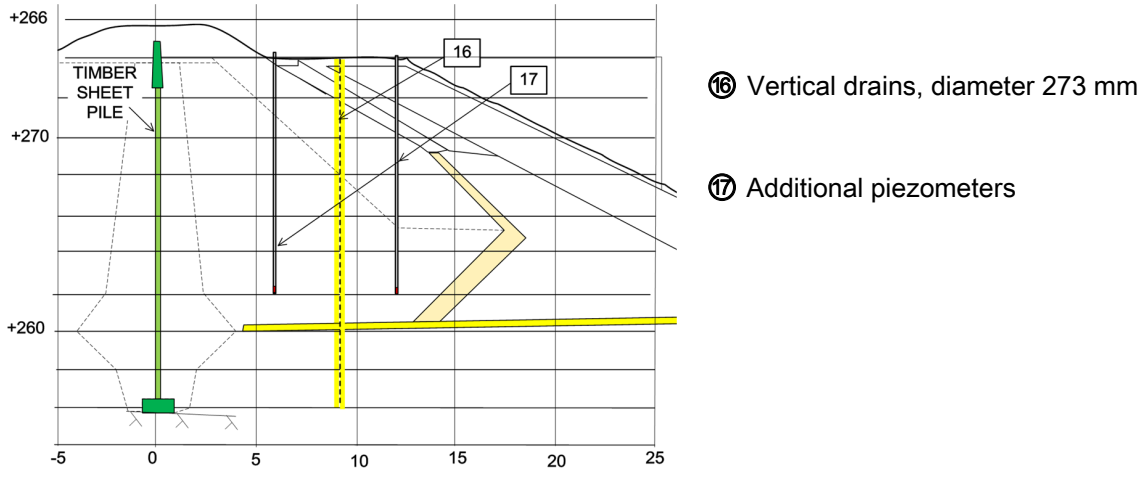
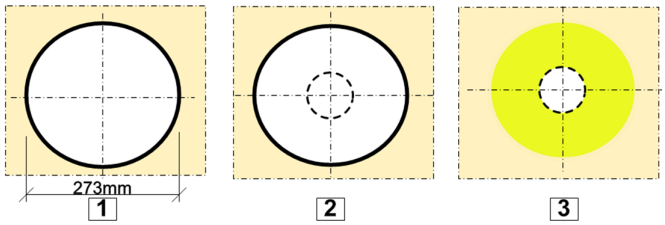


Fig. 8.
Location of vertical drains and installed piezometers

© 2016 Deep Foundations Institute



1. Drilling of casing from the top of the berm down to underlying rock
2. Installation of strainer pipe centred in the casing
3. Filter sand is filled in the casing and the casing is withdrawn

Fig. 9.
Stages for the drilling of drains

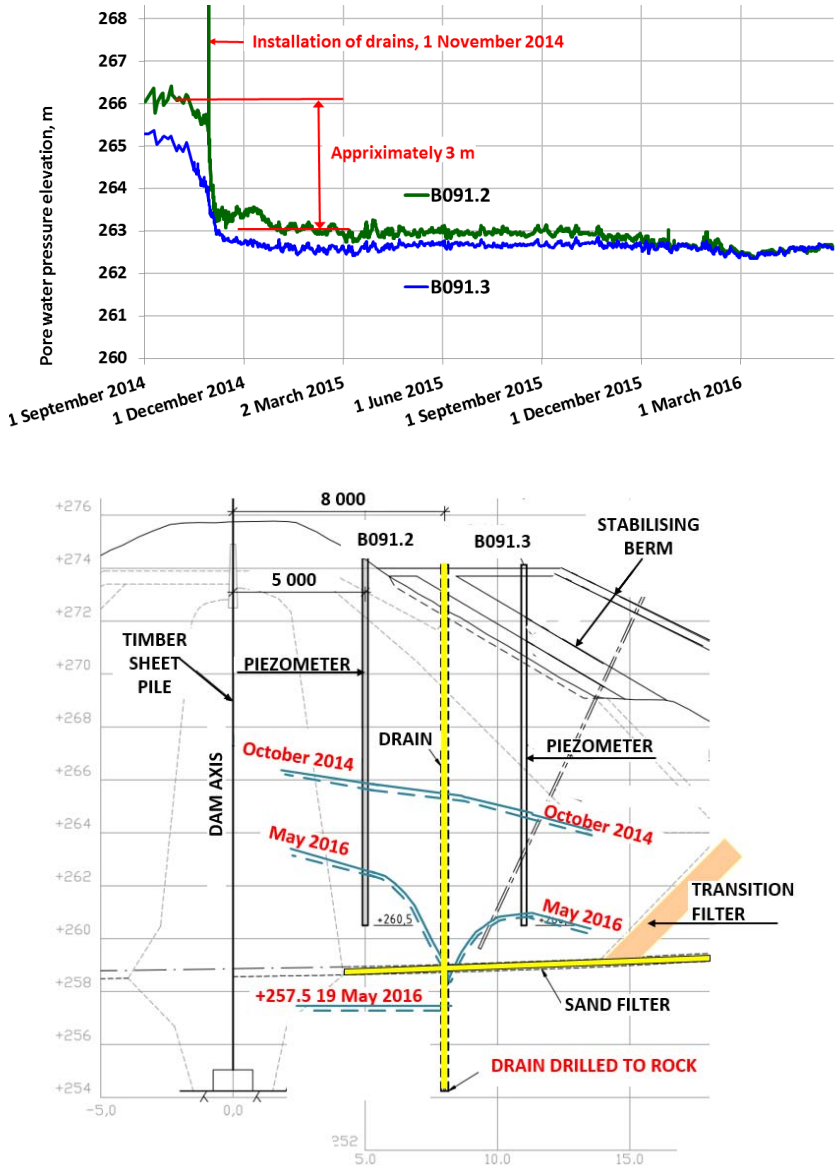


Fig. 10.
Piezometer readings before and after installation of drains (top) and in a typical section showing drain and pore water head before and after installation of drains (bottom).

© 2016 Deep Foundations Institute

The drains in the trial strip were proven to be effective to reduce the pore water pressure significantly. The reductions of pore water pressure one year after the installation of the drains were evaluated to be in the range 2.5 to 3.5 m of water in the part of the strip were the drains were installed with 2 m spacing, and 3.0 to 4.0 m in the part were the drains were installed with 1 m spacing. Thus, drains have now started to be installed along the whole length of the dam.



Fig. 11.
Installation of drains (top left), centering of strainer pipe (top right). Strainer pipes 0.5 mm slots (bottom left), Filter sand 0.5 to 0.7 mm grain size (bottom right)

Installation of drains along the whole embankment dam

The installation of drains along the whole dam is in progress. The spacing was chosen to 1 m distance and the lengths in the range 15 to 25 m. The total drilling length will be some 3 000 m and the work is expected to be completed within year 2016.

© 2016 Deep Foundations Institute

SUMMARY AND CONCLUSIONS

This case shows that small changes in instrumentation readings may be important. The small measured change in head, together with a sealing element which might have a decreasing effectiveness started in this case further investigations. High pore head was found in support fill and was found to be increasing. The sealing of timber sheet piles was judged to be partly rotten after some decades. With the knowledge that the gradation of a downstream transition layer was susceptible to initiate internal erosion new drainage and a supporting berm was first constructed. These measures were judged to be enough to avoid a catastrophic failure, but were not stopping possible initiating of internal erosion. Different additional remedial measures were studied and finally it was decided to install vertical drains in the downstream support fill. This alternative was found to be effective in a trial strip. The installation is now in progress to continue along the whole dam. The drains together with the supporting berm have the goal to achieve satisfactory dam safety in the long term.

ACKNOWLEDGMENTS

The authors would like to thank Professor Kaare Höeg, special adviser at the Norwegian Geotechnical Institute (NGI), for his valuable support and helpful comments during his two visits to the site.

REFERENCES

Foster, M.A. and Fell, R. 2001, Assessing embankment dam filters that do not satisfy design criteria. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 5, May 2001, pp. 398-407.

Nilsson, Å., Torstensson, I., Nilsson, C.-O. 2015. Dam safety upgrade of earthfill dam with timber sheet piles walling - Case study with vertical drains, ICOLD Q98 – R-28, Stavanger. pp 409-428.

© 2016 Deep Foundations Institute

REHABILITATION OF THE KÖLNBREIN DAM / AUSTRIA

Frank Haehnig, Züblin Spezialtiefbau GmbH, Stuttgart, Germany, +49 711 7883 749, frank.haehnig@zueblin.de

Kurt Kogler, Züblin Spezialtiefbau Ges.m.b.H., Vienna, Austria, +43 1 22422 2601, kurt.kogler@zueblin.at

ABSTRACT

With a height of 200 m, a crown width of 626 m and a concrete volume of 1.6 million m³, the Kölnbrein Dam in the upper Malta valley in Carinthia / Austria is the largest concrete dam in Austria. Boasting a storage capacity of 200 million m³ with a received water load of 54 MN, this is one of the strongest arch dams in the world.

During its construction from 1974 to 1977, the dam was equipped with a comprehensive monitoring system capable of comparing developing deformations and stresses with the design values.

With a planned reservoir level of 1,900 m a.s.l. the dam saw the reservoir level of 1,860 m a.s.l. breached in the year 1978. This led to extremely high seepage losses of up to 200 ltr/sec and a rise in the foundation water pressure up to 100% of the dam's height. These observations were caused by the development of horizontal cracks in the area of the middle dam blocks, between the lowest inspection walkway and the waterside.

As a consequence, power plant operations had to significantly be reduced and a number of additional measures were carried out – though these did not lead to the desired sealings.

The developer subsequently decided that the dam should undergo a general rehabilitation from 1998 to 1994 in accordance with a concept developed by Professor Lombardi. This concept involved erecting a 50 m high support structure with a concrete volume of 500,000 m³ on the airside of the dam, along with the addition of neoprene bearings to transfer 1.2 million tons of the horizontal load. These measures were able to reduce the deformations in the dam enough so that mostly load-bearing injections into the cracks in the dam's concrete could successfully be carried out by way of targeted grouting measures using injection resin (RODUR) specifically developed for this purpose.

In the years 1998 - 1994, a total of 130,000 m of grouting, monitoring and control drillings were executed, and around 200,000 kg of synthetic resin was injected into the cracks in the dam's concrete.

These measures have ensured the dam‘s uninterrupted operation since 1994. A number of small re-injections were again carried out on the contact surface in the years 2011 and 2012.

Keywords

Arch dam construction, core drilling, resin grouting, grouting work for grout curtain

Introduction

With a height of 200 m, a crown width of 626 m and a concrete volume of 1.6 million m^3, the Kölnbrein Dam in the upper Malta valley in Carinthia / Austria is the largest concrete dam in Austria. Boasting a storage capacity of 200 million m^3 with a received water load of 54 MN, this is one of the strongest arch dams in the world.

© 2016 Deep Foundations Institute

The complex was constructed between 1971 and 1978 as an annual storage reservoir by the Austrian power company Draukraftwerke (ÖDK) and the dam's first maximum storage could be reached in 1979.



Figure 1: Kölnbrein Dam in the midst of the "Hohe Tauern" mountains

Structural dimensions

The dam is located at a sea level of approx. 2,000 m.
Structural height: 200 m
Crest width: 12 m
Width at the base: 36 m
Concrete volume: 1.6 million m³
The structure consists of 30 individual blocks with a block width of approx. 20 m, an inspection gallery along the base, as well as 4 more horizontal interior inspection galleries.

Geology

The dam is located on the south side of the "Hohe Tauern" mountain range, which is characterised mainly by granite and gneisses with schistose inclusions.

The dam's base is located in a U-shaped valley carved out by glacier ice within a granite body that only arched up in geologically recent times. Partly heavily schistosic gneisses with occasional high shares of

© 2016 Deep Foundations Institute

mica occur at the foot of the left bank. Clay mylonite fissures of several centimetres thickness that run parallel to the schistosity areas indicate higher partial mobility under tectonic influences.

The left flank is made up of lightly fissured, but plate-like gneisses.

The very different deformation behaviour of the slate zones in contrast to the granite areas plays an important role in the later deformation behaviour of the dam.

Behaviour of the dam after the first maximum storage

At the time of its construction, the Kölnbrein Dam was already equipped with a comprehensive monitoring system that enabled a precise comparison with the design values.

In autumn 1978, an extremely high increase in leakage water losses with values of up to 200 l/s was identified at the lowest inspection gallery. In addition, a sudden increase of the foundation water pressure occurred at the base area.

It could be identified relatively quickly from these observations that a tension zone with horizontal cracks up into the backwater area had developed in the upstream foundation area of the tallest dam blocks, and the effect of the grout curtain became ineffective in this area.

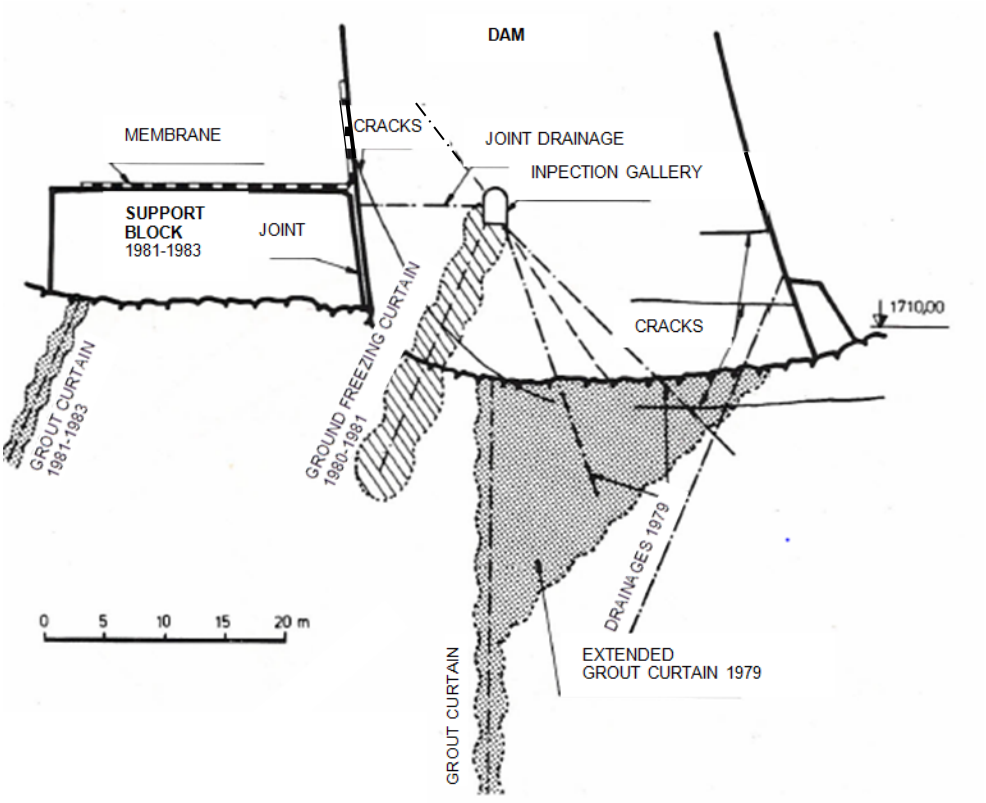


Figure 2: Supplementary measures in the years 1979-1985

Between 1979 and 1985, numerous rehabilitation measures were carried out in order to maintain a limited dam operation, such as

- supplementation of the grout curtain by cement grouting from the lowermost inspection gallery to reduce the foundation water pressure

© 2016 Deep Foundations Institute

- injection of PU resins in the upstream tension zones of the dam concrete
- construction of a provisional freezing curtain with a surface of approx. 4,000 m² and a thickness of 5 m, which was frozen during reservoir rising and thawed again during lowering
- construction of an upstream support block in form of a 2 to 10 m thick reinforced concrete slab and placement of an elastic sealing film with a size of approx. 4,000 m² as well as its connection to the dam concrete
- supplementation of the sealing films on the surface of the support block and supplementary sealing injections in the bedrock.

As the specified measures did not lead to any satisfactory improvement, the ÖDK appointed Dr. Lombardi with the elaboration of a rehabilitation project with the target of increasing stability in cracked area and ensuring an unrestricted operation of the Kölnbrein Dam.

Project ideas for rehabilitation

Over the years, the various repair attempts confirmed that the loaded dam blocks cannot be repaired once and for all by injections and accompanying measures alone.

After intensive consideration of all advantages and disadvantages of the different variants, the decision was taken to construct a downstream support arch dam that enables support of the dam in the order of 1.2 million t and thus reduces the transverse forces to the same extent.

Neoprene layers especially designed for this purpose were selected as bearing element. These can be installed and adjusted in the unloaded condition during low backwater and an open joint, on the one hand, while they cannot be misadjusted by any faulty manoeuvre due to human or mechanical error nor by malicious intent, on the other hand.

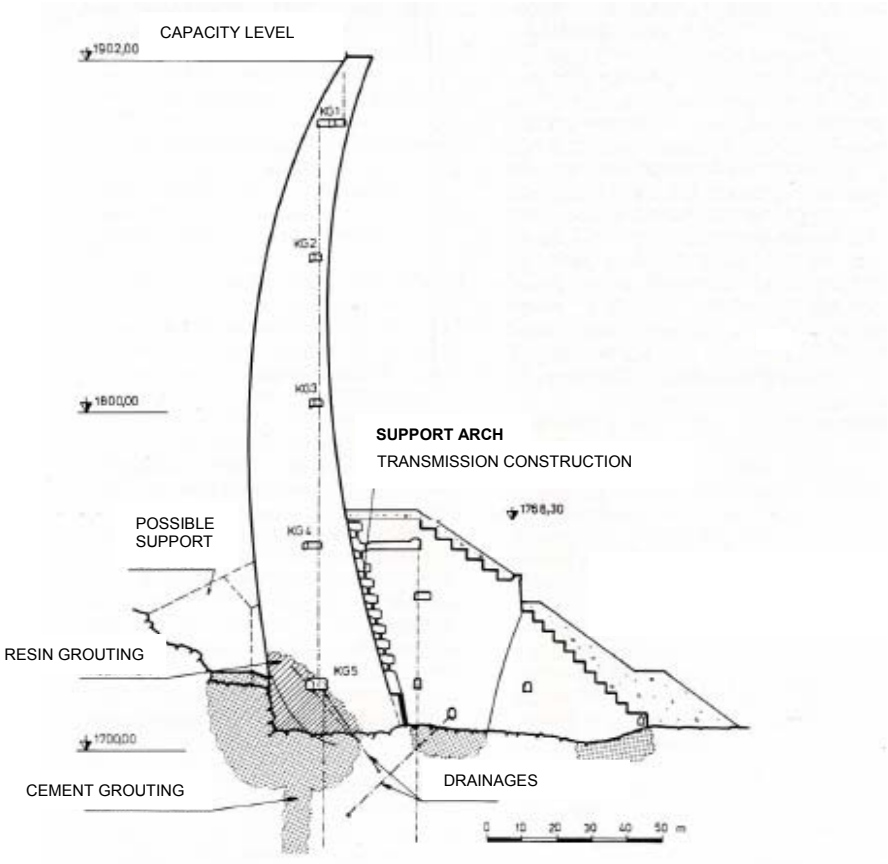


Figure 3: Design of the support arch dam

© 2016 Deep Foundations Institute

Construction of the support arch dam

The support arch dam is an arch-gravity dam with a total cubage of 460,000 m³ concrete and a height of 70 m. Due to the base width of 65 m, it was provided with a longitudinal joint and radial joints at a distance of 35 m. All block joints were fitted with two independent injection systems for repeated application.

Concreting was carried out in zones with a height of 3 m and cubages of up to 4,000 m³ in one work step. Closed ring cooling pipelines in the dam concrete ensure relevant cooling during the concreting works.

As usual, relevant inspection galleries and measuring niches are located in the support arch dam for carrying out the block joint injections and to perform the measuring observations.

The downstream bottom outlet pipe had to be extended accordingly through the support construction. For this purpose, a valve chamber was erected for mounting the hollow jet valve as well delivery of the bearing elements. For landscaping reasons, the support arch dam was filled up with a stone fill of approx. 130,000 m³.

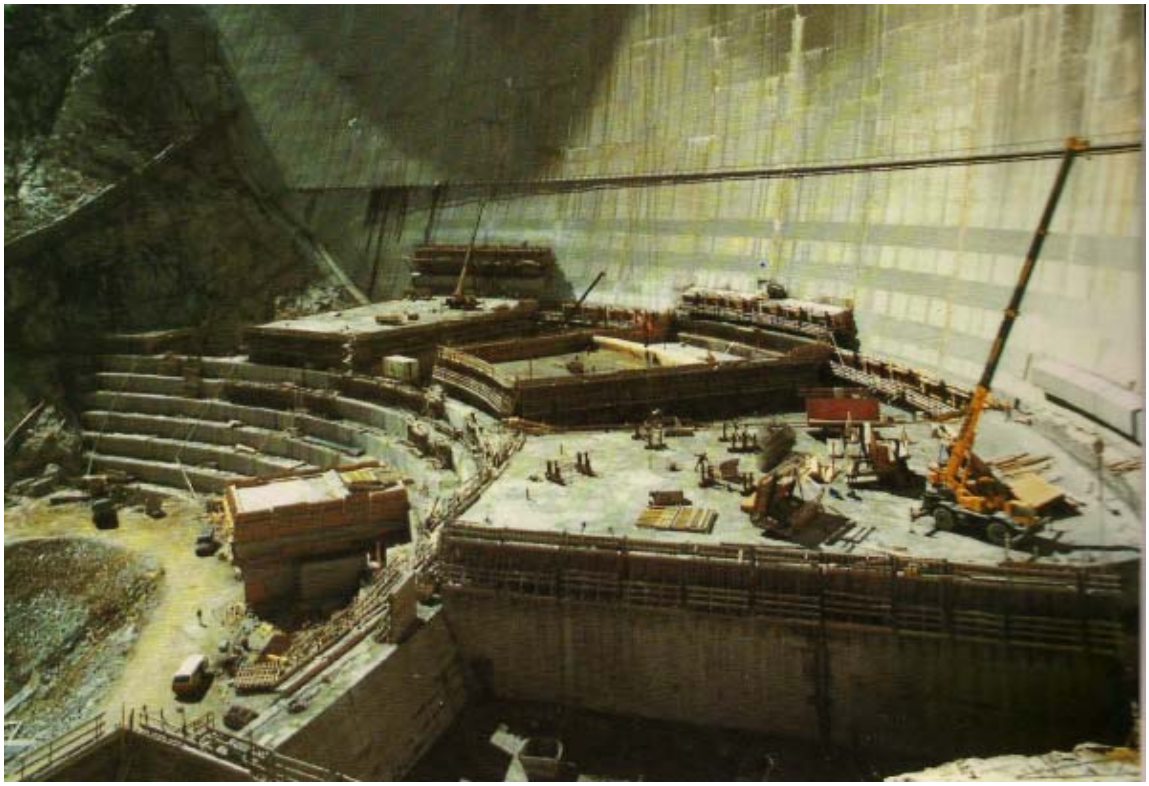


Figure 4: Support arch dam under construction

Drilling and grouting works

The structural interaction of support arch dam and grouting in the entire rehabilitation concept of the dam gives the grouting task an unusual and singular meaning here.

On the one hand, any action must be taken with particular caution, so as not to provoke additional cracks with the grouting process, and on the other hand, the demands on the outcome of the treatment remain upright with the

- most comprehensive sealing of all penetrabilities in the rock beneath the base area possible

© 2016 Deep Foundations Institute

- consolidation of the rock masses beneath the upstream support block as well as the base areas of the dam and the support construction
- as well as non-positive sealing injections in the dam concrete.

The grouting works were divided into 7 grouting areas and designated in phases from “M” to “R”. In addition, a drainage system called “T” is also constructed.

The grouting concept of ÖDK, adapted by Prof. Lombardi, is based on theoretical considerations of the injection of stable cement suspensions in the bedrock and connects these approaches for the area of the dam concrete with the experiences of the RODIO Group, which carried out successful dam repairs with RODUR resin injections. The novelty of this grouting concept was the consistent formulation of grouting criteria in every yoke in which the supplied grouting energy is subject to a variable limitation as the product of quantity and pressure.

A hyperbolic relationship between the uptake of grouting material and the permissible grouting pressure is stipulated for the cement grouting. The permissible maximum pressure is reduced in accordance with the relevant grouting quantity taken up, whereby the specified limit pressure and a limit uptake must also be complied with.

For resin injections into concrete it is primarily the volumes to be injected that are defined, namely as a function of the crack width and the desired range, which on the other hand depend on the distance to the next borehole.

The product of grouting pressure and applied grouting volume is referred to as “grouting intensity”. The injection should be stopped when the relevant value is reached.

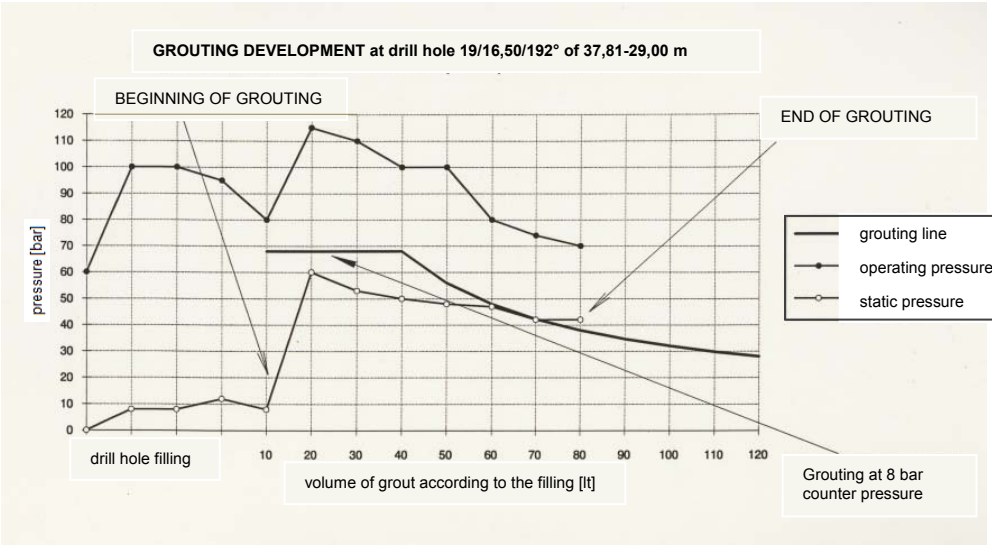


Figure 5: GIN diagram

This procedure became known under the term “GIN” (grouting intensity number) and has meanwhile become the set of rules in various standards and regulations for grouting works.

The drilling and grouting works were carried out mainly in the years 1990 to 1993 in two-shift operation, primarily during the construction periods from April to October, as no works could be carried out in the winter months because of the extreme weather conditions and risks of avalanches due to the altitude.

© 2016 Deep Foundations Institute

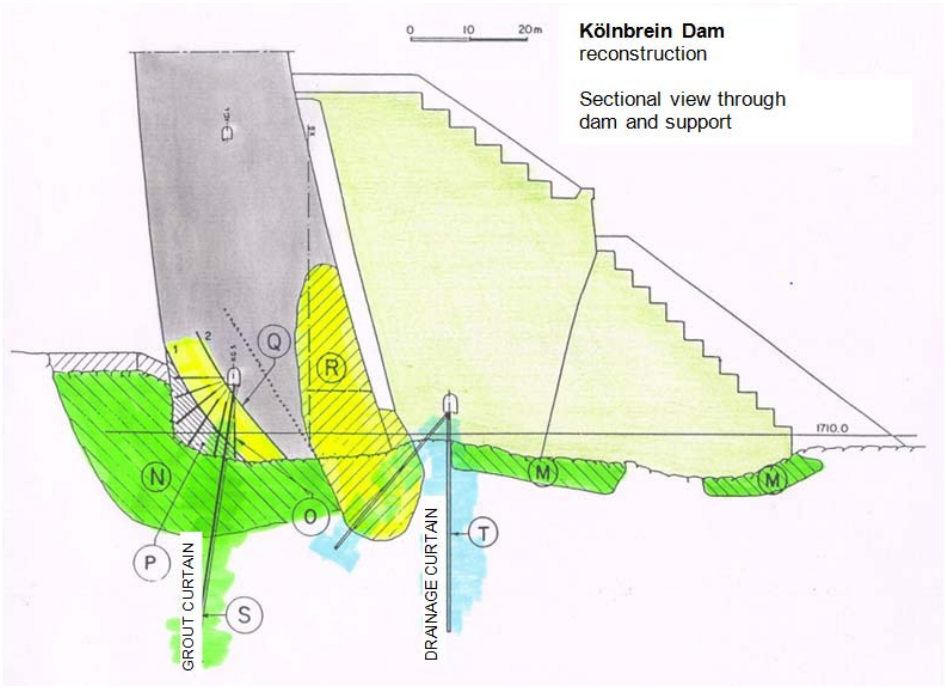


Figure 6: Grouting zones

During this period, a total of approx. 130,000 m injection wells as well as test and control drillings were produced, as well as approx. 200 t of RODUR resin injections carried out.

Table 1: Measures in the grouting zones

Zone	Description	Grouting Material	Water level during grouting	Target
M	Rock under support arch dam	Cement	Free	Consolidation (completion 1990)
N	Rock under main dam and upstream dam floor	Cement	Middle	Consolidation (completion partly 1990)
O	Concrete upstream area	Rodur	Middle	Anchorage (deferred)
P	Concrete crack zone 1	Rodur	Middle	Strain hardening, tightening (completion since 1990)
Q	Concrete crack zone 2	Rodur	Middle	Strain hardening, tightening (completion since 1990)
S	Tight curtain in the rock	Rodur	Middle	Tightening deep curtain
R	Concrete / rock downstream area	Rodur	Not definitely known	Strain hardening (deferred)
T	Rock drainage downstream	---	Free after grouting	Uplift reduction

© 2016 Deep Foundations Institute

Drilling works

The drilling works for improvement of the grout curtain in the bedrock were carried out using 2 SIG hammer drills with hydraulic rock drilling hammers with a drilled diameter of 50 mm and drilled depths of 60 – 70 m. The drilling works in the dam concrete and beneath the upstream support block are performed primarily with rotary core drills with drilled diameters of 46 mm, whereby a majority of the injection wells were constructed as core drillings in order to obtain additional information on the position of the cracks from the analysis of the drilled cores. Additional investigations of the drilled holes with television cameras by the geologist also gained valuable information about the spatial location of the fissures and cracks as well as the existing crack widths.

These drilling works were performed using 2 rotary core drilling rigs of the make DIAMEC 252 (Atlas Copco), whereby the average drilling depths are approx. 25 m. Drilling was carried out according to the pattern of primary and secondary curtains with borehole distances of 3 to 6 m each on the target level of the crack filling.

The drilling programme was enhanced even by the use of 2 additional core drilling rigs for drilling diameters 46 to 76 as well as 100 mm for the execution of control drillings or bored holes for the installation of measuring instruments such as sliding micrometers, extensometers, piezometers or various drainage drillings.

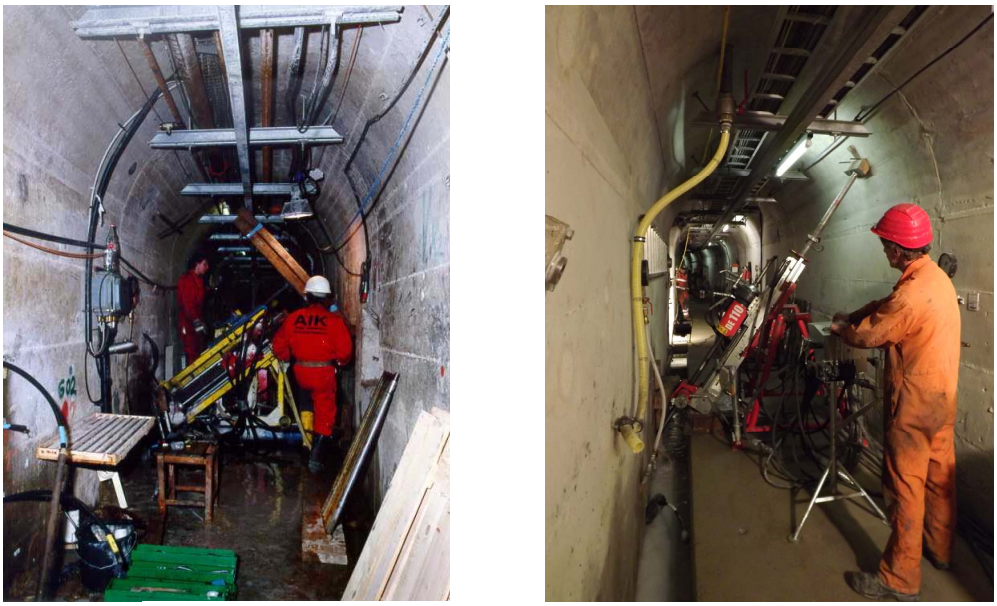


Figure 7: Drilling works inside the gallery (left 1979, right 2013)

Grouting works

The grouting works for cement grouting were carried out through a grouting container with 6 hydrostatically controlled grouting pumps at the dam crest of the left flank. The grouting pipes were laid to the respective grouting points through the inspection gallery along the base area from this central grouting station with a storage silo, a mixing station with automatic dosing of water and solid materials, storage tanks, grouting pumps and grouting data loggers.

Up to a total of 12 resin pumps with data recorders were used for execution of the RODUR resin injections, whereby it was possible to work with a pump from the relevant inspection gallery in every dam block depending on the given backwater level.

© 2016 Deep Foundations Institute

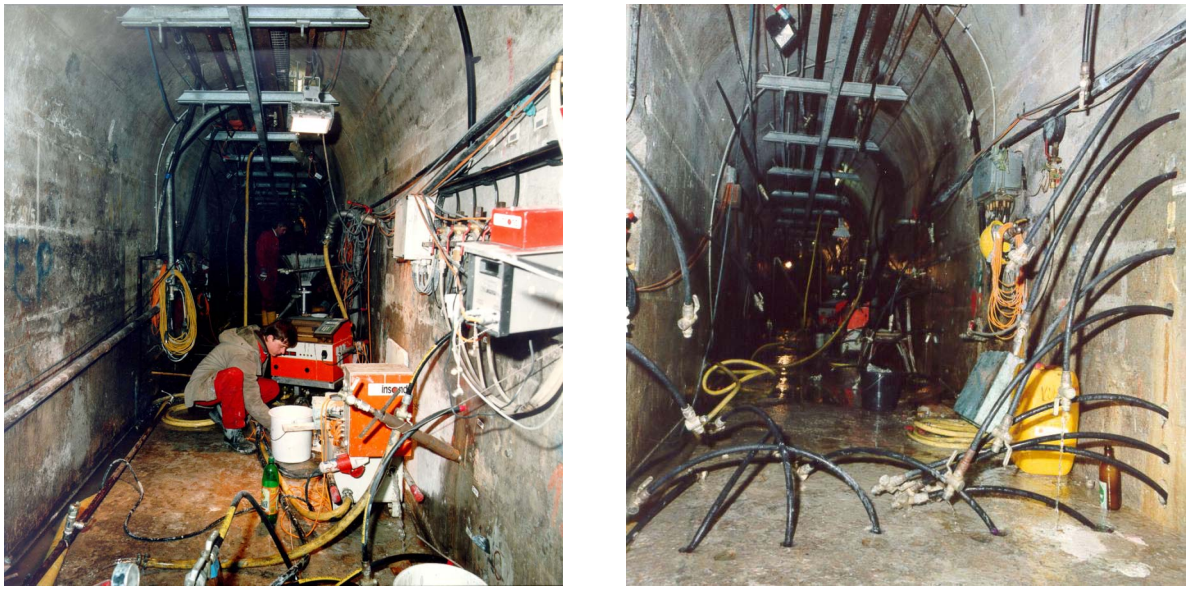


Figure 8: Resin grouting works

Grouting materials:

The grouting materials were adjusted to the defined objective in their characteristics for the individual areas of the treatment.

- In rock they are suspensions of PZ 375 in water with liquefying and cohesive additives.
  The w/c ratio is 1.0, at a settling rate of < 5% and a filtrate water output of less than 10 cm^3.
- In the area beneath the support block, fine cements with Blaine values of more than 8,000 cm^2/g and Marsh times of around 40 s were grouted.
- Exclusively epoxy resins of the make RODUR were used in the dam concrete, as it was only possible to achieve a non-positive injection with a relevant sealing effect against flowing water and high counter pressures using this method.
- Mainly 3 variations of resin – material (RODUR 505, 510 and 520) with different viscosities, depending on the openings of the cracks were used. So with this variations of viscosity, it was possible to grout cracks with width of 0,1 mm up to 2,0 mm and more against the flow of water and the actual water pressure in each actual grouting satage.

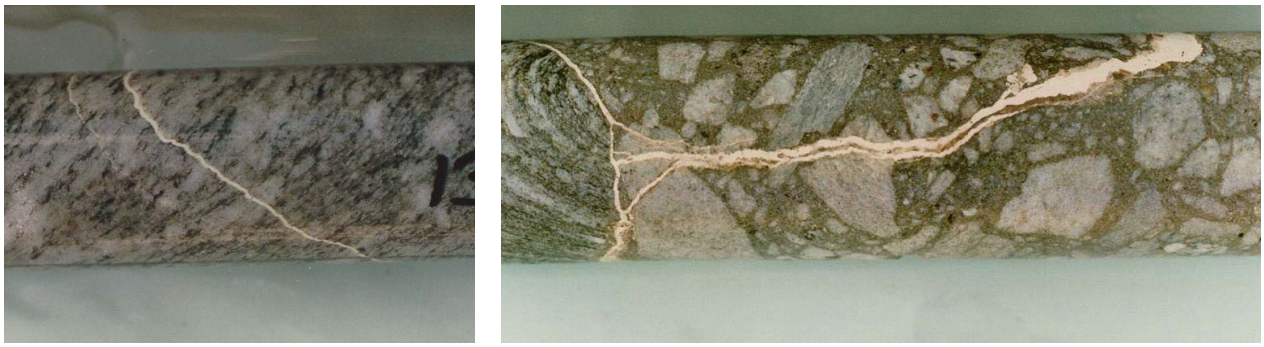


Figure 9: Resin grouting in rock (left), resin grouting in rock and concrete (right)

Recording of grouting data

AIDEK, the automatic grouting data recording was developed further on this construction site and thereby ensures a continuous control of the grouting processes during execution. The grouting pressure at the

© 2016 Deep Foundations Institute

borehole mouth, the grouting quantity and the product of both as energy value (GIN value), as well as two deformation measurements by inclinometers are recorded against the time on a 5 strip chart recorder positioned directly at the pump. Synchronously, all these data are converted and transmitted to the site management, where the individual processes can be monitored online via monitors. At the same time, all data are stored on the construction site and transmitted to the head office of ÖDK in Klagenfurt via a modem line, in order to be available for regular analysis and accompanying control

Installation of the abutment elements

Concreting works for construction of the bearings

Aside from the installation of the mass concrete, the coordination of the workflow for construction of the bearings for the abutment elements on the side of the dam and the support construction presented a logistical challenge. Before the bearings were concreted, 9,000 mortar anchors with a diameter of 30 mm had to be drilled and installed on the roughened surface of the dam concrete. The shuttering, reinforcement and concreting works for the bearings on the dam side were performed in each case 6-9 m above the support construction blocks and had to be adapted to the different dam inclinations and curvatures in each case.

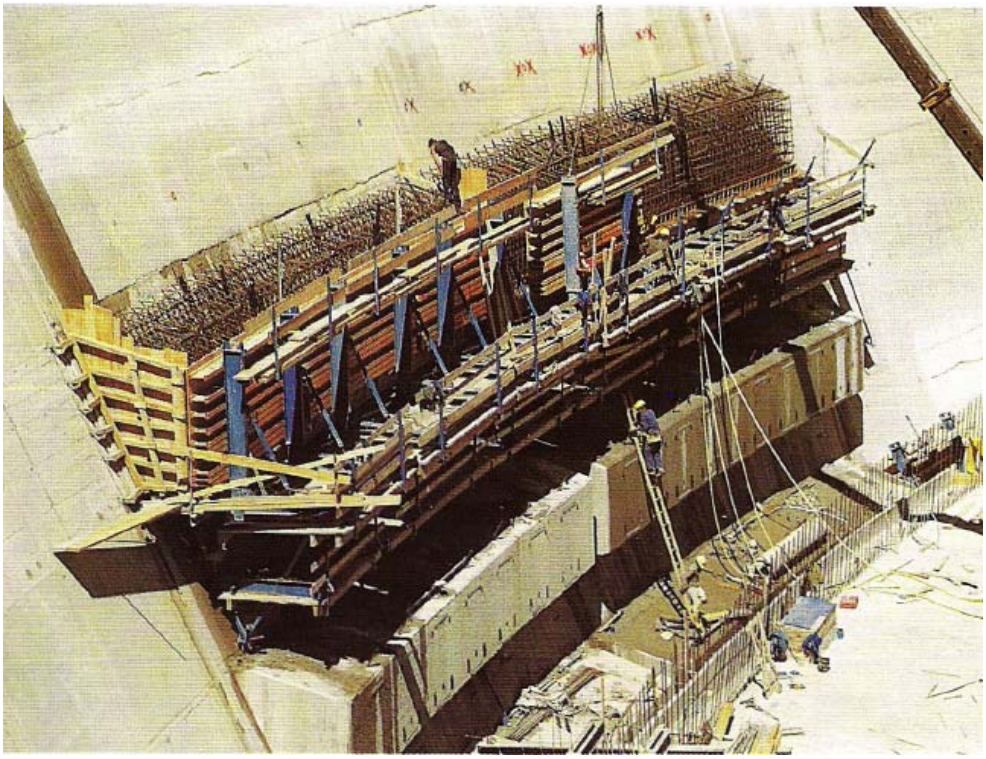


Figure 10: Concreting works for downstream support construstion

Installation of the elastomer bearings

The support of the slim dam with relatively large backwater-related deformations on the stout and rigid support construction must take the different deformation behaviour of the two structures into account. In addition to the safe pick-up of the supporting forces of approximately 10,000 MN (1 million t), these bearings must pick up the occurring relative deformations between the dam and the support construction without constraint. Furthermore, it must be possible to measure the occurring force and it must not be possible to change the backwater-related supporting forces by interventions from outside. Further, the possibility of a replacement or relocation or supplementation must be given.

© 2016 Deep Foundations Institute

All these requirements are fulfilled by appropriately modified elastomer bearings in combination with an adjustable wedge system.

The bearings to be transported and mounted have individual weights of up to 2,000 kg.

For this purpose, a transport structure was mounted on each bearing horizon, which consisted of a vertical rack lift for passenger and material transports as well as horizontally positioned crane rails with a winch.

As a result, approx. 700 bearing structures could be mounted within a period of roughly 6 months.

Monitoring of the structure

With the construction of the downstream support arch dam, it was also necessary to expand the existing monitoring system. When the new facilities were planned, special attention was paid to the fact that it is not just possible to control the behaviour of the support arch dam alone, but also the interaction between both structures including the bedrock.

The following new measuring devices were installed in the course of construction of the support arch dam:

- sounding system – to measure the horizontal shifts in radial and tangential direction
- Invar-wire extensometers – to record the vertical shifts of the support arch dam
- rod extensometers – up to 40 m into the subsoil – to determine the vertical shifts along the base area, as well as to measure the radial movements of the longitudinal joints
- sliding micrometers – to identify possible splitting cracks in the concrete
- clinometers – to determine the radial inclination change of the support arch dam
- block joint width measuring points – were installed at all block joints in the inspection galleries to register the deformations of the block joints
- teleformeters and telepressmeters – measure the extensions and compressions in the support arch dam, as well as the distribution of the supporting forces in the concrete structure
- concrete temperatures – are measured primarily during the setting process by approx.70 concrete thermometers
- piezometers – to measure the fissure and foundation water pressures
- leakage water measuring points - to record any accumulating leakage water in the drain passages
- geodetic measuring devices – to monitor horizontal and vertical shifts at the level of the downstream offset.

Initially, approximately 1,200 measuring points were available for monitoring the Kölnbrein Dam. With the construction of the support arch dam, another 320 measuring points were installed in the structure.

Summary

After a construction period of 5 years, all of the measures described above could be completed successfully and punctually. In 1994, the maximum storage could be reached again for the first time after the rehabilitation. All monitoring measurements of the dam generally complied with the expectations computed in advance. Since then, ÖDK as the power plant operator was able to take up the unrestricted power plant operation of the Kölnbrein Dam.

Regrouting of the grout curtain after 20 years of operation

In the course of the dam's approx. 20 years of operation with annual reservoir rising and lowering phases, only very minor changes showed in relation to the dam's tightness.

© 2016 Deep Foundations Institute

Merely a slight increase in the leakage water losses and thus also a slight increase in the foundation water pressures occurred.

Therefore, the affected central dam blocks were regrouted again at the grout curtain level directly beneath the dam's base area in the years 2011 and 2012.

Thereby, it was identified during the construction of the injection wells (all drilling works were carried out as rotary core drillings with continuous core recovery again) that the fissure injections with RODUR resin carried out at the time showed no impairment whatsoever in their effect. The grouting material encountered still had a flawless frictional connection with the relevant sealing effect. The identified increase in penetrability resulted from the constant stress from the movements during the impoundment phases, which caused new minor cracks to develop over time in addition to the fissure and crack zones that had been sealed off already.

At a cost of approx. 5.0 t of RODUR resin grouting, these leakages could be sealed off again in a short time.

Summary of the grouting methods after 20 years

There was no change in the material development of resin – grouting material (RODUR) over the years. The core drillings from 2011/2012 represented at the resin grouting material was in flawless condition.

Also the grouting method with GIN – criterions is the grouting method what is still in use in most of the actual grouting projects in the middle of Europa.

Other grouting methods, like the philosophy from Prof. Stille (Sweden), is the RTGC – Method (Real Time Grouting Control) is still in use in the hard roc formations of the Scandinavian countries.

The principle of this system is the calculation of the crack – openings from the water pressure tests and the calculation of the theoretical range of the grouting for the largest and the smallest crack, followed by a calculation of a target pumping rate till to the end of the operation. The execution of this system in pressure steered with variable pumping rates.

Grouting specialists from Japan use new methods of grouting by using dynamic effects for the grouting operation.

The complex dynamic grouting method is a technology developed with the objective of providing improved effectiveness and high grouting efficiency in water sealing operations in rock cavern and dams by giving the pulsation of a mixture of two different frequencies. A shortwave pulsation is expected to help the grout penetrate into fine fractures, as the aggregation of the grout particles is broken. On the other hand, a long-wave pulsation has benefit for larger grout penetration than that of the single dynamic grouting method because the pulsation can reach further.

Through the series of experiments, the complex dynamic grouting has proved to be effective for the low conductive rock mass using not only cement grout but also colloidal silica grout compared with both the traditional static grouting method and the dynamic grouting method.

© 2016 Deep Foundations Institute

List of contractors

Construction works for the support construction

Joint venture Kölnbrein Dam (ASK)
- Strabag Austria, Linz
- Hofman &Maculan, Vienna
- Polensky & Zöllner, Salzburg
- Porr AG, Vienna
- Illbau, Spittal
- Mayreder, Keil, List &Co, Graz

Subcontractors, drilling and grouting works

- Insond Ges.m.b.H, Vienna
- Sonderbau Ges.m.b.H, Vienna
- Stuag Bau AG, Himberg

References

1 Clar E., Demmer W.: "Die Geologie der Kraftwerksgruppe Malta" OZE, Year 32, 1979
2 Demmer W.: Geological Documentation for the Dam Foundation", ICOLD 1979
3 Demmer W., Ludescher H.: "Measures Taken to Reduce Uplift and Seepage at the Kölnbrein Dam", ICOLD 1979
4 Lombardi G.: "Kölnbrein Dam, a Special Problem" Water Power, ICOLD ISSUE Vienna, 1991
5 Ludescher H.: "Die Sanierung der Kölnbreinsperre" ÖIAZ 135. (1990)
6 Obernhuber P.: "Remedial Works for the Kölnbrein Dam" Design and Analysis, ICOLD 1991
7 Petter O.: "Betonierung des Stützgewölbes an der Kölnbreinsperre", publications by the Austrian Concrete Association, Sept 1990
8 Widmann R.: "Die Gewölbmauer Kölnbrein" ÖIAZ Year 32 (1979)
9 Grillitsch J., Kohler R., Kogler K., Hechenbichler J.: Dichtungsinjektionen an der Kölnbeinsperre 2011/2012, Wasserwirtschaft 5/2013

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

A REVIEW OF GUIDELINES FOR DEWATERING DESIGN WITHIN THE ABU DHABI MUNICIPALITY

Juan J. Gutierrez, RIZZO Associates, Pittsburgh, PA, USA, juan.gutierrez@rizzoassoc.com
Raghav Ramanathan, RIZZO Associates, Pittsburgh, PA, USA, raghav.ramanathan@rizzoassoc.com
M. Melih Demirkan, RIZZO Associates, Pittsburgh, PA, USA, melih.demirkan@rizzoassoc.com
Jared Deible, RIZZO Associates, Pittsburgh, PA, USA, jared.dieble@rizzoassoc.com
Michael Rosenmeier, RIZZO Associates, Pittsburgh, PA, USA, michael.rosenmeier@rizzoassoc.com
Mazen Elias Adib, Geocomp, Pasadena, CA, USA, madib@geocomp.com

ABSTRACT

Dewatering and groundwater control is required frequently for construction projects and needs to be accounted for and dealt with care during design and construction to complete the project successfully. Dewatering can cause changes in porewater pressure, which changes the effective vertical stress in a soil mass. Dewatering can also cause changes in groundwater chemistry which in turn can cause changes in porewater pressure and also dissolution of soluble minerals in the soil and rock. The Guideline developed in this study addresses the engineering issues, approaches to lessen the potential risks associated with construction dewatering, and presents dewatering practices to support dewatering applications regulated by the Municipality of Abu Dhabi City (ADM) in the United Arab Emirates (UAE). The subsurface condition within the city presents significant geotechnical problems due to presence of shallow subsurface cavities and soluble minerals in the soil and rock. Therefore it is critical to develop standards for dewatering practices to provide information for local dewatering engineers and contractors with special emphasis to control and to eliminate the subsidence/collapse problems related to dewatering projects within the Abu Dhabi city. The ADM is also responsible for approval of permit applications for projects that involve dewatering. In order to minimize the risk related to subsidence/collapse problems due to dewatering, construction companies and dewatering contractors need to be informed about possible mitigative measures that can be employed to reduce risks. The guidelines were developed as part of this study based on detailed analysis of the local subsurface conditions. A set of rules and regulations are developed for geotechnical and hydrogeological investigations, dewatering design and monitoring criteria depending on the hazard zones, proximity of excavation to critical structures, excavation depth. The Guidelines include examples of permit application packages for three types of dewatering projects; one involving shallow dewatering, one involving intermediate depth dewatering, and one involving deep dewatering. These Guidelines are considered as living document and will be updated as required, as additional information is collected and additional local experience is gained.

Keywords: dewatering, groundwater control, construction dewatering, guidelines, Abu Dhabi Municipality

INTRODUCTION

Groundwater is frequently encountered in construction projects and needs to be accounted for and dealt with during design and construction to complete the project successfully. The most common reason for construction dewatering is to lower the water table at a site to a level that will allow for excavation to a foundation level and safe, dry construction of a structure. Construction dewatering must be carefully planned and executed to ensure that the dewatering system provides the desired outcome for the Project and to limit the risk of unwanted impacts to the environment, adjacent structures, and personal safety.

Dewatering can cause changes in porewater pressure, which changes the effective vertical stress in a soil mass and can also cause changes in groundwater chemistry. Changes in porewater pressure and groundwater chemistry can have significant impacts on the soil and the rock. Potential consequences include erosion of soil particles, piping, settlement, collapse of subsurface cavities, and dissolution of soluble materials. These effects can have a significant impact on the area being dewatered and on nearby structures located within the zone of dewatering influence. Proper dewatering design must take into consideration the potential negative impacts and implement mitigation measures to minimize or eliminate these impacts. Several cases of ground failure and extensive settlements in Abu Dhabi have been attributed to nearby uncontrolled dewatering activities.as shown on Fig. 1.

© 2016 Deep Foundations Institute



Figure 1: Ground failure and settlement problems due to uncontrolled dewatering in Abu Dhabi City

In order to minimize future subsidence/collapse problems related to dewatering, construction companies and dewatering contractors need to be informed about the risks associated with dewatering and possible mitigative measures that can be employed to reduce risks. The ADM is responsible for approval of permit applications for construction projects that involve dewatering in Abu Dhabi City and has been regulating the dewatering permits in the City Municipality area. As part of this study, a set of guidelines were developed in association with the Town Planning Sector of ADM; these guidelines present standard dewatering practices to support the ADM with the dewatering permit applications.

The dewatering guidelines address issues and engineering approaches to lessen the potential risks associated with construction dewatering. The guidelines also include a review of engineering practices employed worldwide, guidance for conducting dewatering activities, and permit application packages, which will allow potential risks to be evaluated. These Guidelines were designed to support designers, contractors, consultants, and third party reviewers regarding technical aspects of the dewatering project.

GEOLOGICAL CONDITIONS

Most surfaces in the ADM are covered with a variable amount of made ground as much of the coastal strip has been reclaimed or developed. In the coastal strip, the made ground is often composed of carbonate sand dredged from neighboring lagoons. Further inland, many areas have been extensively landscaped, with large areas of dunes leveled flat or quarried for fill (Farrant et al., 2012a).

© 2016 Deep Foundations Institute

Underlying the made ground is typically Quaternary sediments, including a range of marine and supratidal facies such as coastal spits, bars and beach ridges, lagoonal muds, algal mats, and ooidal tidal deltas of the Abu Dhabi Formation deposited in the coastal areas (Farrant et al., 2012a, b). Landward of the coastal deposits is mostly Aeolian sand, of the Ghayathi Formation and Fluvial sand and gravel deposits of the Hili Formation, which represent Quaternary outwash from the Hajar Mountains to the east. In much of the ADM area, there has been extensive carbonate-evaporitic sabkha development, particularly in the coastal region, but also in some interdune locations.The sabkha is characteristically found in low lying areas, which are prone to flooding at high spring tides. They consist mainly of loose silty fine carbonate sands, with cementation increasing with depth especially in coarser grained deposits.

Stratigraphically, below these sediments are rock layers of the middle to late Miocene age, referred to locally as part of the Fars Group (Farrant et al., 2012a). These units, which are composed of sandstones of the Shuwaihat and Baynunah Formations in the southern areas, dolomitic conglomerates, sandstones and siltstones of the Barzaman Formation in the northern areas, dolomites and limestones of the Dam Formation, and evaporitic mudstone and siltstone of the Gachsaran Formation (Farrant et al., 2012b). These units are mostly present in the eastern portions of the ADM in the higher elevation mesas, where they are overlain by Aeolian sand and made ground.

Descriptions of the sediments and rocks generally encountered at shallow depth in the ADM area are summarized below in Table 1.

Table 1: Generalized Sediments and Rocks Encountered with Depth in the ADM

APPROXIMATE ELEVATION (NADD)	LITHOLOGIC DESCRIPTION*
+50 to +10	Sand and Silty Sands (aeolian and fluvial), Fluvial Gravels, Sandstones and Conglomerates, and Mudstones and Gypsum
+10 to 0	Silty Sand (Fill Material and Sand Dunes), silty fine carbonate Sands (Sabkha Sands), and sandy Silts
0 to -10	Silty fine carbonate Sands (Sabkha Sands), Crystalline Gypsum, Calcareous Mudstone (with gypsum nodules), and Calcarenite
-10 to -20	Crystalline Gypsum and Calcareous Mudstone (with gypsum nodules), and Siltstone
-20 to -30	Calcareous Mudstone (with gypsum nodules) and Crystalline Gypsum layers
-30 to -70	Calcareous Mudstone (with gypsum nodules) with Crystalline Gypsum layers and intercalations of Calcarenite
-70 to -90	Calcareous Mudstone (with gypsum nodules), and Dolomite
-90 to -110	Calcareous Mudstone (with gypsum nodules), Dolomite, and Calcarenite
-110 to -120	Calcareous Mudstone and Calcarenite

Note:
*Compiled from published and unpublished data for the ADM, National Abu Dhabi Datum (NADD)

During various geotechnical investigations in the ADM area, voids have been identified by tool drops in fractured calcareous mudstones/siltstones with gypsum inclusions, calcarenite, and sands, between or above massive gypsum layers, and in what is referred to as the weathered/fractured top of rock. Intensive dewatering has also been interpreted as having the potential for either increasing the size of pre-existing voids in subsurface, or creating them by removal of fine particles that is commonly found in the voids (Farrant et al., 2012b). Irrigation of forestry, gardens, and farmland areas inland is exacerbating the situation by increasing local groundwater head, which, coupled with construction related dewatering within the urban area, are changing the hydraulic gradient creating one of the key triggers for sinkhole development. Tool drops and loss of water circulation have also been described in the calcarenite layers encountered at depth in the ADM. Water loss is also commonly associated with highly permeable soils

© 2016 Deep Foundations Institute

(silty sand with gravel/shells, and gravels). Settlement has been observed in certain areas after water level drops due to intense dewatering during construction activities, especially in those areas in which more permeable soils are described (gravel, sandy gravel, and sand bars). Dissolution of salt crystals in fill material is also a factor contributing to settlement in areas of the ADM.

HYDROGEOLOGICAL CONDITIONS

Very little published information is available with regard to the hydrogeology of the ADM's 89 municipal zones and the "core" ADM zones, wherein significant geotechnical issues have been encountered (e.g., Mohammed Bin Zayed City, Khalifa City A, and Khalifa City B). Hydrogeological data for the ADM is effectively limited to the static water levels reported for geotechnical boreholes completed in conjunction with major infrastructure and building projects, although some long-term groundwater level monitoring data, groundwater quality data, and minimal information related to aquifer or aquitard hydraulic properties are available.

Interpreted potentiometric surfaces for the ADM indicates that groundwater elevations in the western and north-central ADM are relatively flat, ranging from approximately -15 up to about 10 meters mean seal level (m msl) (Figures 2 and 3). Groundwater elevations generally increase in an easterly direction, again mimicking ground surface elevations. In this eastern area, groundwater elevation data suggest that localized areas with steep hydraulic gradients are common. As shown on Fig. 2, the general direction of groundwater flow is from the east and southeast toward the west and northwest, toward the Arabian Gulf coastline. The shallow groundwater system is recharged primarily from the ground surface via precipitation, irrigation, and stormwater runoff detention ponds. Some groundwater flows laterally into the ADM region from upgradient areas (to the west and northwest) and vertically upward from deeper bedrock aquifers

A portion of the groundwater flow is discharged laterally into the Arabian Gulf. However, a large portion of groundwater discharge can be attributed to evaporation to the atmosphere, especially in sabkhas and low elevation areas, where the water table surface is very close to the ground elevation. It is important to note that top of bedrock elevations within the ADM typically range from about -20 to 100 m msl. Generally, the lowest top of bedrock surface elevations are located in western areas, but rise inland, effectively mimicking surface elevation changes. In turn, total thickness of the unconsolidated sediments and fill material (made ground) within the ADM (i.e., surficial aquifer) ranges from 0 m to almost 24 m, although in most areas, the total thickness of unconsolidated sediments is typically 10 m or less. Overall, this suggests that the thickness of the unconfined aquifer system within the ADM ranges from 0 to 24 m (Fig. 3). Greater saturated thicknesses generally occur in the south-central ADM, within the aforementioned core geotechnical hazard area. Within extreme eastern ADM regions, the upper Quaternary aquifer (unconsolidated overburden) is generally dry.

According to Sanford and Wood (2001), a water budget was estimated for United Arab Emirates (UAE) coastal sabkhas, based on data collected at two locations along the UAE coast. Their calculations showed annual precipitation at Abu Dhabi City is about 90 millimeters (mm)/year. Average annual recharge to the sabkha groundwater is about 45 mm/year (approximately 50 percent of precipitation. Annual pan evaporation is about 2,900 mm/year. Average evaporation from the sabkha surface is 88 mm/year (about 3 percent of pan evaporation).

© 2016 Deep Foundations Institute

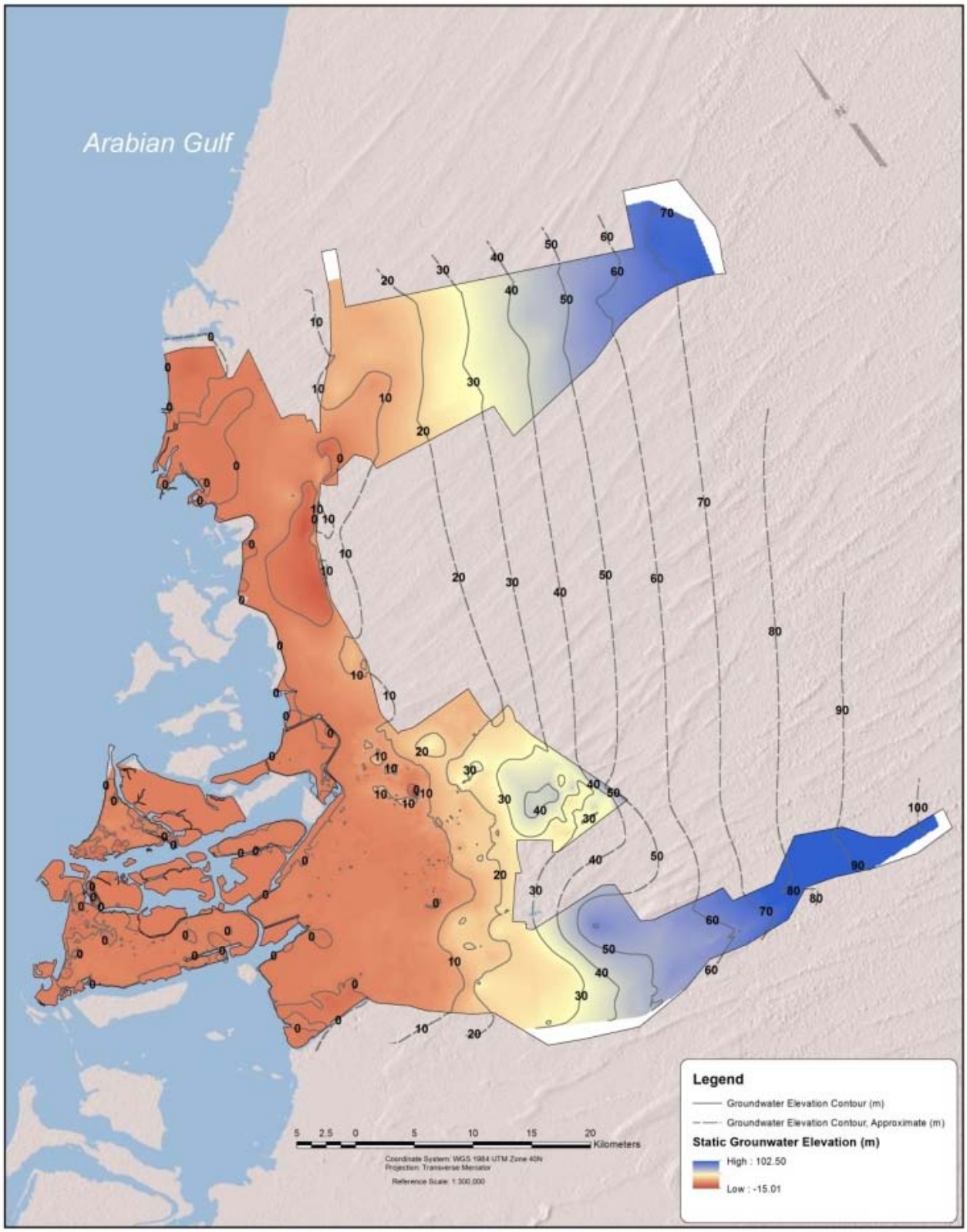


Figure 2: Groundwater Elevations in Abu Dhabi Municipality

© 2016 Deep Foundations Institute

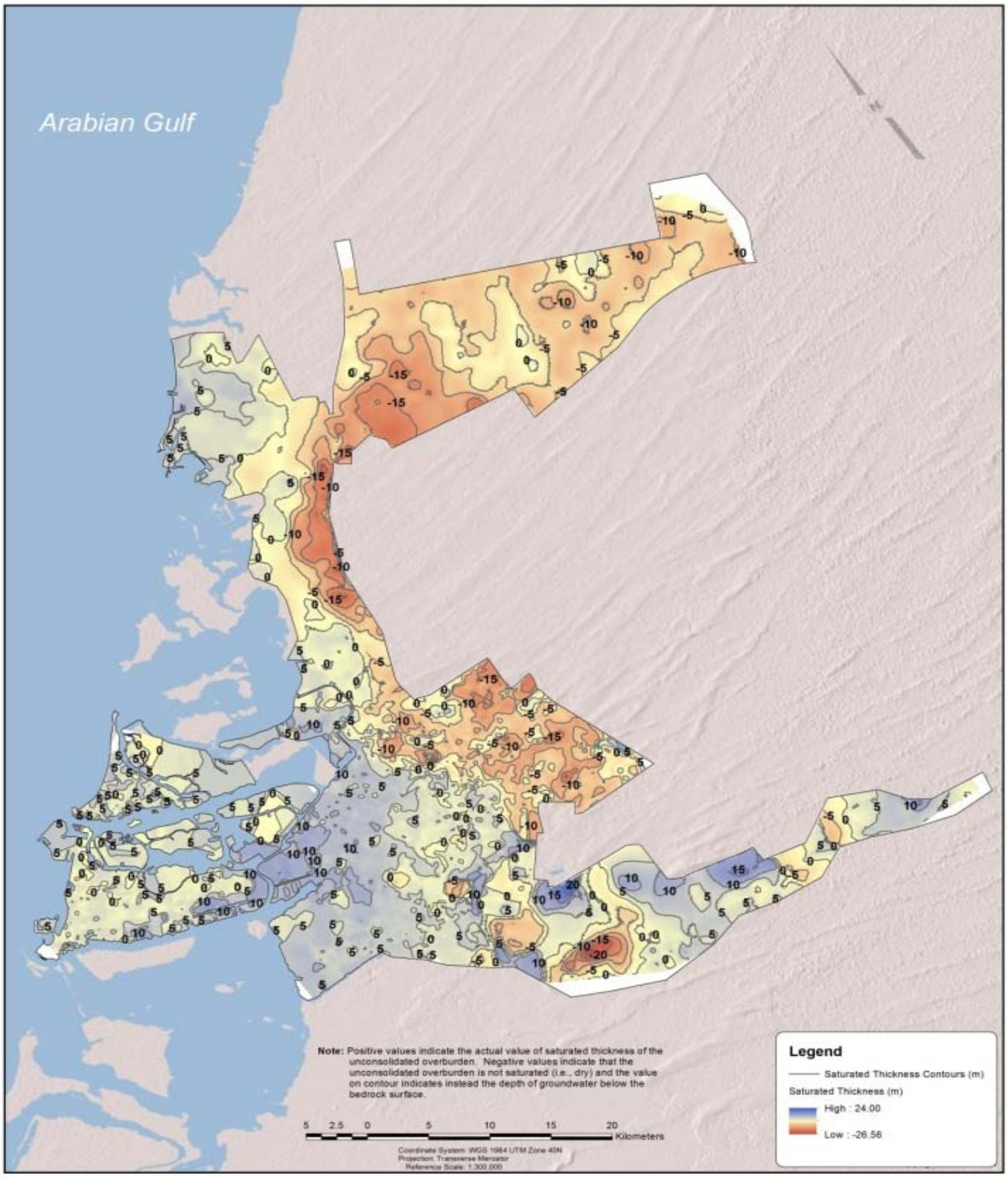


Figure 3: Saturated Thickness of Unconsolidated Sediments and Fill

Approximately 1,000 different hydraulic conductivity field tests performed in the Abu Dhabi area were collected from engineering reports; this data shows that sands and gravels in the overburden, and sandstones, calcarenites, and conglomerates in the bedrock formations generally have the highest hydraulic conductivity values. Lower values were generally found in the finer-grained sediments, gypsum, claystone, and mudstones. However, there is

© 2016 Deep Foundations Institute

considerable variation and overlap within and between lithologic types. In the ADM, natural hydraulic gradients within the surficial aquifer are generally low. However, when large-scale construction dewatering programs are implemented, water levels can be drawn down considerably and thus create relatively steep hydraulic gradients, increased groundwater seepage velocities, and increased hydraulic uplift pressures. Changed hydraulic conditions at a construction site associated with groundwater dewatering can create forces that result in geotechnical problems.

CONSTRUCTION DEWATERING

There are three general methods of groundwater control for construction: (i) Open pumping, (ii) Predrainage, (iii) Cutoffs, and (iv) Exclusion. The most common construction dewatering methods are open pumping and predrainage; however, it is not uncommon to see dewatering systems that combine two or more of these methods to effectively dewater a site.

Open Pumping: The use of drains, sumps, and open pumping is typically the least expensive dewatering option when site conditions make it feasible. In general, this method consists of establishing a series of French drain trenches graded to a sump established at the low point of the excavation, allowing seepage from the side slopes and floor of the excavation to collect in the sump and be pumped out. For staged or deep excavations, intermediate sumps may be established at various elevations as the excavation progresses. French drain trenches are typically arranged around the perimeter of the excavation to minimize the distance across the work area water must travel to be removed by the sump. In Abu Dhabi City, the use of open pumping is not recommended, especially in urban areas and near roads or infrastructure. Their use must be on limited and as-needed basis in Abu Dhabi City and only in strict accordance with permit conditions as issued by ADM where no adverse effects will occur. Specifically, open pumping should be limited to dewatering operations involving cutoff structures, where the entire area of the bottom of the excavation is in unweathered portions of sedimentary rock formations. In such cases, the role of open pumping shall be limited to extracting the relatively small amounts of water seeping through the rock bottom. In areas of weathered rock, open pumping shall not be performed as this would lead to erosion of unconsolidated materials through the mechanism of piping.

When site conditions are not conducive to the use of open pumping and surface drainage, attempting to dewater the site with these methods is very difficult and possibly catastrophic. Surface pumping has caused several cases of damage in Abu Dhabi City, largely due to the removal of fine materials because of improper intake design and lack of filter around the intake. At sites where open pumping appears to be feasible, it is advisable to maintain flexibility in the dewatering scheme to allow for additional dewatering measures to be enacted if changed or unexpected conditions are encountered during construction. Table 2 lists favorable conditions for open pumping dewatering systems.

Table 2: Favorable Conditions for Open Pumping Dewatering (Powers et al, 2007)

CONDITION	DISCUSSION
Soil Characteristics:	
Dense, well graded granular soil, especially with cementation or cohesive binder	Soils are typically low in hydraulic conductivity, resulting in lower seepage volumes. Well graded material is naturally resistant to piping erosion.
Sedimentary rocks	Hard strata are naturally resistant to piping erosion and base heave. As such, they are amenable to open pumping dewatering methods.
Hydrology Characteristics	
Low to moderate dewatering head	These characteristics all contribute to expected low groundwater seepage volumes, allowing effective use of sumps and drainage ditches (French drains)
Remote source of recharge (adjacent bodies of water, etc.)	
Low to moderate hydraulic conductivity	
Minor storage depletion	
Excavation Method	

© 2016 Deep Foundations Institute

CONDITION	DISCUSSION
Dragline, clamshell, and excavating equipment not operated from inside the excavation.	These methods do not require traffic in the excavation, so the soft and wet material being excavated does not impede progress.
Excavation Slopes and Support	
Relatively flat slopes	Flat slopes selected appropriately for the soil type are inherently more stable than steeper slopes and more resistant to lateral seepage.
Excavation shoring structures – steel sheets, slurry walls, other cutoff structures	Excavation shoring helps to reduce or eliminate lateral seepage, increasing excavation stability and aiding open pumping systems.
Miscellaneous Other Items	
Open site with limited adjacent structures nearby	Limited slope slides may be acceptable if no adverse effects to adjacent structures will occur.
Large excavations that are slowly advanced	Very large excavations that take a long time to perform may allow drains and sumps to effectively drain the excavation.
Light foundation loads	If foundation loads on the soils at the base of the excavation are expected, disturbance of the surface by boils and seepage may be acceptable if the foundation design accounts for them.

Maintaining stable slopes when using open pumping to dewater an excavation can be a challenge. As excavation proceeds, careful observation of the slopes and drainage features must be performed to ensure that slope instability, excessive erosion, piping, boils, or heave are not occurring. In the event that these conditions cannot be mitigated, the excavation must be allowed to flood and additional dewatering measures installed. Other construction considerations include employment of a gravel bedding layer to provide drainage for water coming through the material at the base of an excavation to the sump and allow for foundation work to be performed in the dry. This bedding layer may be underlain with a non-woven geotextile to increase filtration, if necessary. The use of a bedding layer may not be effective if the base material is cohesionless, particularly in the case of fine sand. Piping of fine particles from the subgrade materials must be prevented in order to preserve the support of the foundation materials. If gravel bedding is found to be ineffective at managing the seepage volume through the floor of the excavation, additional measures, such as wellpoints or deeper drains, and sumps are warranted.

Predrainage: Predrainage may be performed via wellpoint systems or dewatering wells. Wellpoint systems are the most common and often the least expensive predrainage option for construction dewatering. They are generally well suited to situations where the water table needs to be lowered to 4.5 to 6 m or less, or when the excavation is suited to the use of multiple stages of wellpoints every 4.5 to 6 m of water table elevation. The effective depth of wellpoint systems is limited due to the limitations of vacuum lift. It is important to note that a wellpoint system is only efficient to the depth of the shallowest installed wellpoint. The implication of this observation is that the depth reached by individual wellpoints should be uniform across a wellpoint system when installed in a uniform geologic layering. The unit cost for individual wellpoints is typically low and multiple wellpoints can be operated with a single pump and header system. Because of the low unit cost of wellpoints, they are well suited for conditions requiring very close spacing, such as stratified conditions and excavations, where an impermeable layer is near to the base. A wellpoint dewatering system consists of three basic items, namely the wellpoint pump, the header pipe and connections, and the individual wellpoints.

Dewatering wells generally have larger diameter than wellpoints, typically 15 cm up to greater than a meter, and consist of a solid casing provided with a screened interval in the target zone. Dewatering wells in smaller diameters are possible with the use of recently developed very small pumping equipment. Similar to wellpoint systems, a filter zone is constructed around the screened interval to prevent the pumping of fine particles from the surrounding soil. A well pump is placed in each well casing near the screened interval, allowing for very high production from an individual well without the limitations of suction pressure. Dewatering wells are typically spaced widely and installed at a depth well below the excavation bottom. They are most effective in situations where a relatively homogenous and transmissive aquifer exists, allowing for maximized production from each well. The unit cost for dewatering wells is high, due to the depth and size of the holes required for installation and the cost of the individual pump installed in each casing. Power costs for high production well systems can also be

© 2016 Deep Foundations Institute

significant. Table 3 lists conditions suited for selection of predrainage methods.

Cutoffs: Cutoffs are required for excavations where the groundwater inflow is expected to be too high to be safely or efficiently managed and needed to provide redundancy in a dewatering system in the event of a loss of dewatering capacity. Groundwater cutoffs are physical barriers to groundwater flow, and may consist of walls made of steel or plastic sheet piles; concrete, soil-bentonite, cement-bentonite, or soil-cement-bentonite walls constructed by excavating a trench and backfilling with impermeable material; jet grouted walls; deep soil mixed walls; and cutoff walls installed in rock with secant piles or overlapping panels

A cutoff wall should be designed by following well-established design standards and manuals such as Eurocode 7 (EN 1997, 2007), Strom and Ebeling, (2001), CIRIA C580 (2003), and Xanthakos (1994). Input, assumptions, the design code followed, analysis procedure and output should be properly documented. Seasonal changes and any fluctuations of groundwater levels or adverse changes due to dewatering should be considered in the design of the walls.

Table 3: Conditions for Selection of Predrainage Methods (Powers et al, 2007)

CONDITION	WELLPOINT SYSTEMS	SUCTION WELLS	DEEP WELLS	EJECTOR SYSTEMS	HORIZONTAL DRAINS
Soil					
Silt and clayey sands	Good	Poor	Poor to Fair	Good	Good [1]
Clean Sands and Gravels	Good	Good	Good	Poor	Good
Stratified Soils	Good	Poor	Poor to fair	Good [2]	Good
Clay or rock at subgrade	Fair to Good	Poor	Poor	Fair to Good	Good [3]
Hydrology					
High hydraulic conductivity	Good	Good	Good	Poor	Good
Low hydraulic conductivity	Good	Poor	Poor to Fair	Good	Good
Proximate recharge	Good	Poor	Poor	Poor to Good	Good
Remote recharge	Good	Good	Good	Good	Good
Schedule					
Excavation					
Shallow (<6m below water table)	OK	OK	OK	OK	OK
Deep (>6m below water table)	Multiple stages required	Multiple stages required	OK	OK	Special equipment required
Cramped/limited access	Interference expected	Interference expected	OK	OK	May be OK
Typical System Characteristics					
Normal Spacing	1.5 – 3 m	6 – 12 m	>15 m	3 – 6 m	Continuous horizontal line
Range of Capacity – Per UNIT	0.4 – 95 lpm	190 – 2270 lpm	0.4 – 11360 lpm	0.4 – 150 lpm	Dependent on pipe size and pump capacity
Range of Capacity – Total SYSTEM	Low – 18930 lpm	7570 – 94635 lpm	Low – 227125 lpm	Low – 3785 lpm	Low – 7570 lpm
Efficiency of proper designed system	Good	Good	Fair	Poor	Good
Unit Cost Per Individual UNIT	Low to moderate	Moderate	High	Low to moderate	Low to moderate

Notes:

[1] If backfilled with sand or gravel

[2] If keyed into clay or rock

[3] Double pipe ejectors with wellscreen full length

Cutoff walls may require a tie-back system for structural stability. Typical methods used to analyze a cutoff wall

© 2016 Deep Foundations Institute

tie-back system usually include beam on rigid support method, beam on inelastic supports analysis, and linear and nonlinear soil-structure finite element analysis. In general, the continuous beam on rigid support method, with staged excavation analysis, will provide reasonable estimates of anchor forces and wall bending moments. This type of construction sequencing analysis is often followed by a beam on elastic or inelastic foundation (Winkler) construction sequencing analysis to verify anchor forces (Strom and Ebeling, 2001). The behavior of multi-anchored systems may be strongly influenced by factors such as the sequence of excavation and installation of anchors, fluctuations in the water table, and the nonlinear stress-strain behavior of soils. In order to accurately obtain the magnitudes of stresses and deformations in the structure and the surrounding soil, it would be necessary to perform soil-structure interaction analyses that model the construction and operation stages of the system.

Exlcusion: In this method compressed air, earth pressure shields, is used to exclude groundwater from the excavation. This method is primarily applicable for tunnel excavation.

PRE-CONSTRUCTION PLANNING AND DEWATERING DESIGN

Prior to beginning the construction, it is critical to evaluate subsurface conditions, including groundwater levels and conditions, and thoroughly plan the dewatering program that will be required to complete the Project successfully. The location, geometry, type of excavation, type of soil to be excavated, and the duration of dewatering are important considerations in the selection and design of a dewatering system. For most granular soils, such as sandy soils, the groundwater table during construction should be maintained at least 1 m below the slopes and the bottom of an excavation in order to ensure “dry” working conditions. It may need to be maintained at lower depths for silts (1.5 to 3 m below subgrade) to prevent water pumping to the surface or causing wet and spongy conditions in the excavation. Where the bottom of an excavation is underlain by a less pervious stratum, like soft carbonate rock interbedded with gypsum that is underlain by a pervious formation under artesian pressure, the upward pressure or seepage may cause wet conditions or even heave at the bottom of the excavation without a proper dewatering system, even though the excavated pit and slopes may be dry.

Uncontrolled or improperly controlled groundwater can cause piping, heave, slope instability, settlement, and other impacts, which can impact the Project being constructed and other nearby structures. Therefore, subsurface construction should not be done without appropriate control of the groundwater and (subsurface) hydrostatic pressure. The construction schedule must be determined and evaluated before proceeding with the design of a dewatering system. The time required to achieve the required level of the dewatering depends on the soil type, size, and volume of the excavation, method(s) of dewatering and the design of the dewatering system. Dewatering in fine grained soils may require a significantly longer time than dewatering in coarser grained soils. The number, size, spacing, and depth of wells or other dewatering measures, and the rate at which water must be removed to achieve the required groundwater drawdown or pressure relief, must be determined and adjusted to meet the construction scheduling. In some cases, such as construction of a utility line, it may be necessary to perform dewatering in zones along the alignment of the work zone in stages, with the dewatered zone moving along with and ahead of the excavation or work zone. In other cases, it may be possible to dewater the entire alignment or larger portions of the work zone at once.

Based on borehole logging, monitoring of piezometers and wells, and geophysical measurements, potential permeable zones are identified. Depending upon the Project requirements, additional field tests, such as borehole seepage and pumping tests may be performed to evaluate the quantity of the water likely to be encountered during dewatering operations. In the analysis of any dewatering system, the source of seepage must be determined, and the boundaries and seepage flow characteristics of geologic and soil formations at and adjacent to the site must be generalized into a form that can be analyzed. In some cases, the dewatering system and soil, and groundwater flow conditions can be generalized into rather simple configurations. For example, the source of seepage can be modeled as a line or circle; the aquifer as a homogeneous, isotropic formation of uniform thickness; and the dewatering system as one or two parallel lines or circles of wells or wellpoints.

The scope of the geotechnical investigation and laboratory testing for a project should include evaluation of potential zones of cavity collapse. The formation of cavities involves natural processes of erosion or gradual

© 2016 Deep Foundations Institute

removal of slightly soluble bedrock (such as limestone, gypsum, or rock salt) by percolating water, the collapse of a cave roof, or a lowering of the water table. The geotechnical investigation should identify the existence of the collapsible or soluble materials. If such material exists, which is likely in the ADM, the dewatering system and excavation should be designed to prevent the collapse of potential cavities, which could be influenced by the dewatering.

There is a potential risk that dewatering may result in undesirable consequences, such as settlement when the water table is lowered, it increases the effective stress in the soil mass. Compressible soils and loose granular soils have the potential to compress and consolidate when the water table is lowered and the effective stress is increased. If dewatering is carried out properly so that no loss of material occurs due to open pumping and there is no pumping of fine particles from the wells, then settlement is not likely to occur in competent soils. If weak soils are present in the vicinity, such as normally consolidated clays, silts, peats, or loose granular deposits, then settlement or consolidation is a possibility during dewatering.

Tables 4 and 5 provides the recommended scope of dewatering analysis and design based on the level of hazard of the Project, the proximity of sensitive structures to the dewatering site, the depth of excavation, and the type of dewatering and excavation techniques selected

Table 4: Dewatering Design Scope

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	A	Analytical Solution
	B	Flow Net
	C	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required

© 2016 Deep Foundations Institute

Table 5: Hazard Zones and Respective Dewatering Design Types

HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	PUMPING CAPACITY ANALYSIS[1]	SETTLEMENT ANALYSIS	THIRD PARTY REVIEW
C	1,2	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	c	I	X
	3	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	b	I	
B	1,2	i	Shallow	a	I	
		i, ii, iii	Medium	b	I	X
			Deep	c	II	X
	3	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	b	II	
A	1,2	i	Shallow	c	I	X
		ii, iii		c	I	X
		i, ii, iii	Medium	c	II	X
			Deep	c	II	X
	3	i	Shallow	a	I	
		ii, iii		a	I	
		i	Medium	b	I	X
		ii, iii		b	I	X
		i	Deep	c	II	X
		ii, iii		c	II	X

Notes:

[1] Simplified hand calculations are recommended when numerical models are developed. Hand calculations can provide useful checks of more advanced models.

REVIEW AND APPROVAL PROCESS

Incorrect implementation of the dewatering process causes serious problems in the entire UAE, since engineering properties of soil and rocks, specifically in ADM region are sensitive to ground water changes. Rapid drawdown or frequent fluctuation of ground water level or loss of fine soil due to dewatering causes serious settlement and even structural damage of the nearby structures. These issues are amplified if natural ground has soluble zones, such as halite/salt layers or if manmade fill is not constructed properly. Therefore, ADM scrutinizes each dewatering permit application. Adequate time should be reserved for dewatering permit application and approval in the construction schedule.

All application documents shall be reviewed and approved by an independent Third Party Engineering Consulting Company, which is qualified by the ADM for this purpose. Design drawings, calculations, method statement shall be signed and stamped the by the third party reviewer before submitting to ADM.
ADM has a state-of-the-art internet-based Dewatering Permit Application System as a part of Construction Permitting System. The dewatering applications are divided into two main categories: building projects of any size and infrastructure projects, such as roads, pipelines, trenches, and any kind of excavation that will require discharge of water (which changes the water ground water regime). The dewatering application review and approval process is managed by a reviewer who is an ADM Engineer. All the submitted documents will be reviewed by the ADM reviewer prior to approval of the permit. Members of Geotechnical Engineering Unit, Division of Health, Safety, and Environment and Town Planning Division review and approve parts of the application. Installation of instrumentation to monitor the discharge of the water may be requested during the review process. Figure 4 shows the various steps involved in obtaining a dewatering permit.

© 2016 Deep Foundations Institute



Figure 4: Procedure to acquire a dewatering permit from the ADM as per the dewatering guidelines

Once the approval is issued, the permit can be printed from any customer services counter in ADM centers. The construction can only start after obtaining the permit. The consultant and the contractor are required to inform ADM on planned start and termination date of dewatering. It is also required to notify the ADM before the actual start and the termination dates. Issuance of a permit shall not remove the responsibility of the consultant and contractor on the dewatering system. The consultant and contractor shall be responsible for any damage to existing structures and services in the vicinity of the excavation. ADM inspectors might perform a field inspection of the dewatering works in any stage of the construction. Although it is not required, ADM inspectors may give an advance notice before these inspections.

These inspections may focus on the compliance of constructed dewatering system with approved design and procedures. Type and dimension of shoring system, number of wellpoints, filter design, and depths of the wells are among the subject of the inspection. Discharge points, street crossings of dewatering pipes, and location of other equipment might be examined. Based on the number of working dewatering wells, fine particle content in the sediment tanks may be inspected. ADM inspectors may review the monitoring records. Water samples may be taken if needed.

CONCLUSIONS

The Guidelines are prepared as part of the study presents standard dewatering practices to support dewatering applications in the ADM. These Guidelines provide information for local dewatering engineers and contractors. A special emphasis is given to control and to eliminate the future subsidence/collapse problems related to dewatering projects in the ADM area.

These Guidelines also include examples of permit application packages for three types of dewatering projects; one involving shallow dewatering, one involving intermediate depth dewatering, and one involving deep dewatering. These Guidelines are a living document and will be updated as required, as additional information is collected and additional experience is gained. These Guidelines are designed to support designers, contractors, consultants, and third-party reviewers regarding technical aspects of the dewatering project according to the three main phases involved in a dewatering project: (i) Field investigation, (ii) dewatering design, and (iii) monitoring program. The requirements addressed in these Guidelines are scalable to the magnitude of the dewatering project.

© 2016 Deep Foundations Institute

ACKNOWLEDGMENTS

The authors appreciate the support of the Town Planning Sector, and the Spatial Data Division of the ADM and the cooperation offered by all agencies that provided assistance and contributed data towards the completion of this study.

REFERENCES

Anderson, Mary P., Woessner, William W., 2004 "Applied Groundwater Modeling: Simulation of Flow and Advective Transport," Academic Press.

Brook, M., 2005, "Water Resources of Abu Dhabi Emirate U.A.E.," Environment Agency – Abu Dhabi, p.167, 2005.

Brook, M., H. Al Houqani, T. Darawsha, M. Al Alawneh, and S. Achary, 2006, "Groundwater Resources: Development & Management in the Emirate of Abu Dhabi, United Arab Emirates," 2006

Caltrans Construction Division, "Field Guide to Construction Site Dewatering," Caltrans Construction Division, October 2001.

CIRIA C580, 2003, "Embeded Retaining Walls – Guidance for Economic Design," Construction Industry Research and Information Association, 2003.

Das, Braja M., "Principles of Foundation Engineering – Third Edition," PWS Publishing Company, 1995.

Das, Braja M., "Principles of Geotechnical Engineering – Fifth Edition," Brooks/Cole, 2002.

Dawoud, M., 2008, "Water Resources in the Abu Dhabi Emirate, United Arab Emirates," Environment Agency – Abu Dhabi, p.141, 2008.

EN 1997, 2007, Eurocode 7 - Geotechnical Design, European Committee for Standardization (CEN), 2007

Farrant, A.R, Ellison, R.A., Merritt, J.W., Merritt, J.E., Newell, A.J., Lee, J.R., Price, S.J., Thomas, R.J., and Leslie, A. 2012a, "Geology of the Abu Dhabi 1:100,000 map sheet,
United Arab Emirates," (Keyworth, Nottingham: British Geological Survey).

Farrant, A.R , Ellison, R.A. Leslie, A., Finlayson, A., Thomas, R.J., Lee, J.R., Burke, H.F., Price, S.J., Merritt, J. and Merritt, J.W. 20 12b, "Geology of the Al Wathba 1: 100,000 map sheet, United Arab Emirates," (Keyworth, Nottingham: British Geological Survey).

Joint Departments of the Army, the Air Force, and the Navy, USA (1983), Technical Manual TM 5-818-5/AFM 88-5, Chap 6/NAVFAC P-418, Dewatering and Groundwater Control.

Powers, J. Patrick, Corwin, Arthur B., Schmall, Paul C., and Kaeck, Walter E., "Construction Dewatering and Groundwater Control – New Methods and Applications," Third Edition, John Wiley & Sons, Inc., 2007.

Rizk, Z.S., and A.S. Alsharhan, 2003, "Water Resources in the United Arab Emirates," in Water Resources Perspectives: Evaluation, Management and Policy (A.S. Alsharhan and W.W. Wood, eds.), Elsevier Science, Amsterdam, pp. 245-264, 2003.

Strom, R.W. and R.M. Ebeling, (2001), State of Practice in the Design of Tall, Stiff and Flexible Tieback Retaining Walls. ERDC/ITL TR-01-1. Department of the Army, Army Corps of Engineers, Engineer Research and Development Center, Vicksburg, MS.

© 2016 Deep Foundations Institute

DRILLING AND GROUTING FOR THE REHABILITATION OF CONTAMINATED AREAS: AN ITALIAN CASE HISTORY

Raffaella Granata, Trevi S.p.A, Italy, +393479727588, rgranata@trevispa.com
Daniele Vanni, Trevi S.p.A, Italy, +3905473191, dvanni@trevispa.com

ABSTRACT

This paper presents the drilling and grouting program performed at two abandoned landfills located into former calcarenite rock quarries in Manfredonia, Italy. Tests performed after the abandonment of the landfills detected leachate infiltration into the groundwater, which required the implementation of a remedial program. In order to seal the rock fissures and create an impervious basin at the base of the two landfills, sub-horizontal boreholes were drilled and grouted into the rock under the waste deposits. At the same time, sub-vertical boreholes were drilled and grouted along the perimeter, to laterally confine the landfills. Finally, the landfills were properly capped with an HDPE membrane, geogrid, geotextile and topsoil. The methodology implemented to create a watertight barrier under the two landfills was innovative and cost-effective. Moreover, the drilling and grouting minimized the environmental impact of the remedial work, since the waste remained in their original location and were not affected by the remedial process.

Keywords: Directional Drilling, Grouting, Cutoff, Rehabilitation

INTRODUCTION

The two landfills, "Pariti-1 RSU" and "Conte di Troia", are located approximately 6 km (4 miles) southwest of Manfredonia, a small town close to Foggia, Italy (Fig. 1). These landfills are located into two rock quarries, formerly used for the extraction of calcarenite (i.e., a weak calcareous sandstone, in tuff facies), and were abandoned in the early 90s. The two landfills cover a total area of about 45,000 m^2 (11.2 acres), and mainly include urban solid waste with a depth of up to 25 m (82 ft). The main characteristics of the two waste landfills are listed in Table 1.



Fig. 1 Location of the two waste landfills at the Manfredonia site

© 2016 Deep Foundations Institute

Table 1. Main characteristics of the two waste landfills

	Landfill Conte di Troia		Landfill Pariti 1 RSU	
Landfill Area	25,454 m^2	6.291 ac	19,600 m^2	4.893 ac
Landfill Perimeter	780 m	2560 ft	660 m	2165 ft
Waste deposit Total Volume	140,000 m^3	183,000 cy	260,000 m^3	340,000 cy
Waste deposit Max Thickness	16 m	52 ft	25 m	82 ft

In the years following the abandonment of these landfills, the analysis of groundwater in the Manfredonia area showed evidence of significant leachate infiltration through the rock at the base of the landfills. After a series of events, the European Union issued an infringement procedure against Italy. In November 2008, the EU Court of Justice issued a conviction for failure to complete the works aimed to restore safe environmental condition, and imposed severe sanctions. The sentence was suspended following the commitment by the Italian Government to complete the remedial work by 2010. Following a series of legal issues and bureaucratic procedures, the remedial work started in early 2010, and was successfully completed in January 2011.

SITE CONDITION AND PRELIMINARY SITE INVESTIGATION

The area where the two landfills are located is characterized by a series of small ravines originated by erosion phenomena. Surface water is absent, but this karst area houses a remarkable diversified ecosystem.

Several geotechnical and hydraulic investigations were performed between 2003 and 2009. The investigations detected large steps in the rock at the base of the landfills, which date back to the time when the area was a rock quarry. The investigations also ascertained that the outcropping calcarenite in the area is about 25 m to 40 m (82 to 130 ft) thick (Fig. 2). The thickness of the calcarenite underlying the Conte di Troia landfill is in the order of tens of meters. The Pariti landfill is located at the opposite side of the ravine, at a lower elevation. Here the calcarenite overlying the limestone formation is in some places only a few meters thick.

The calcarenite has a variable degree of cementation, from low to high, with a UCS of about 5-15 MPa (700-2200 psi). The measured primary permeability of the rock (i.e. due to the interstitial porosity) is mainly in the order of 10^{-8} cm/s. Nevertheless, this calcarenite is extensively fissured, due to tectonic and geo-mechanical stresses. Fractures and voids have variable distribution, and locally determine a huge increase in the secondary hydraulic conductivity. Photos of the two lithological facies are shown in Fig. 3.

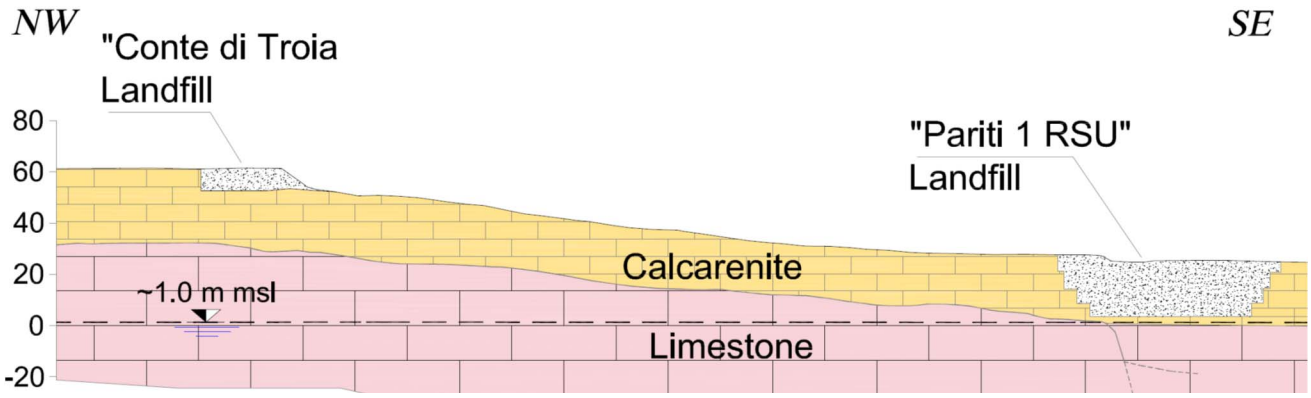


Fig. 2 Scheme of the site stratigraphy

© 2016 Deep Foundations Institute



Fig. 3 Photos of the two lithological facies: Calcarenite (left) and Limestone (right)

The white-pinkish limestone underlying the calcarenite has a relatively high mechanical strength, with UCS in the range 20-35 MPa (2900-5000 psi). The limestone is well layered and intensely fractured, with fractures often enlarged by dissolution karst phenomena. The limestone permeability is locally high to very high, and only due to cracking and karst.

The absence of a measurable level of leachate at the site confirms the presence of a system of connected fractures, resulting in the complete drainage of both rainwater and leachate.

THE DESIGN

At first, the temporary transfer of the waste material was considered. The landfill locations were to be remediated with basal and lateral impervious linings (in accordance with the current landfill regulations), prior the relocation of the waste. However, considering the environmental risks and the long time required to move the waste, it was decided to design and build an in-situ containment.

The main technical challenge was to build the impervious barriers underneath the landfills. Grouting of the rock stratum beneath the landfills was selected. The treatment included injections of cement grouts and silicate mixtures, able to seal the cracks, gaps and discontinuities affecting the rock permeability, for a minimum thickness of 3 m (10 ft). This in-situ barrier had to decrease the secondary permeability of the rock (due to fissures) substantially. Locally, also the primary permeability, due to the presence of weathered calcarenite, had to be reduced. The designers established a target hydraulic conductivity of 5 UL (Unit Lugeon) of the treated rock.

The grouted layer was designed to follow the geometry of the landfill's base, at least 2.5-3.0 m (8-10 ft) below the waste-rock interface. The bottom plug was then to be tied to the lateral diaphragm wall, to create a continuous containment basin. However, in order to grout the rock, a huge quantity of vertical boreholes should have had to be drilled through the waste deposit: up to 25,000 holes for approximately 500,000 m of drilling (1,640,000 ft). The drilling and grouting through vertical holes would have taken a long time to construct, and it was not free from environmental hazards.

At the end, the Engineer (Sviluppo Italia Engineering in cooperation with Studio Sintesi) selected a very innovative design, requiring an impervious bottom slab created by grouting the calcarenite rock through sub horizontal boreholes. Due to the required drilling length (150 m, 490 ft), accurate position (max allowed deviation 300 mm, 1 ft), and curvilinear path, these boreholes had to be installed by directional drillings. Likewise, the lateral confinement had to be installed by grouting sub-vertical boreholes (Fig. 4).

© 2016 Deep Foundations Institute

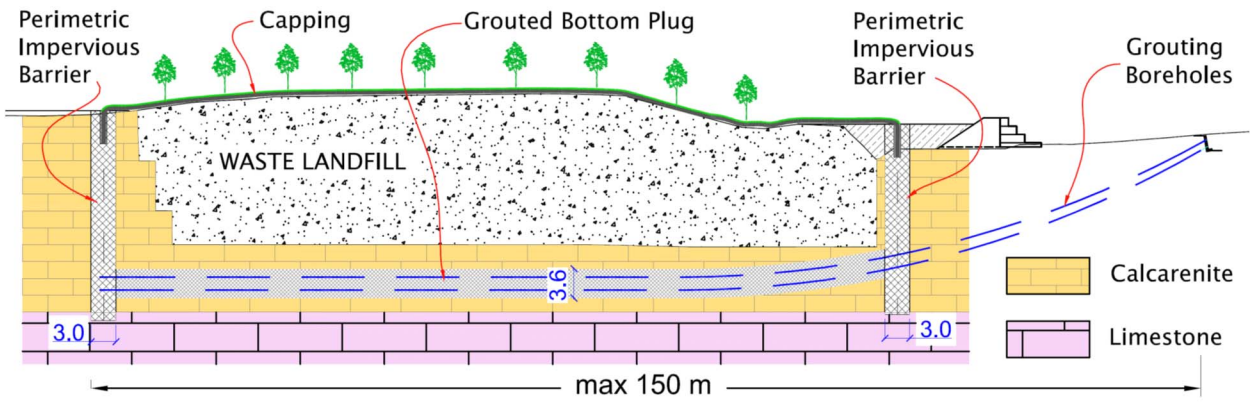


Fig. 4 General section of the confinement system: Impervious Barriers (grout curtains) at the bottom and along the perimeter, connected to a traditional capping at the top

In 2010, the public tender was awarded to a joint venture, with Trevi as specialist subcontractor. The work started immediately after the award. The tight schedule required considerable resources working 24 hours a day, 7 days a week.

THE CONSTRUCTION

Additional Site Investigation

In order to better define the design and technical procedures, it was necessary to perform a new investigation campaign, both inside and along the perimeter of the two landfills. The site investigation included core drillings, video inspections and permeability tests (Fig. 5). At the same time, a series of field tests was carried out, in order to perfect technological and operational details and verify that the project goals could be achieved.

The investigations defined the geometry of the landfill. About 20-30% of the drillings detected interconnected macro-cracks, varying in size from a few centimeters to decimeters (inches to feet).



Fig. 5 Video-inspection of Cored Boreholes, before grouting

Lugeon water tests were performed to determine the hydraulic conductivity of the rock before the treatment. By definition, the Lugeon Unit (UL) corresponds to the absorption of 1 liter of water, in 1 minute, in 1 meter long section of the hole, under a pressure of 1 MPa. In the standard Lugeon test, measurements are performed at three different pressure steps, with sequence A-B-C-B-A (with A<B<C). Since the calcarenite and limestone formations were characterized by secondary-type permeability only, the measured values depended on the number and type of cracks intercepted during the test. Typically, the

© 2016 Deep Foundations Institute

measured permeability value was in the order of hundreds of UL. In many cases, the magnitude of cracking was so severe (and the water-take so high) that it was impossible to measure a constant flow rate and determine the UL value. This scenario was confirmed by the video-inspections of the investigation holes.

Drilling and Grouting

A new and innovative technology was implemented for the waterproofing underneath the landfills. This technology is based the newly introduced Horizontal Directional Drilling (HDD) in the geo-engineering field. The HDD technique allows the drilling of curvilinear sub-horizontal boreholes onto long distance, with minimal deviations from the designed path.



Fig. 6 Left: Drilling the Bottom Grout Curtain. Right: Panoramic view of drilling activities at the Conte di Troia landfill.

The HDD boreholes were started from one side of the landfills and advanced in the calcarenite under the waste deposits, after the installation of the impervious grout barriers along the perimeter (Fig. 6). As anticipated, the project design required curvilinear boreholes, up to 150 m (492 ft) long, to be installed with a deviation of less than 300 mm (1 ft) from the theoretical position.

Since the depth of the horizontal drilling (up to 30-35 m, 100-115 ft) hindered the signal transmission, the traditional "surface magnetic guidance method" was abandoned. The adopted system allowed the tracking of the drilling tool position by creating an artificial magnetic field within the hole. Power and direction of the generated signal were detected by the instrumentation within the drill string. The data were transmitted in real time to the computer of the steering engineer, who compared the surveyed coordinates with the theoretical position of the hole. In case of deviation, action was taken using special eccentric tools and the hole was brought back within the designed path.

The grout holes were drilled 1.8 m (5.9 ft) spaced, on two levels. The bottom curtains included a total of 565 holes, equal to 58,380 m (191,535 ft) of drilling. The average length was 103 m (338 ft), the maximum length 150 m (492 ft). Drilling and grouting were carried out in primary, secondary and tertiary holes, by split method. Both cementitious grouts and durable mineral-silicate mixes were utilized, to seal both large fractures and thin fissures.

Locally, when drilling the primary holes, the downstage method was utilized, due to the voids and fractures endangering the borehole stability. In these cases, the drilling was stopped, and low mobility grout was injected in the unstable section. After hardening of the grout, the drilling was resumed. After

© 2016 Deep Foundations Institute

plugging the main voids and fractures, the grouting was performed utilizing the MPSP system, a technique suitable for the treatment of degraded rock (MPSP: Multi-Packer-Sleeved-Pipe, Bruce et Al., 1988). The MPSP grouting method is described in Fig. 7.

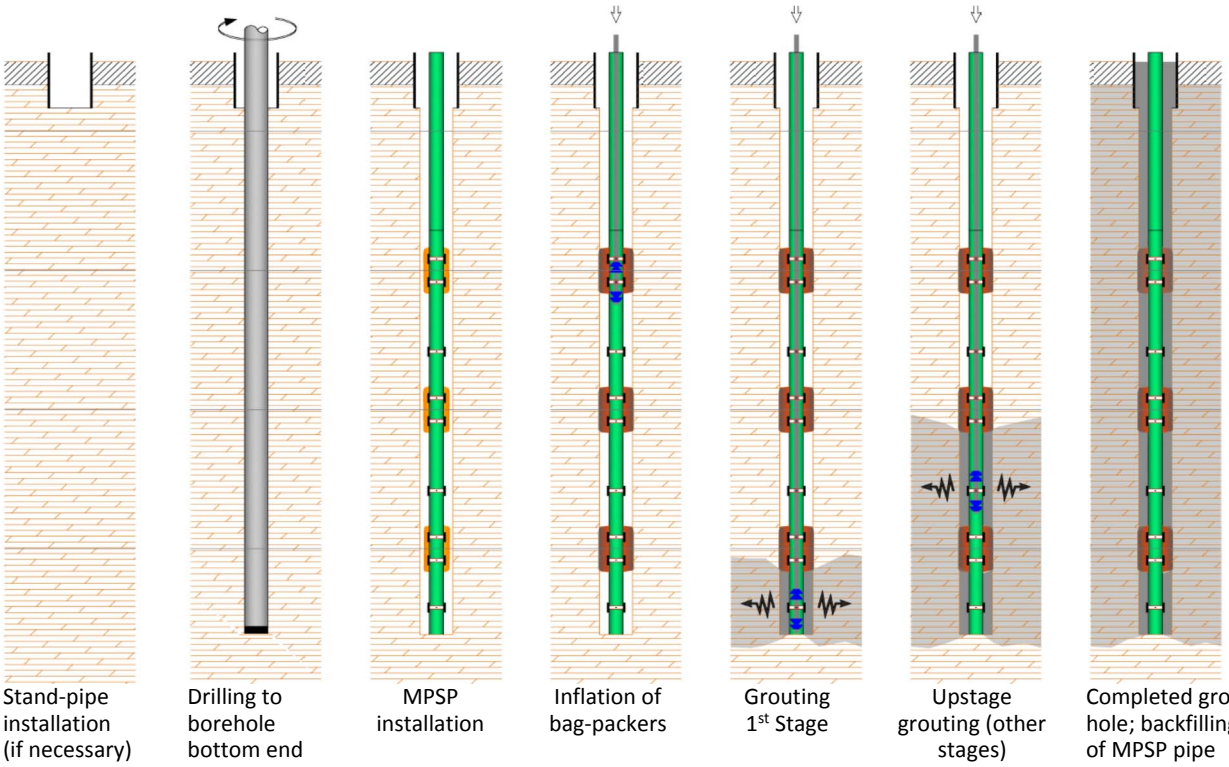


Fig. 7 MPSP (Multi-Packer Sleeved Pipe) Grouting Method

The drilled boreholes were equipped with 2-in-dia. grouting pipes fitted with external packers, which subdivided the hole in sections that could be grouted independently. The injections were carried in sections ≤ 7 m long (23 ft) utilizing double packers lowered into the grout pipes (Fig. 8).



Fig. 8 Grouting work: Bottom Curtain (left) and Lateral Curtain (right)

© 2016 Deep Foundations Institute



Fig. 9 Grouting Unit and Grouting Parameter Control



Fig. 10 Batching-and–Mixing Plants

The grout lines were connected to the injection units, where tens of injectors were individually managed by a computerized system, for the control, monitoring and recording of the grouting data (pressure, volume, flow rate, Fig. 9). The batching/mixing units for the preparation of the grout mixes were installed close to the injection units (Fig. 10).

The installation of the lateral barriers was performed in a similar way. Three rows of grout holes were drilled, 1.3 m spaced (4¼ ft). The holes on the upstream and downstream rows were inclined 15° from vertical, in opposite directions, and were grouted with cement-base mixes. The holes of the central row were vertical, and were grouted as tertiary holes with durable mineral-silicate mixes.

The grout takes in the different sections of each hole were carefully recorded, allowing a comprehensive evaluation of the data and results obtained. Figure 11 is an example of a map illustrating the grout takes in a certain treated area. The grout takes are expressed as percentage of the theoretical volume related to each grout hole, utilizing a color scale.

© 2016 Deep Foundations Institute

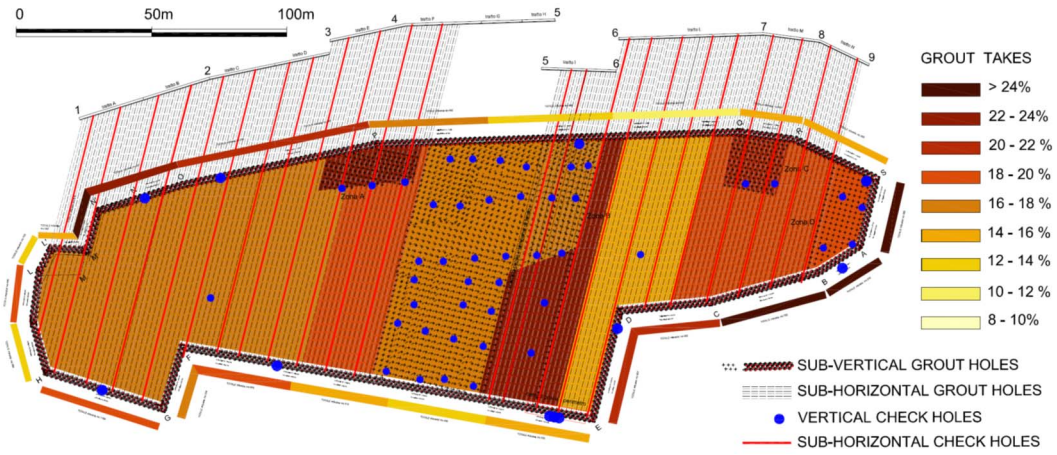


Fig. 11 Grout takes (Lateral and Bottom Grout Curtains) and location of the Check Holes at Pariti 1 RSU Landfill.

Verification Tests

After the completion of the grouting activity, site investigations were performed to check the quality of the barriers, both around and below the landfills. The site investigations included vertical and sub-horizontal coring, video-inspections and water tests in the cored boreholes. Coring, inspection and Lugeon water tests were performed at regular spacing, in locations selected by the Engineer (Fig. 11). The photos of the core samples and the video inspections well documented the quality of the final result (Fig. 12 and Fig. 13). The water tests provided results between 2 and 5 UL, which were in line with the design values.



Fig. 12 Cored Samples of Grouted Rock. To be noted the fractures filled of grout (black is cement grout, white is silica grout)



Fig. 13 Video-inspection of Cored Boreholes, after grouting

© 2016 Deep Foundations Institute

During testing, care had to be taken not to damage the mixture injected to seal the rock joints, fractures, cracks and micro cracks. Under high test water pressures, the injected water could have opened the fissures, with a consequent detachment of the mixture from the fissure walls, and the washing away of the filler material, either natural clay or mixture injected.

Cross-hole tests were performed to control the treatment performed, as integration of a linear tomographic exploration (seismic and electrical). Purpose of the survey was to determine the reduction of the geo-structural discontinuities after treatment, which indirectly means a reduction of secondary permeability. In particular, the tests were performed to detect variations in the speed of compression waves (P) along a vertical section. With reference to the geological context modified by injections, the comparative analysis led to the conclusion that the tests investigated a discrete homogeneous ground (Vanni et Al., 2011).

Capping

The landfill surfaces were reshaped, creating slopes adequate to the capping construction and rainwater drainage. Finally, the landfills were properly capped by HDPE, geogrid, geotextile and topsoil (Fig. 14). Upon the completion of grouting work, at the top of each boundary grout curtain a trench was excavated 4 m deep (13 ft), by trench-cutter. The capping was embedded in the trench, cast with durable concrete, thus obtaining the complete encapsulation of the waste material.

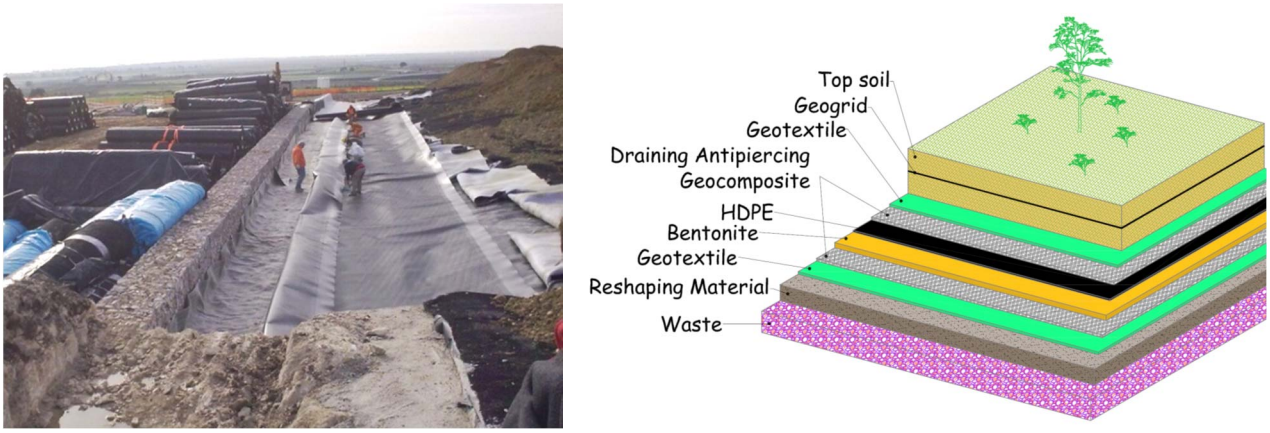


Fig. 14 Left: Capping works. Right: Detail of Capping Strata

CONCLUSIONS

HDD technology enabled the installation of a network of sub-horizontal grout holes under the landfill, up to 150 m long (492 ft), curved and directed with precision of 0.3 m (1 ft). The treated rock layer below and around the landfills had a thickness of ≥ 3 m (10 ft) and permeability of less than 5 UL.

The techniques chosen and special injection mixtures used for the creation of the lateral and bottom barriers made the encapsulation of the landfills possible, without removing or perforating the waste deposits. The remedial work started in early 2010 and was successfully completed in January 2011. The strategy implemented to create a watertight barrier under the two landfills is innovative and cost-effective. Moreover, it has no environmental impact, since the waste remains in the original situ and is not affected by the remedial work.

© 2016 Deep Foundations Institute

REFERENCES

Bruce, D.A., and Gallavresi, F., 1988. The MPSP System: a new method of grouting difficult rock formations. Proc. ASCE Convention Nashville, TN. May 9-13, and published in ASCE Geotechnical Special Publication No. 14, ‘‘Geotechnical Aspects of Karst Terrains’’, pp 97–114.

Vanni ,D., Granata, R., Specchio, V., Croce, M., 2011, “Utilizzo di tecnologie innovative di perforazione ed iniezione per la messa in sicurezza di aree contaminate: Il caso delle discariche RSU di Manfredonia”, Memorie del XXIV Convegno Nazionale di Geotecnica - “Innovazione tecnologica nell’ingegneria geotecnica”, Napoli 22-24 giugno 2011, pp.799-806.

© 2016 Deep Foundations Institute

LABORATORY MODELING OF INTERNAL EROSION IN DAM FOUNDATIONS

John D. Rice, Utah State University, Logan, Utah, USA, 435-797-8611, john.rice@usu.edu
Kenneth E. Henn, USACE, Louisville, Kentucky, USA, kenneth.e.henn@usace.army.mil
David B. Paul, USACE, Washington, D.C., USA, david.b.paul@usace.army.mil
Brian H. Greene, Gannett Fleming, Inc., Pittsburg, Pennsylvania, USA, bgreene@GFNET.com

ABSTRACT

Several laboratory modeling programs have been conducted to investigate the mechanisms associated with internal erosion of dam embankments and foundations. The investigation consists of developing laboratory devices to model three separate mechanisms associated with process: 1) contact erosion along bedrock joints, and 2) piping into an open bedrock defect. The contact erosion device subjects the soil above a bedrock joint to high velocity flows in the joint. The erosion into a bedrock defect device examines a number of static and transient hydraulic conditions to study the mechanisms associated with this type of erosion. Numerous tests have been conducted using both devices. The results of the modeling will be discussed along with a discussion of how this information could be used in future dam design and risk assessment studies.

Keywords: Internal erosion, piping, dam foundations, cutoffs

INTRODUCTION

A series of laboratory modeling programs were performed to assess the seepage and internal erosion behavior of several man-made and natural structures within dams and their foundations. Each model was performed on a "bench scale" and depicted an isolated location where internal erosion was likely to occur. The models were subjected to hydraulic boundary conditions to simulate the in-situ conditions so that the processes of internal erosion could be observed. Two laboratory devices were developed to model conditions depicting 1) contact erosion along a bedrock joint in contact with a dam embankment, and 2) backward erosion (piping) into an open bedrock defect (joint, solution void, etc.).

Embankment dams often have rock foundations containing defects which may detrimentally impact future performance. Further, there are existing foundations which contain rock defects that may already be detrimentally impacting the dams; however, due to the inherent aspects of the mechanism, the signs of distress may not have propagated to the surface and thus not observed during inspections. A better understanding of the mechanisms which are influenced by, and impose their influence upon, rock defects will enhance the ability of dam safety professionals to analyze the risks associated with internal erosion due to rock defects, increase our ability to monitor and successfully initiate earlier detection, and also increase our ability to design remediation methods to contend with rock defect failure modes.

EROSION INTO BEDROCK DEFECTS

Hydraulic loading conditions leading to erosion of soil particles into pre-existing and untreated bedrock defects is a function of both the erodibility of the soil and the flow of water in the vicinity of the defect. If the hydraulic gradient is high enough, then soil particles can be conveyed into the rock defect to be transmitted down gradient, or temporarily stored within the system of discontinuities. In either case, when this occurs under an embankment dam, there is ample potential for backward erosion or scour to occur which, left unchecked, can ultimately lead to failure of the structure.

Exacerbating the condition of erosion into a defect or series of defects within rock is the process of cyclic loading. As with most flood mitigation reservoirs, pools are drawn down to maintain storage and then rise

© 2016 Deep Foundations Institute

as storms occur and the reservoirs retain water as intended. This natural operation of a dam and reservoir can however result in a surging effect that results in moving core material from a dam into bedrock defects at the foundation level where it is difficult to detect. When simulating conditions that may occur under normal reservoir operations, it is important to attempt to model conditions of cyclic hydraulic loading in the laboratory.

TESTING PROGRAM

Purpose and Goals

To investigate the mechanisms of the processes discussed above, two laboratory modeling devices were designed and constructed in Utah State University's Earth Structures Laboratory. The modeling devices were designed to provide qualitative and quantitative assessments of the mechanisms and parameters that affect the mode of internal erosion development and the rate at which internal erosion initiates and develops.

Joint Erosion Testing Device

A schematic illustration showing side, top and end views of the Joint Erosion Testing Device (JETD) is presented in Fig. 1. The JETD consists of a 14 in. (36 cm) wide by 45 in. (114 cm) long by 16 in. (41 cm) deep reinforced steel box enclosing a jointed concrete block in contact with an area for placing the soil to be tested. Screw jacks embedded in the concrete block are used to set the aperture of the joint. An air bladder on top of the soil area imposes pressure on the soil to simulate overburden pressure on the soil. A differential hydraulic head is imposed across the device and the water is directed through the joint in the concrete block to induce contact erosion in the soil adjacent to the joint. A photograph of the JETD is presented in Fig. 2. Details of the testing procedure are presented in the full report (Zaleski and Rice, 2016)

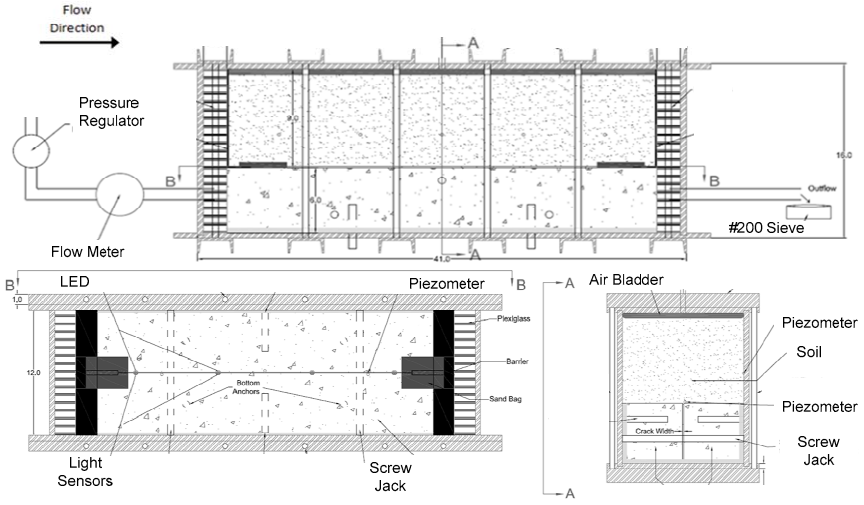


Fig 1. Side, top and end views of the Joint Erosion Testing Device

The JETD is instrumented to monitor the hydraulic pressure and flow and to provide indications of the initiation and progression of erosion. The hydraulic pressure is monitored with pressure sensors upstream and downstream of the device as well as three locations along the joint. The flow is monitored with an in-line ultrasonic flowmeter placed upstream of the JETD. The erosion at the top of the joint is monitored using LED lights and light sensors. At four locations in the soil at the top of the joint, LEDs are placed

© 2016 Deep Foundations Institute

on one side of the joint while two light sensors are placed on the other at distances of ¼ and ½ inches (0.6 and 1.3 cm) from the crack. As the soil erodes from above the joint the light sensors will incrementally begin to detect the light, indicating erosion has taken place. All of the instrumentation data is automatically collected during the tests using a data logger and can be viewed in real time during the test on a dedicated computer screen.

Tests are performed by gradually increasing the differential head across the JETD and collecting data. The initiation of erosion is noted as decreases in differential pressure across the device or increases in flow. The progression of erosion is detected using the LED/light sensor configuration. At the conclusion of the test, the soil is carefully exhumed and the amount and nature of erosion along the joint recorded.

Nine models were prepared and tested using a variety of soils. Key parameters of the soils tested are presented in the first three columns of Table 1. Seven tests were performed using fabricated soils consisting of fine sand (#100 sieve) and different proportions of kaolinite and bentonite clay. The other two tests were performed on samples of soil from the core of Teton Dam in Idaho and the embankment of East Branch Dam in Pennsylvania.

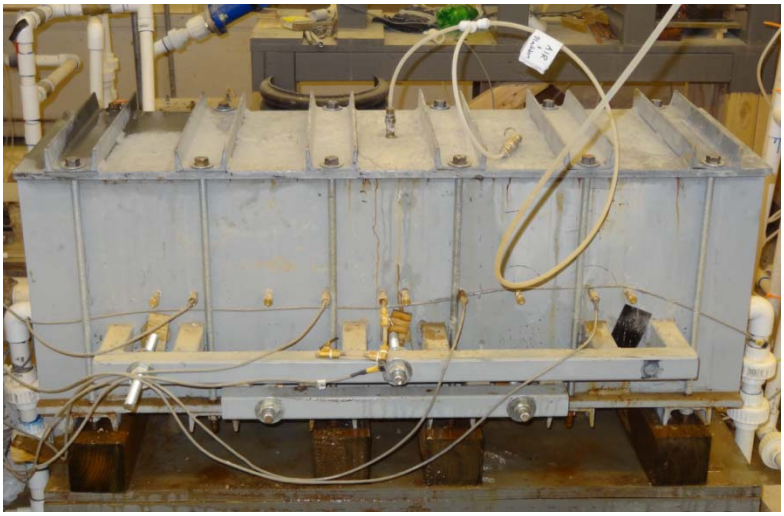


Fig 2. Photograph of Joint Erosion Testing Device

The results of the tests performed on sample "Sand/Clay (75/25)(a)" (see Table 1) are presented on Figs. 3 and 4. Figure 3 presents the readings of the flow meter (right vertical axis) and the light sensor readings (left vertical axis) against the time of the test (horizontal axis). The sharp increases in flow at 7 and 12 days are the result of increasing the differential head across the JETD. As can be seen in the plots of the light sensor readings, it took between 14 and 22 days for sufficient erosion to occur to expose the light sensors.

However, in Fig. 4, a plot of the piezometer readings versus time, it can be inferred from the data that the initiation of erosion occurred much earlier; specifically, immediately after the increase in differential head at 7 days. Note the plots of piezometers 3 and 4; after the increase in differential head at 7 days the measured heads of these piezometers moved closer to each other over the next few days. This decrease in head can be interpreted to be the result of decreased flow resistance between these piezometers that is the result of an erosion channel. After the initial erosion, the distance between the lines became constant as the eroded channel reached equilibrium with the imposed hydralulic conditions. When the differential head is again increased 12 days into the test, the erosion begins again and progresses over the next few days. Then, at about 20 days into the test, the data indicate a rapid expansion of the erosion area occurs:

© 2016 Deep Foundations Institute

the difference between piezometers 3 and 4 becomes nil, and piezometer 2 decreases rapidly to converge with the readings of piezometers 2 and 4. This indicates that the erosion that was occurring between piezometers 3 and 4 spread to the area between piezometers 2 and 3. These deductions correlate well with the observations in Fig. 3 where the spreading of the erosion indicated by the light sensors matched a similar pattern of spreading erosion.

Table 1. Properties of soils tested in the JETD

Test Material Used	Percent #100 Sand	Percent Kaol.	Percent Bent.	Median Grain Size, D_{50} (mm)	Average Grain Size, D_H (mm)	Total Unit Weight (pcf)	Liquid Limit/ Plasticity Index	Water Content (%)	Relative Compaction (%)
Sand/Clay (80/20)	80	20	-	0.125	0.0141	143	NP	13	89
Sand/Clay (75/25)(a)	75	25	-	0.125	0.0115	148	NP	12	92
Sand/Clay (75/25)(b)	75	25	-	0.125	0.0115	146	NP	14	91
Sand/Clay (70/30)	70	30	-	0.125	0.00977	142	NP	15	88
Sand/Bentonite (87.5/12.5)(a)	87.5	-	12.5	0.125	0.00236	137	NP	12	87
Sand/Bentonite (87.5/12.5)(b)	87.5	-	12.5	0.125	0.00236	133	NP	15	85
Sand/Bentonite (85/15)	85	-	15	0.125	0.00197	145	NP	16	95
Teton Core	-	-	-	0.033	0.033	117	NP	14	91
East Branch Core	-	-	-	0.1	0.1	138	4 / 22	9.5	95

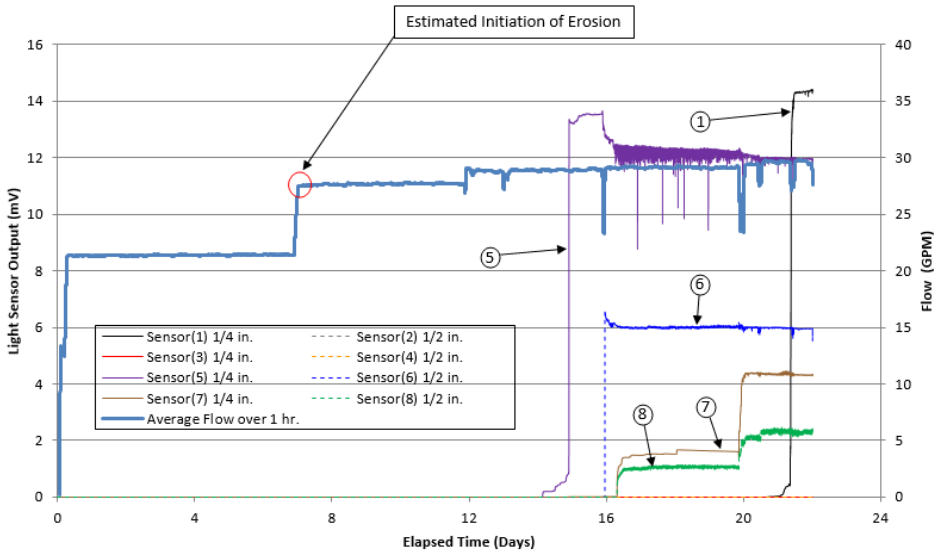


Fig 3. Plot of light sensor readings and flow volume versus time for test: Sand/Clay (75/25)(a)

© 2016 Deep Foundations Institute

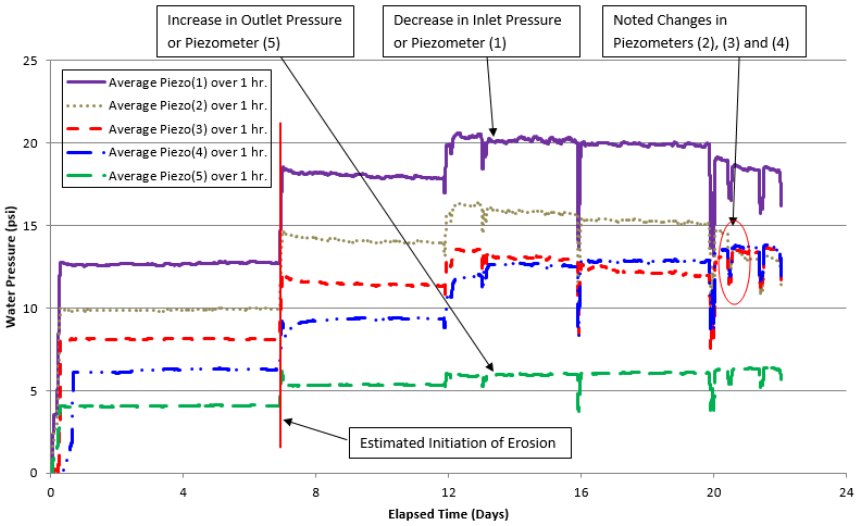


Fig 4. Plot of pore pressure readings versus time for test: Sand/Clay (75/25)(a)

The challenge then becomes correlating the initiation of erosion with a specific value of hydraulic velocity or hydraulic shear stress that is being imposed on the soil. The process for this correlation is illustrated in Fig. 5 where the hydraulic shear stress imposed on the walls of the crack are compared with the measured flow volume. The average shear stress on the walls of the crack, f_{ave}, are calculated using Equation 1:

$$f_{avg} = h/L * \gamma_w * A/P \quad [1]$$

Where, h (m) is the differential head between inlet and outlet or a set portion of the crack, L (m) is the length of the crack or a set portion of the crack, γ_w (kg/m3) is the unit weight of water (used 9810 kg/m^3), A (m^2) is the area of the crack and P (m) is the wetted perimeter of the crack. In Fig. 5, the differential head, h, is calculated between sensors 1 and 5. It is observed that the shear stress on the crack increases, the flow also increases in a nearly linear fashion until an erosive condition is reached at a flow volume of about 0.062 cfs (0.00175 m3/s) (about 7 days into the test as shown on Fig. 4). At this point, erosion initiates and the progression is illustrated by changing the color of the plot symbols for every day of the test hereafter. The erosion at the top of the crack results in a change in differential head across the sensors resulting in a decrease in f_{ave}. The f_{ave} decreases until an equilibrium condition is achieved at approximately day 12 (see Fig. 4) when the total differential head across the JETD is again increased and the progression of erosion is again charted by changing plot colors until equilibrium was achieved.

This test resulted in critical shear stresses of between 2.7 and 2.9 psf (130 and 140 Pa). Hole erosion tests (HETs) were also performed on identical samples of the tested soils (same water content and relative compaction). The HET performed on the sample of Sand/Clay (75/25)(a) resulted in a critical shear stress of about 1.3 psf (62 Pa), considerably less than that indicated by the JETD test.

As stated previously, tests were performed on the nine samples presented in Table 1. It can be observed that samples of the same soil [i.e. Sand/Clay (75/25) and Sand/Bentonite (87.5/12.5)] behaved significantly different due to slight differences in density or compaction water content. In both cases, compaction water content appears to increase the resistance to erosion even at lower densities.

© 2016 Deep Foundations Institute

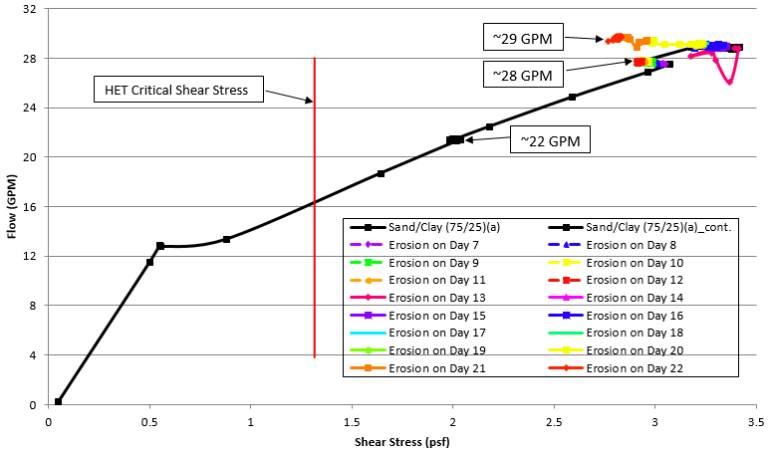


Fig 5. Plot of calculated shear stress versus flow volume for test: Sand/Clay (75/25)(a)

Table 2. Summary of Critical Shear Stress results for soils tested in the JETD

Test Material	Total Unit Weight (pcf)	Water Content (%)	JETD Critical Shear Stress, τ_c (psf/Pa)
Sand/Clay (80/20)	143	13	1.3/54
Sand/Clay (75/25)(a)	148	12	2.7-2.9/130-140
Sand/Clay (75/25)(b)	146	14	2.7/132
Sand/Clay (70/30)	142	15	>2.4/>113*
Sand/Bentonite (87.5/12.5)(a)	137	12	1.9/90
Sand/Bentonite (87.5/12.5)(b)	133	15	3.9/190
Sand/Bentonite (85/15)	145	16	>3.3/>160*
Teton Core	117	14	>4.7>225*
East Branch Core	138	9.5	3.9/190

*Tests in which no appreciable erosion occurred.

Erosion into Voids Device

A schematic illustration of the foundation void test cell (FVTC) used for modeling seepage erosion through an open foundation void such as a crack or solutioned rock within karst, is presented in Fig. 6. The cell consists of a 4 ft. (1.22 m) wide, 4 ft. (1.22 m) tall, 6 in. (0.15 m) deep zone where soil is compacted. An air bladder is placed on top of the soil sample to provide overburden pressure to the soil. Below this air bladder is a 6+ in. (152.4 mm) thick gravel layer which contains a perforated pipe used to disperse the inflow of water to prevent jetting into the soil sample. At the base of the soil section is an adjustable void formed by the gap between two sliding steel plates. These plates can be adjusted to mimic a void ranging from a few millimeters to up to 12 inches (although the largest gap used in the testing was 2 inches). Below the steel plates is a 4 ft. (1.22 m) wide, 2ft. (0.61 m) tall, and 6 in. (.15 m) deep lower cell used to collect the eroding soil. The lower cell is left open to represent an open cavity. The entire upper and lower sections are sealed using 1.5 in. (38.1 mm) thick sheets of Plexiglas. The Plexiglas allows for viewing the soil during erosion while providing a rigid wall to confine the soil. Plumbing is connected to both the upper and lower sections allowing for hydraulic gradients to be applied in either in the upward or downward direction. On the top of the upper section two connections can act as either inlets or drains to allow for flow to occur in the upward or downward direction through the soil

© 2016 Deep Foundations Institute

section, depending on which valves are open or closed and direction of flow. The multiple directions of flow allowed for simulation of seepage erosion under different scenarios that may exist in the field. Details of the testing procedure are available in the full report for the project (Coy and Rice, 2015).

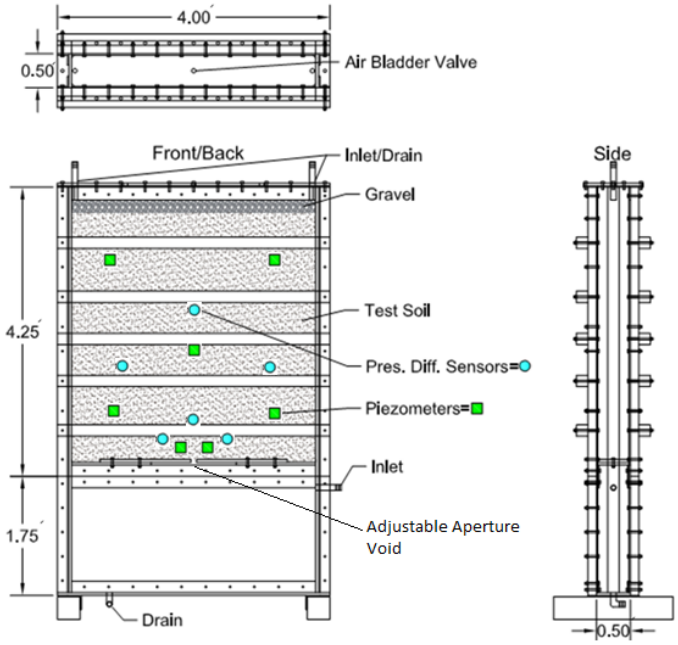


Fig 6. Schematic illustration of the FVTC: Sand/Clay (75/25)(a)

The FVTC is instrumented to measure the flow into the cell and hydraulic pressures within the cell. Flow is measured using an inline ultrasonic flowmeter placed upstream of the cell. Downstream of the flowmeter the line is split and controlled by valves so that the water can be directed to the top or bottom of the cell. Hydraulic pressures are measured throughout the cell using a combination of differential pore pressure transducers and piezometers.

Three of the soils tested consist of soils from actual dams: the Teton Dam Shell, Teton Dam Core, and East Branch Dam Embankment. A fourth sample was manufactured as a mixture of 65 percent by weight medium to coarse poorly graded sand and 35 percent kaolinite clay. The proportions of this sample were chosen to provide the minimum clay content where the sample began to exhibit cohesion. Six models were tested in the FVTC. Summaries of these models and the results are presented in Table 4. Discussions of each of the modes are presented below.

Test 1 – Teton Dam Shell. This test was performed with a 2-inch gap aperture and a constant flow down through the sample. The differential water pressure across the sample was set at 11 psi and the flow was shut off twice during the test to simulate cycling of the flow. The air bladder was set at 21.5 psi (172 kPa) to simulate a soil depth of about 25 feet. The test resulted in piping erosion of the soil resulting in pathways of preferential seepage. However, a total piping failure did not occur due to the formation of filters that prevented progressive washing of the soil. The filters formed when coarse-grained particles bridged the developing pipe and progressively finer particles were subsequently deposited upstream as shown on Fig. 7a. However, despite the filter formation, the erosion did progress throughout the test as evidenced by a continuous increase in flow during the test duration as shown in Fig. 7b. It should be noted that the flow shutoffs did not result in a significant change in the flow, however, several drops in flow were observed over the length of the test. These flow drops were likely due to the collapse of pipes.

© 2016 Deep Foundations Institute

Table 3. Summary of models performed in the FVTC.

Test	Soil	Void Aperture	Loading	Duration	Notes
1	Teton Dam Shell	2 in.	Constant downward flow	23 days	Formed preferential seepage pathways. No complete collapse due to filter formation.
2	East Branch Dam Embankment	1 in.	Cycled from below	25 days (27 cycles)	Slow start of erosion followed by rapid progression. Formed narrow vertical stope prior to widening the erosion. High hydraulic gradient above stope was primary erosion mechanism.
3	Quartz Sand-Kaolinite	1 in.	Cycled from below	10 days (11 cycles)	Slow start of erosion followed by rapid progression. Stope widened at angle of repose from start.
4	Teton Dam Core	1 in.	Cycled from below	14 days (18 cycles)	Slow start of erosion followed by rapid progression. Formed narrow vertical stope prior to widening the erosion. High hydraulic gradient above stope was primary erosion mechanism
5	Teton Dam Core	2 in.	Cycled from below	2 days (2 cycles)	Rapid erosion due to bridging capacity of the soil being exceeded early in the test.
6	Teton Dam Core	1. in	Constant downward flow	25 days	Piping erosion occurred without the development of a concentrated stope

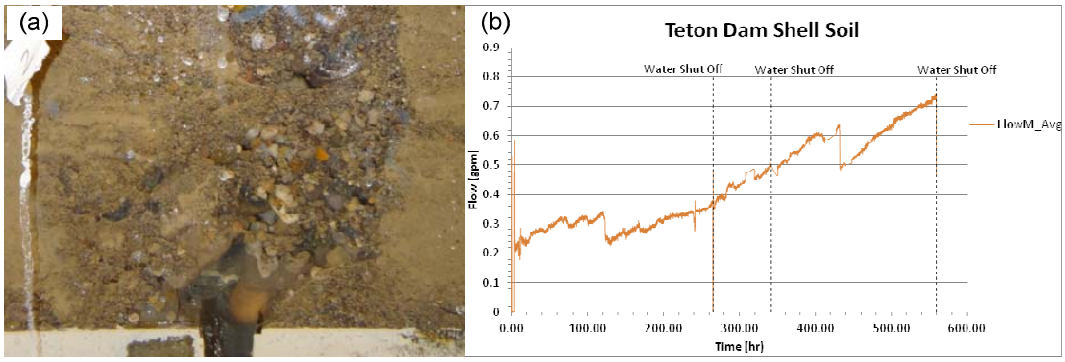


Fig 7 Results of Test 1 in FVTC: (a) photograph of filter formation near the void at the base of the soil, (b) plot of flow volume versus time for the test duration

Test 2 – East Branch Dam Embankment. This test was performed with a 1-inch gap aperture and a cyclic flow from the bottom of the sample. Cyclic flow from the bottom represents groundwater fluctuations that are caused by seasonal rainfall or reservoir fluctuations. The water pressure at the base of the sample was set at 7.5 psi (52kPa) and maintained until equilibrium flow was achieved. The pressure was then shut off and the model allowed to drain through the void into the lower section. The failure progression of this model is illustrated in the photos in Fig. 8. A small amount of erosion occurred at the void after about 4 cycles (Fig. 8a). After 20 cycles (Fig. 8b) the void had not considerably increased in size although signs of upward stoping were evident from the piezometer readings. After 24 cycles (Fig. 8c) the upward stope was observed to be rapidly forming. After the formation of the near vertical stope, the

© 2016 Deep Foundations Institute

sides of the stope eroded back to an angle of repose in one additional cycle (total of 25 cycles in Fig. 8d). Once the stope had widened, the void progressed rapidly upward and broke through to the top of the model in two more cycles (27 total cycles). Thus, the failure progression is one of slow initial void formation, followed by rapid stope formation, erosion of the stope walls once it has reached a critical height, and rapid progression of the widened stope once the bridging width of the void has reached a critical width.



Fig 8 Failure progression of Test 2 on (a) small void formation after 4 cycles, (b) slight increase in void size after 20 cycles, (c) formation of vertical stope after 24 cycles, and (d) widening of the stope after 25 cycles

Test 3 – Quartz Sand-Kaolinite. This test was performed with a 1-inch gap aperture and a cyclic flow from the bottom of the sample, similar to Test 2. The failure of this sample progressed without the formation of the vertical stope as illustrated in Fig. 9. The lack of a vertical stope is likely due to the low cohesive strength of this material.

Test 4 – Teton Dam Core. This test was performed with a 1-inch gap aperture and a cyclic flow from the bottom of the sample, similar to Tests 2 and 3. The failure mechanism was similar to that of Test 2 only occurring in less time (14 cycles versus 25 cycles for Test 2). The increased rate of erosion was likely due to the Teton Dam Core material being more erodible.

Test 5 – Teton Dam Core. This test was performed with a 2-inch gap aperture and a cyclic flow from the bottom of the sample, similar to Tests 2 through 4. The wider initial gap resulted in this test failing very rapidly (in 2 cycles rather than the 13 required for Test 4).

Test 6 – Teton Dam Core. This test was performed with a 1-inch gap aperture and a constant downward flow from the top of the sample, similar to Test 1. A stope did not form in this model, rather, the erosion occurred by developing relatively small aperture pipes through the sample that eventually connected with the gravel layer along the upper edge of the soil.

© 2016 Deep Foundations Institute



Fig 9 Failure progression of Test 2 on (a) small void formation after 4 cycles, (b) slight increase in void size after 20 cycles, (c) formation of vertical stope after 24 cycles, and (d) widening of the stope after 25 cycles

CONCLUSIONS

Two laboratory modeling devices were designed and constructed to model internal erosion mechanisms associated with bedrock defects. The JETD device models internal erosion of soil adjacent to bedrock joints that have considerable bedrock flow. The FVTC device models erosion into bedrock voids such as karstic solution voids and open joints. Conclusions derived from the modeling performed using these two devices are presented below.

JETD Device

Tests were performed on nine samples of soil representing a variety of fines contents, fines plasticity, compaction density, and compaction water content. The results of the tests were analyzed to assess the critical shear stresses needed to erode the respective soils. The resulting critical shear stresses were compared to critical shear strength values obtained from HET tests on similarly compacted soils. The following observations were made:

There are considerable differences between the critical shear stress values for the JETD and HET tests. These differences are likely attributable to several factors: 1) different flow patterns between the two tests results in variation of the turbulence patterns on the soil; 2) the duration of the JETD is considerably longer than the HET, thus allowing for erosion occurring at slower rates to be observable; and 3) minute differences in soil density and compaction water content may have a significant effect on the erodibilty of the soil.

It can also be observed that samples of the same soil behaved significantly different due to slight differences in density or compaction water content. In both cases, compaction water content appears to increase the resistance to erosion, even at lower densities.

FVTC Device

Six models were performed in the FVTC device using four different soil types and two different hydraulic loading conditions. Measurements of the flow volume through the device and the hydraulic pressures within the device were measured during testing. The following observations were made:

In models conducted with constant downward flow (Tests 1 and 6) the mode of internal erosion was the development of discrete but relatively small piping channels that progressed upward from the void to the top of the soil. In the well graded granular soil, the development of natural filters within the pipes slowed the erosion of the soil.

© 2016 Deep Foundations Institute

In models conducted with cyclic hydraulic loading from below (Tests 2 through 5) the mode of internal erosion was the development of a narrow, upward propagating stope that eventually widened when the sides of the stope collapsed back to an angle of repose. In the model with soil of low cohesive strength (Test 3) the stope did not develop to a significant height before the sides collapsed.
The stoping failure mechanism started slowly, taking many cycles to develop, and then accelerated once the narrow stope started to progress vertically.

An internal erosion mechanism was identified where a zone of unsaturated, low-permeability soil develops above the stope resulting in the development of high hydraulic gradients and seepage pressures when the water pressure in the stope is removed and the soil drains. The high resulting gradients were observed in the piezometer and pore pressure transducer data collected during the tests. It is likely that these high gradients accelerated erosion in an upward direction, thus resulting in advancement of the stopes.

ACKNOWLEDGEMENT

The authors would like to acknowledge the US Army Corps of Engineers, Risk Management Center for their support in funding this research.

REFERENCES

Coy, T.K. and Rice, J. D. (2014). Laboratory Modeling of Erosion Potential in Dam Foundations Due to Foundation Voids. A Report to the United States Army Corps of Engineers Risk Management Center. October 2015.

Zaleski, J. and Rice, J. D. (2014). Laboratory Modeling of Erosion Potential of Earthen Embankments in Contact with Open Bedrock Joints. A Report to the United States Army Corps of Engineers Risk Management Center. May 2016.

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

ALTERNATIVE ANALYTICAL METHODS FOR BOREHOLE PERMEABILITY TESTS IN SEEPAGE BARRIER WALLS

R. Lee Wooten, P.E., GEI Consultants, Inc., Woburn, MA, 781-721-4034, lwooten@geiconsultants.com
David R. Shields, P.E., GEI Consultants, Inc., Woburn, MA, 781-721-4032, dshields@geiconsultants.com
Darren D. Clark, P.E., GEI Consultants, Inc., Portland, ME, 207-797-8910, dclark@geiconsultants.com

Keywords: permeability, seepage barrier walls, tests

ABSTRACT

Borehole permeability tests that measure flow or head changes in boreholes located within seepage barrier walls are commonly used to estimate the in situ permeabilities of those walls for quality control and assurance. The current state of practice for estimating permeabilities from these barrier wall borehole tests is to use equations published by M. J. Hvorslev in 1951. Hvorslev's borehole test equations model an infinite stratum of uniform permeability, which is not the actual condition for a test performed in a low permeability barrier wall located within a high permeability stratum. We examine and develop alternative equations based on line source analytical models, which more closely match barrier wall test conditions. We describe how these alternate equations can be used to estimate barrier wall permeabilities from borehole tests and compare the results to those obtained from the Hvorslev models. We also provide recommendations that consider differing upstream and downstream water levels.

BARRIER WALL PERMEABILITY TESTS - STATE OF PRACTICE

The installation of seepage barrier walls has become more common in a variety of applications ranging from dams and levees to contaminated sites requiring retardation of groundwater flow. The types of seepage barrier walls and installation methods include soil-bentonite, soil-cement-bentonite, concrete, self-hardening slurry, geomembranes, and sheet piles, installed with various soil mixing, slurry-supported trench, and driving methods.

Owners and contractors have used a variety of methods to evaluate the permeability of these seepage barriers. The following table lists commonly used quality control/assurance tests for evaluating the permeability of a seepage barrier.

Permeability Test Type	Advantages	Disadvantages
Lab Tests on Wet Grab Samples	• Less expensive to collect samples than other sampling methods • Samples can be collected during construction	• Wet grab sampling collects more fluid part of barrier wall, likely to contain more additive than soil (low bias) • Test sample is processed by screening and molding (low bias) • Lab test can be lengthy and moderately expensive • Tests a small volume

© 2016 Deep Foundations Institute

Permeability Test Type	Advantages	Disadvantages
Lab Tests on Cored Samples	• Tests actual wall sample	• Requires cored borehole, which can only be performed through wide barrier walls that have cured with enough strength to withstand coring stresses • Coring process stresses samples and can cause cracking (high bias) • Only intact samples can be tested (low bias) • Lab test can be lengthy and moderately expensive • Tests a small volume
Borehole Permeability Tests (Rising Head, Falling Head, and Constant Head) Using Analytical Seepage Models	• Tests engage a wider and deeper volume of the barrier wall • Tests are relatively quick	• Requires drilled borehole, which can only be performed through wide barrier walls that have cured with enough strength to withstand drilling stresses • Drilling process stresses barrier wall and can cause cracking (high bias) • Analytical models require simplifying assumptions (high and low biases)
Two Dimensional Numerical Seepage Models Using Piezometer Data	• Tests engage the largest volume of the seepage barrier	• Requires installation of piezometers • Requires completion of most of the seepage barrier to establish two dimensional conditions • Requires development of numerical models with assumptions that affect the results

Our notes of high or low bias indicate that the test type either overestimates (high bias) or underestimates (low bias) seepage barrier permeability. Test types that overestimate permeabilities (high bias) provide conservative quality control/assurance criteria for seepage barriers for which a low permeability is desired.

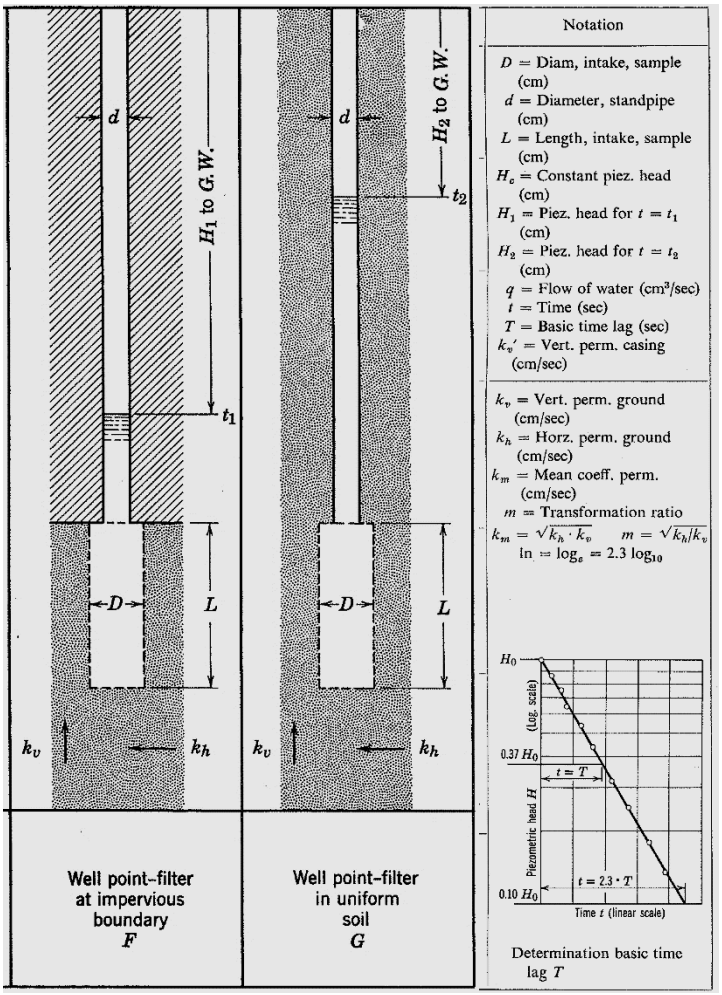
Our paper focuses on the analytical models used to evaluate the results of borehole permeability tests performed in seepage barrier walls. We do not address the numerous practical considerations involved in the actual field performance of these tests. Also, we note that quality control and assurance of seepage barrier walls requires checks on other wall characteristics besides permeability (e.g. position, depth, dimensions, continuity), which we do not address.

HVORSLEV ANALYTICAL MODELS

In our experience, the borehole permeability test analytical model that is most commonly used for geotechnical applications, including seepage barrier walls, is the one described by Hvorslev in his 1951 Waterways Experiment Station Bulletin (Hvorslev 1951) as Case G, shown in Fig. 1. While there are some

© 2016 Deep Foundations Institute

other analytical methods that are used to evaluate aquifer permeability from pump tests and slug tests (Halford and Kuniansky 2002), we have focused on the Hvorslev method because of its common use for geotechnical applications. We also note that these other methods share drawbacks similar to the Hvorslev method if applied to barrier walls.



Case	Constant Head	Variable Head	Basic Time Lag
F	$k_h = \frac{q \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_e}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ for $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{2mL}{D} > 4$
G	$k_h = \frac{q \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_e}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln\frac{H_1}{H_2}$ for $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{mL}{D} > 4$

Fig. 1. Cases F and G from Hvorslev (1951)

© 2016 Deep Foundations Institute

Using the assumption of isotropic permeability (m = 1) and the condition that the borehole diameter is both constant (D = d) and less than one quarter of the saturated length of the borehole (L/D > 4), the Case G equation is typically simplified to the following formulas for use with borehole permeability tests:

$$k = q \frac{\ln\left(\left(\frac{L}{d}\right) + \sqrt{1 + \left(\frac{L}{d}\right)^2}\right)}{2\pi L H_c}$$

for constant head pumping tests, and

$$k = \frac{d^2 \ln\left(\frac{2L}{d}\right) \ln\left(\frac{H_1}{H_2}\right)}{8L(t_2 - t_1)}$$

for variable (rising or falling) head tests where

k = stratum permeability
q = rate of pumping in a pump test
L = vertical height of the test zone
d = borehole diameter
H = head = difference between the water level in the borehole and the water table
H_c = constant head in a pumping test
H_1 = head at time t_1 in a variable head test
H_2 = head at time t_2 in a variable head test

The Hvorslev Case G model is for a borehole permeability test performed in an unconfined aquifer. Hvorslev presents a similar model, Case F, for a test performed in a confined aquifer, shown in Fig. 1. Applying the same simplifications used for the Case G model above yields the following equations for Case F:

$$k = q \frac{\ln\left(\left(\frac{2L}{d}\right) + \sqrt{1 + \left(\frac{2L}{d}\right)^2}\right)}{2\pi L H_c}$$

for constant head pumping tests, and

$$k = \frac{d^2 \ln\left(\frac{4L}{d}\right) \ln\left(\frac{H_1}{H_2}\right)}{8L(t_2 - t_1)}$$

for variable (rising or falling) head tests.

© 2016 Deep Foundations Institute

The most obvious difference between the Hvorslev models and the conditions of a borehole test in a seepage barrier wall is that the barrier wall has a limited width, typically about 3 feet, compared to the Hvorslev model assumption of flow through an infinite uniform stratum. In a barrier wall permeability test, where the barrier wall has significantly lower permeability than the surrounding strata, the effective flow path, from the driving head source at the wall face to the borehole, is much shorter than that assumed in the Hvorslev models. Consequently, permeabilities estimated from borehole permeability test data using the Hvorslev models are higher than the actual seepage barrier permeabilities, a high bias.

Two other differences between the Hvorslev models and the conditions in a seepage barrier borehole test are that:

1. Water levels on each side of the seepage barrier wall may be different.
2. Borehole tests in barrier walls are commonly performed in open (uncased) holes that do not have a set well screen length, L, as in the Hvorslev models.

We discuss methods to adapt the Hvorslev models to conditions with differing water levels on each side of a barrier wall in a later section. Options for selection of L for applying the Hvorslev models to uncased boreholes are described below.

For applying the Hvorslev Case G model to an uncased borehole test in a barrier wall located in an unconfined aquifer, the obvious options for L are:

1. Length from groundwater level outside the barrier wall to the bottom of the borehole.
2. Length from average water level inside the borehole to the bottom of the borehole.

The first option seems reasonable for a constant head test or rising head test because it represents the length over which water enters the test zone. The second option seems reasonable for a falling head test for the same reason. Note that these selections yield higher values for L than the other choices, which results in lower estimates of permeability. Determining which choice is more "correct" is problematic because the Hvorslev model is not really applicable to an uncased borehole.

For applying the Hvorslev Case F model to an uncased borehole test in a barrier wall located in a confined aquifer, it seems reasonable to set L equal to the borehole length that is located within the confined pervious stratum.

LINE SOURCE MODELS

Mansur and Kaufman, in the Dewatering chapter of Leonards's *Foundation Engineering* (Mansur and Kaufman 1962), presented a section entitled "Flow to a Single Well—Line Source," which describes analytical models for flow to a well from a linear constant head boundary, a "line source" in plan or, more precisely, a vertical plane source in three dimensions. In their analytical models, an infinite constant head line source is created by using an "image well" (a mirror image recharge well) located on the opposite side of the line source (Figs. 2, 3, and 4). Mansur and Kaufman then apply the superposition procedure of Forchheimer (1886) and Dachler (1924) to develop equations for the quantity of flow from the line source

© 2016 Deep Foundations Institute

to the well as a function of the drawdown in the well. They present equations for fully penetrating wells in both confined and unconfined aquifers.

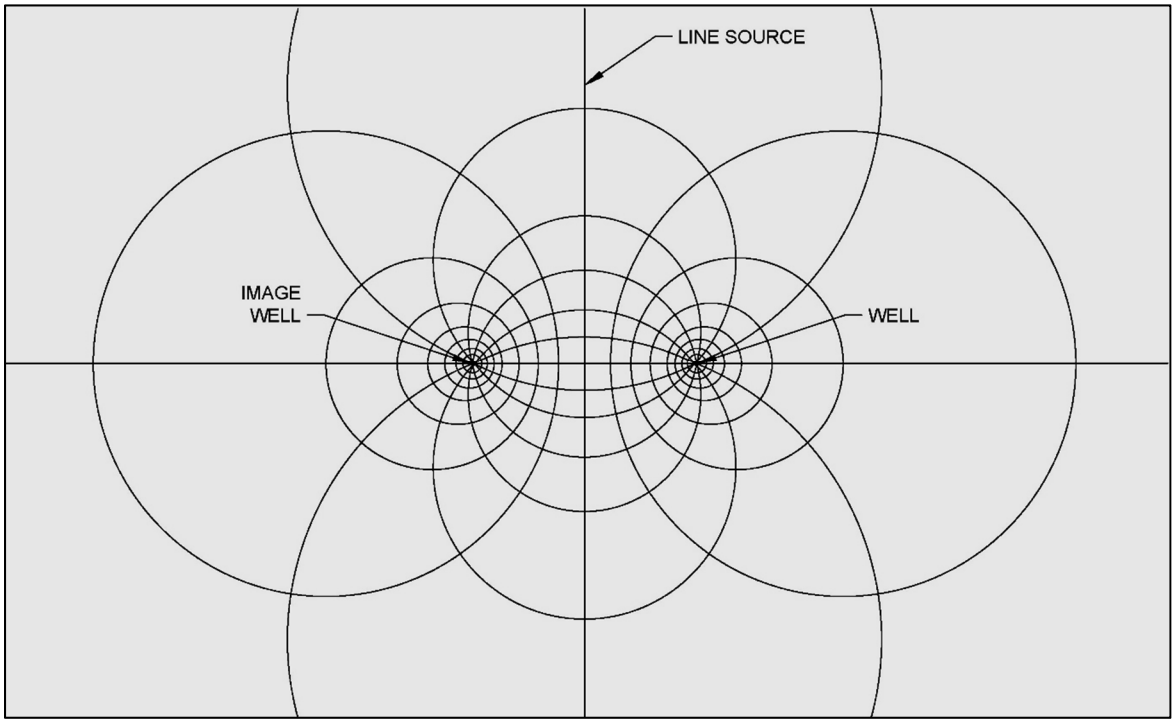


Fig. 2. Plan View of Line Source Model Flow Net

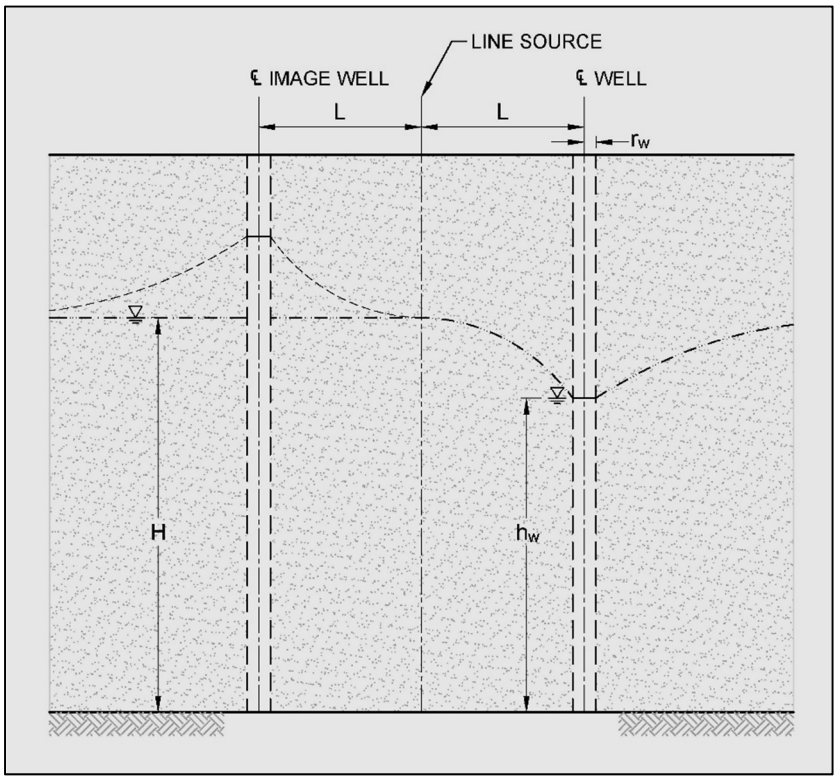


Fig. 3. Vertical Section of Line Source Model in an Unconfined Aquifer

© 2016 Deep Foundations Institute

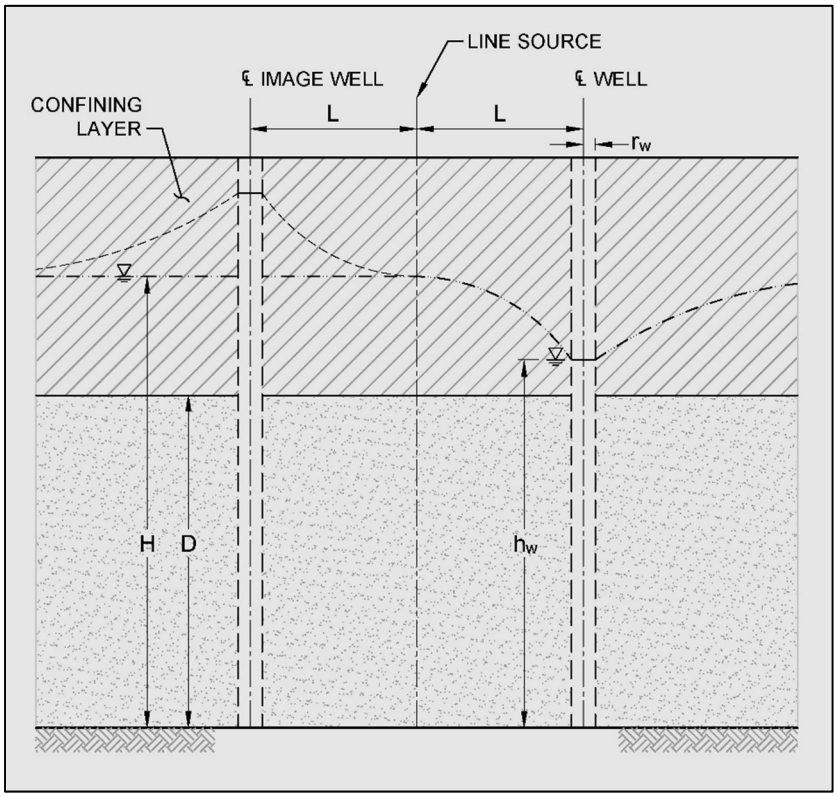


Fig. 4. Vertical Section of Line Source Model in a Confined Aquifer

The models shown in Figs. 3 and 4 assume that there is no well entry head loss, so the water level in the well matches the adjacent water level in the stratum.

The line source models can be applied to analysis of borehole permeability tests in barrier walls as follows:

- The borehole (located at the center of the barrier wall) is modeled as a well located within a stratum that consists of the barrier wall. Because the material outside the barrier wall has a much greater permeability than the wall, it acts as a constant head line source located at the wall face.
- Each face of the barrier wall is modeled as a constant head line source. Thus the flow into the borehole will be twice the flow given by the equations derived for a single line source.
- An impervious boundary is assumed at the bottom of the borehole (i.e., the flow entering from below the bottom of the borehole is assumed to be negligible). This assumption is conservative and should have little effect when the borehole length is much greater than the barrier wall thickness.
- The unconfined aquifer model can be used for an uncased borehole test in a barrier wall located in an unconfined aquifer.
- The confined aquifer model can be used for a barrier wall that cuts off a confined pervious stratum. (It could also be used for a test performed in a partially cased borehole by assuming the upper confining layer is located at the top of the uncased section. However, partially cased borehole tests are not commonly performed in barrier walls.)

© 2016 Deep Foundations Institute

Application of the line source models to barrier wall tests by using a line source on each side of the wall is illustrated in Fig. 5 (unconfined aquifer) and Fig. 6 (confined aquifer). Equations for the line source models are presented in the following sections.

Unconfined Aquifer Model

Figure 3 shows a vertical section through a single line source model for a fully penetrating well in an unconfined aquifer. We use the same variable designations used by Mansur and Kaufman, which differ from the variable designations used by Hvorslev that we presented previously. We use the original authors' variable designations to minimize confusion when referring back to the original papers.

Mansur and Kaufman present the following equation for flow to the well from an infinite line source:

$$Q_w = \frac{\pi k\left(H^2 - {h_w}^2\right)}{\ln\left(\frac{2L}{r_w}\right)}$$

where

Q_w = flow to the well
k = stratum permeability (homogeneous, isotropic assumption)
H = head of line source above bottom of aquifer (same as bottom of well)
h_w = head in well above bottom of aquifer
L = minimum (perpendicular) distance from the well center to the line source
r_w = radius of well

Rearranging the terms provides the following analytical equation for calculating permeability from a constant head pumping test with recharge from a single line source:

$$k = \frac{Q_w \ln\left(\frac{2L}{r_w}\right)}{\pi\left(H^2 - {h_w}^2\right)}$$

To apply this equation to a constant head pumping test performed in an uncased borehole located at the center of a barrier wall in an unconfined aquifer, L would be one-half of the wall thickness, and k would equal one-half of the value calculated by the above equation because there are two line sources. This is illustrated in Fig. 5 and the corresponding equation is given below:

$$k = \frac{Q_w \ln\left(\frac{T}{r_w}\right)}{2\pi\left(H^2 - {h_w}^2\right)}$$

where T = barrier wall thickness

© 2016 Deep Foundations Institute

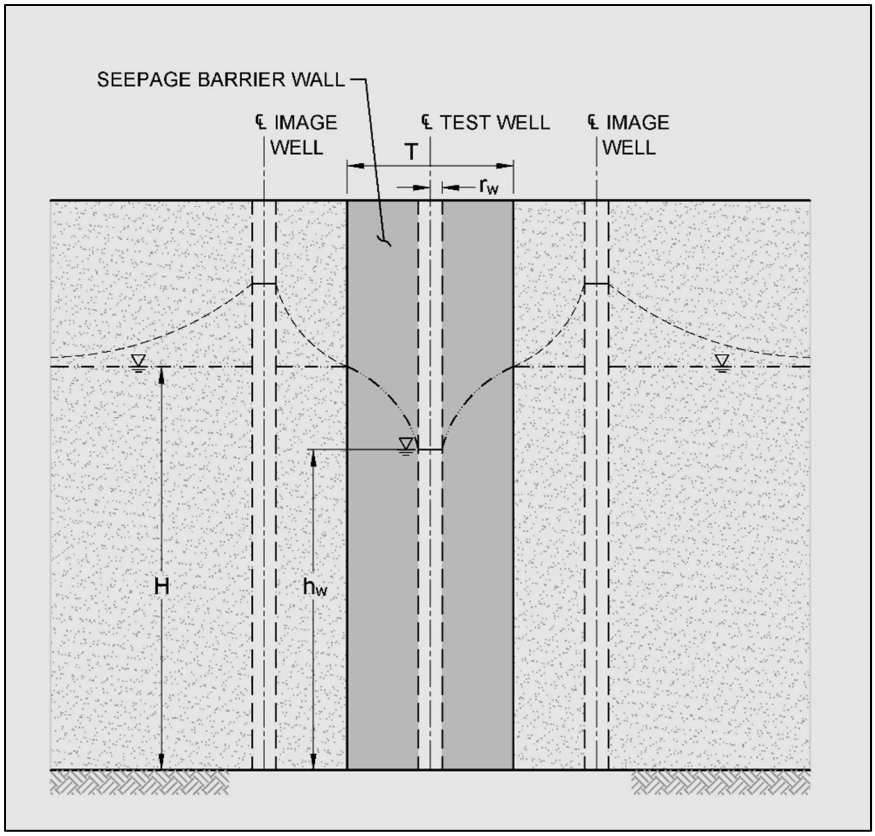


Fig. 5. Line Source Model for Borehole Test in a Barrier Wall in an Unconfined Aquifer

For a variable head test with recharge from a single line source, we set Q_w to equal the rate of volume change in the well:

$$Q_w(t) = \pi {r_w}^2 \left(\frac{dh_w}{dt}\right)$$

The permeability model then becomes:

$$k(h_w, t) = \frac{{r_w}^2 \ln\left(\frac{2L}{r_w}\right)\left(\frac{dh_w}{dt}\right)}{\left(H^2 - {h_w}^2\right)}$$

For a variable head borehole test with h_{w1} and h_{w2} measurements at times t_1 and t_2, we have the equation:

$$k = \left({r_w}^2 \ln\left(\frac{2L}{r_w}\right)\right) \int_{h_{w1}}^{h_{w2}} \frac{1}{H^2 - {h_w}^2} dh_w \left(\frac{1}{t_2 - t_1}\right)$$

which simplifies to:

© 2016 Deep Foundations Institute

$$k = \left(\frac{{r_w}^2 \ln\left(\frac{2L}{r_w}\right)}{2H(t_2 - t_1)} \right) \ln\left(\frac{(H + h_{w2})(H - h_{w1})}{(H - h_{w2})(H + h_{w1})} \right)$$

To apply this equation to a variable head test performed in an uncased borehole located at the center of a barrier wall in an unconfined aquifer, L would be one-half of the wall thickness, and k would equal one-half of the value calculated by the above equation because there are two line sources. The corresponding equation is given below:

$$k = \left(\frac{{r_w}^2 \ln\left(\frac{T}{r_w}\right)}{4H(t_2 - t_1)} \right) \ln\left(\frac{(H + h_{w2})(H - h_{w1})}{(H - h_{w2})(H + h_{w1})} \right)$$

Confined Aquifer Model

Figure 4 shows a vertical section through a single line source model for a fully penetrating well in a confined aquifer. We use the same variable designations used by Mansur and Kaufman, which differ from the variable designations used by Hvorslev that we presented previously. We use the original authors' variable designations to minimize confusion when referring back to the original papers.

Mansur and Kaufman present the following equation for flow to the well from an infinite line source:

$$Q_w = \frac{2\pi kD(H - h_w)}{\ln\left(\frac{2L}{r_w}\right)}$$

where

Q_w = flow to the well
k = stratum permeability (homogeneous, isotropic assumption)
D = stratum thickness
H = head of line source above bottom of aquifer (same as bottom of well)
h_w = head in well above bottom of aquifer
L = minimum (perpendicular) distance from the well center to the line source
r_w = radius of well

Rearranging the equation provides the following analytical equation for calculating permeability from a constant head pumping test with recharge from a single line source:

$$k = \frac{Q_w \ln\left(\frac{2L}{r_w}\right)}{2\pi D(H - h_w)}$$

To apply this equation to a constant head pumping test performed in an uncased borehole located at the center of a barrier wall in a confined aquifer, L would be one-half of the wall thickness, D would be the

© 2016 Deep Foundations Institute

borehole length located within the confined pervious stratum, and k would equal one-half of the value calculated by the above equation because there are two line sources. This is illustrated in Fig. 6 and the corresponding equation is given below:

$$k = \frac{Q_w \ln\left(\frac{T}{r_w}\right)}{4\pi L_{ps}(H - h_w)}$$

where

T = barrier wall thickness
L_{ps} = borehole length located within the confined pervious stratum

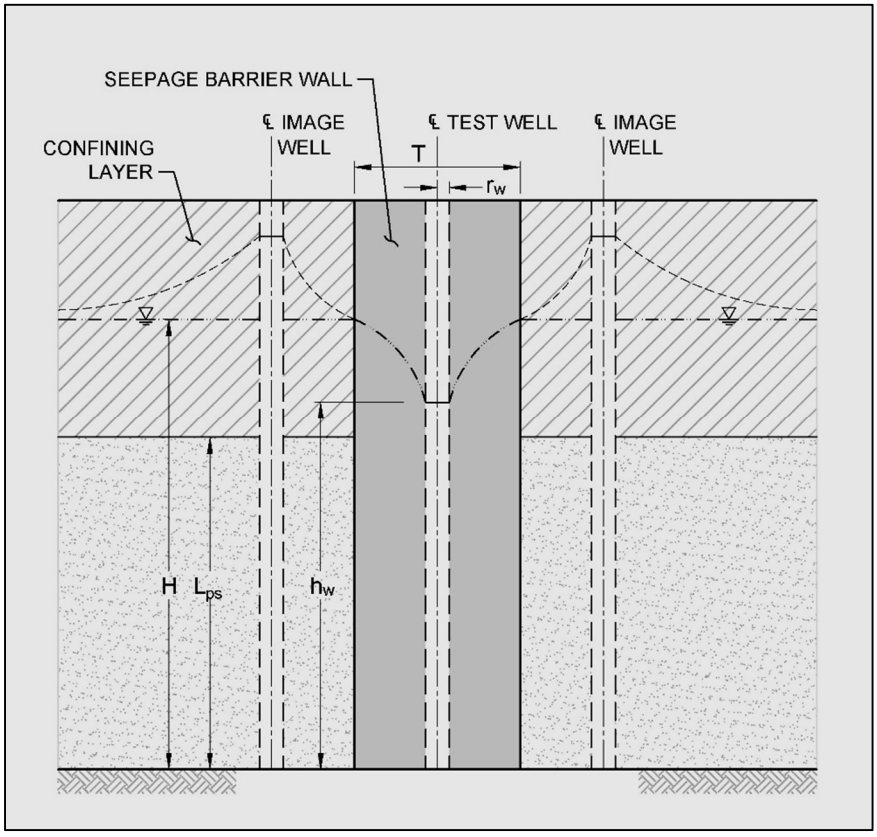


Fig. 6. Line Source Model for Borehole Test in a Barrier Wall in a Confined Aquifer

For a variable head test with recharge from a single line source, we set Q_w equal to the rate of volume change in the well:

$$Q_w(t) = \pi {r_w}^2 \left(\frac{dh_w}{dt}\right)$$

The permeability model then becomes:

© 2016 Deep Foundations Institute

$$k(h_w, t) = \frac{{r_w}^2 \ln\left(\frac{2L}{r_w}\right)\left(\frac{dh_w}{dt}\right)}{2D(H - h_w)}$$

For a variable head borehole test with h_{w1} and h_{w2} measurements at times t_1 and t_2, we have the equation:

$$k = \left(\frac{{r_w}^2 \ln\left(\frac{2L}{r_w}\right)}{2D}\right)\int_{h_{w1}}^{h_{w2}} \frac{1}{H - h_w}\, dh_w \left(\frac{1}{t_2 - t_1}\right)$$

which simplifies to:

$$k = \left(\frac{{r_w}^2 \ln\left(\frac{2L}{r_w}\right)}{2D(t_2 - t_1)}\right)\ln\left(\frac{H - h_{w1}}{H - h_{w2}}\right)$$

To apply this equation to a variable head test performed in an uncased borehole located at the center of a barrier wall in a confined aquifer, L would be one-half of the wall thickness, D would be the borehole length located within the confined pervious stratum, and k would equal one-half of the value calculated by the above equation because there are two line sources. The corresponding equation is given below:

$$k = \left(\frac{{r_w}^2 \ln\left(\frac{T}{r_w}\right)}{4L_{ps}(t_2 - t_1)}\right)\ln\left(\frac{H - h_{w1}}{H - h_{w2}}\right)$$

DIFFERING UPSTREAM AND DOWNSTREAM WATER LEVEL CONDITIONS

A significant factor in seepage barrier borehole permeability tests that is not considered in the Hvorslev or line source models presented thus far is that the outside water levels are often not equal on both sides of the barrier wall; whereas, the models assume a uniform water table. To address this condition, we can divide the models into two infinite half-spaces separated by a vertical impermeable boundary through the center of the borehole. We can then evaluate the flow contributed from each side.

For a constant head pumping test, this divided half space model results in the same equations as before with the head term H being replaced with H_{ave}, the average of the upstream head and the downstream head. This modification applies to both the Hvorslev and line source equations.

Unfortunately, the variable head equations do not simplify as easily as the constant head test equations, a complication that exists because the variable head terms ($\ln\left(\frac{H_1}{H_2}\right)$ in the Hvorslev models, $\ln\left(\frac{(H+h_{w2})(H-h_{w1})}{(H-h_{w2})(H+h_{w1})}\right)$ in the line source unconfined aquifer model, and $\ln\left(\frac{H-h_{w1}}{H-h_{w2}}\right)$ in the line source confined aquifer model) capture both gradient and flow volume factors. Using the divided half-space approach described above, we need to make the additional assumption that the flow to or from each side contributes

© 2016 Deep Foundations Institute

a constant fraction of the resulting change in the borehole water volume. Then we can set up three equations to solve for the three unknowns, which are k and the fractions of borehole water volume change contributed by each side. This yields the following equation:

$$k = \frac{2k_d k_u}{k_d + k_u}$$

where

k_d = value of k computed by assuming a uniform water table equal to the downstream water level
k_u = value of k computed by assuming a uniform water table equal to the upstream water level

This equation is subject to the following limitations:

- The water level in the borehole must remain above both of the outside water levels in a falling head test, or remain below both of the outside water levels in a rising head test.
- The assumption that each side contributes a constant fraction of the change in borehole water volume results in different calculated permeabilities for different test intervals.

Considering these limitations and the computational effort required to apply the analysis method for variable head tests described above, it appears preferable to us to simply set the head term H equal to H_{ave} to estimate k in each of the equations. While not an exact solution, this simplification should provide a reasonable estimate of permeability.

Borehole tests are often performed without knowing the upstream and/or downstream water levels because of the lack of piezometers in the area of the tests. For these conditions, the equilibrium water level in the borehole can be used as an approximation of the average groundwater level outside the barrier wall. An advantage of using the borehole equilibrium water level is that it can be easily measured or estimated from test data in the field at the time of the test.

COMPARISON OF MODELS

To compare the Hvorslev and line source models for borehole tests, we needed to make assumptions about the value to use for the well screen length, L in the Hvorslev equations. These assumptions are required because the Hvorslev model assumes a fixed well screen length, which does not apply to uncased holes. For our comparisons we assumed values for Hvorslev's L as indicated below.

Hvorslev Case G applied to an unconfined aquifer:

- Constant head pumping test or rising head test - L equals length from groundwater level outside the barrier wall to the bottom of the borehole
- Constant head recharge test or falling head test - L equals length from average water level inside the borehole to the bottom of the borehole

Hvorslev Case F applied to a confined aquifer:

- All tests - L equals the borehole length that is located within the confined pervious stratum.

© 2016 Deep Foundations Institute

We compared permeability values calculated by the Hvorslev equations and line source equations for a typical barrier wall test example consisting of a 4-inch-diameter borehole inside a 3-foot-thick barrier wall. We varied the different parameters in the models to see how each parameter affected the results. The results are presented as the ratio of the permeability obtained from the Hvorslev model to the permeability obtained from the line source model in the following tables.

Table 1. Constant Head Test Model Comparisons for Unconfined Aquifers

Height of Water[1] (ft)			
Outside Barrier Wall	Inside Borehole	L (Hvorslev) (ft)	Ratio: k (Hvorslev)/ k (Line Source)
Constant Head Pumping Test: Vary Height of Water Outside Barrier Wall			
25	10	25	2.4
75	10	75	2.4
Constant Head Pumping Test: Vary Height of Water Inside Borehole			
50	20	50	2.8
50	40	50	3.6
Constant Head Recharge Test: Vary Height of Water Outside Barrier Wall			
25	80	80	2.8
75	80	80	4.1
Constant Head Recharge Test: Vary Height of Water Inside Borehole			
50	75	75	3.5
50	55	55	3.8

Table 2. Constant Head Test Model Comparisons for Confined Aquifers

Height of Water[1] (ft)		Height of Confined Aquifer (ft)	Ratio: k (Hvorslev)/ k (Line Source)
Outside Barrier Wall	Inside Borehole		
Constant Head Pumping Test: Vary Height of Water Outside Barrier Wall			
25	5	25	3.9
75	5	25	3.9
Constant Head Pumping Test: Vary Height of Water Inside Borehole			
50	10	25	3.9
50	30	25	3.9
Constant Head Pumping Test: Vary Height of Confined Aquifer			
50	5	10	3.3
50	5	30	4.1
Constant Head Recharge Test: Vary Height of Water Outside Barrier Wall			
25	80	25	3.9
75	80	25	3.9
Constant Head Recharge Test: Vary Height of Water Inside Borehole			
50	90	25	3.9
50	70	25	3.9
Constant Head Recharge Test: Vary Height of Confined Aquifer			
50	80	10	3.3
50	80	30	4.1

Notes for Tables 1 and 2:

1. Height of water above the bottom of the borehole.
2. For constant head tests the permeability ratio is independent of the flow rate.

© 2016 Deep Foundations Institute

Table 3. Variable Head Test Model Comparisons for Unconfined Aquifers

Height of Water[1] (ft) Outside Barrier Wall	Inside Borehole t_1	Inside Borehole t_2	L (Hvorslev) (ft)	Ratio: k (Hvorslev)/ k (Line Source)
Rising Head Test: Vary Height of Water Outside Barrier Wall				
25	5	10	25	2.3
75	5	10	75	2.3
Rising Head Test: Vary Height of Water Inside Borehole at Time t_2				
50	5	20	50	2.5
50	5	40	50	3.0
Falling Head Test: Vary Height of Water Outside Barrier Wall				
25	85	80	82.5	2.8
75	85	80	82.5	4.1
Falling Head Test: Vary Height of Water Inside Borehole at Time t_2				
50	85	75	80	3.5
50	85	55	70	3.4

Table 4. Variable Head Test Model Comparisons for Confined Aquifers

Height of Water[1] (ft) Outside Barrier Wall	Inside Borehole t_1	Inside Borehole t_2	Height of Confined Aquifer (ft)	Ratio: k (Hvorslev)/ k (Line Source)
Rising Head Test: Vary Height of Water Outside Barrier Wall				
25	5	10	25	3.9
75	5	10	25	3.9
Rising Head Test: Vary Height of Water Inside Borehole at Time t_2				
50	5	20	25	3.9
50	5	40	25	3.9
Rising Head Test: Vary Height of Confined Aquifer				
50	5	10	10	3.3
50	5	10	30	4.1
Falling Head Test: Vary Height of Water Outside Barrier Wall				
25	85	80	25	3.9
75	85	80	25	3.9
Falling Head Test: Vary Height of Water Inside Borehole at Time t_2				
50	85	75	25	3.9
50	85	55	25	3.9
Falling Head Test: Vary Height of Confined Aquifer				
50	85	80	10	3.3
50	85	80	30	4.1

Notes for Tables 3 and 4:

1. Height of water above the bottom of the borehole.
2. For variable head tests the permeability ratio is independent of the test duration.

These comparisons indicate that the Hvorslev models overestimate the permeability compared to the line source models by factors ranging from about 2.3 to 4.1 for the range of test conditions that we considered. While these are modest differences compared to the potential variation of the permeability, which can vary by orders of magnitude, the differences are significant enough to justify use of the more realistic line source models.

© 2016 Deep Foundations Institute

MODEL BIASES

The application of the line source models to analyses of borehole permeability tests in barrier walls employs two simplifying assumptions that introduce model bias, as discussed below.

The assumption of no well entry head loss introduces a potential low (unconservative) bias. The existence of any well entry head loss would result in an underestimate of permeability by the model. This is considered unlikely for the low velocity flow in a seepage barrier test. We also note that the Hvorslev models assume no entry head loss, and thus are subject to the same potential bias.

The assumption of an impermeable boundary at the bottom of the borehole, and at the top of the test zone in a confined aquifer test, introduces a high (conservative) bias. The small amount of flow that enters the borehole from flow paths located above and below the assumed impermeable boundaries results in an overestimate of permeability by the model. This bias should be small for a typical borehole test with a test length that is much greater than the barrier wall thickness.

CONCLUSIONS

None of the analytical models presented here match the realities and complexities of a constructed seepage barrier. The material of a seepage barrier is usually designed such that permeabilities of the barrier material are well below the typical target of 10^{-6} cm/s. Defects, such as zones of poorly treated soil or cracks, not seepage barrier material permeability, often govern the effectiveness of seepage barriers. Those same defects may also control the outcome of borehole permeability tests.

With these considerations in mind, we offer the following conclusions:

- Borehole permeability tests performed inside seepage barrier walls are commonly analyzed using the Hvorslev Case G or Case F models. These models overestimate the barrier wall permeability because they assume flow from an infinite stratum, while a barrier wall actually has a limited width with a short flow path from the wall face to the borehole.

- Borehole permeability tests in barrier walls can be analyzed with line source models, which are more representative of the actual test conditions. Use of these line source models eliminates the overestimation of permeability obtained from use of the Hvorslev models for barrier wall tests.

- Borehole permeability tests in seepage barrier walls may be performed under conditions where the water table on the downstream side of the wall is lower than the water table on the upstream side. The Hvorslev and line source models can be used to estimate permeability for this condition by using the average of the two different outside water levels or by using the equilibrium water level in the borehole as the outside water level.

On a final note, our derived equations, particularly those for variable head tests, evolved as we considered each analytical model in depth. We welcome additional scrutiny of our model development and input regarding the use of the equations that we present.

© 2016 Deep Foundations Institute

REFERENCES

Dachler, R., 1924. Einige Bemerkungen zur Grundwasserabsenkung mittels Brunnengruppen. *Wasserkraft und Wasserwirtschaft*, Munich, vol. l9, no. 21.

Forchheimer, P., 1886. Ueber die Ergiebigkeit von Brunnen-Anlagen und Sickerschlitzen. *Zeitschrift der Architekten und Ingenieur-Verein*, Hannover, vol. 32, no. 7.

Halford, Keith J. and Kuniansky, Eve L., 2002. Documentation of Spreadsheets for the Analysis of Aquifer-Test and Slug-Test Data, U.S. Geological Survey Open-File Report 02-197. Carson City, Nevada. 51 p.

Hvorslev, M. Juul, 1951. Time Lag and Soil Permeability in Ground-Water Observations. *Bulletin No. 36.* Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Mississippi, April. 50 p.

Mansur, Charles I. and Kaufman, Robert I., 1962. Dewatering. *Foundation Engineering.* Leonards, G. A., editor, McGraw-Hill Book Company, pp. 296-297.

© 2016 Deep Foundations Institute

This page left intentionally blank

© 2016 Deep Foundations Institute

ASSESSMENT OF THE DURABILITY OF CEMENTITIOUS BARRIER MATERIALS

Stephan A. Jefferis, Environmental Geotechnics Ltd and Visiting Professor, University of Oxford, Banbury, Oxfordshire, England
+44 1295 811074, egl@environmentalgeotechnics.com
Adrian H. Bath, Intellisci Ltd, Willoughby-on-the-Wolds, Loughborough, England
+44 01509 889229, abath@intellisci.co.uk

ABSTRACT

This paper draws together some themes on the durability of barrier systems, and particularly cementitious systems. For contaminated land and water retaining structures, durability can be poorly addressed at all stages of a project, from specification, to barrier material selection, barrier design including toe-in and through to performance assessment in operation. Permeability (hydraulic conductivity) is a dynamic property which will vary over time as reaction processes propagate through a barrier. However, laboratory testing, when undertaken, tends to be focused on short term tests with aggressive chemicals so that the effects of longer-term processes such as carbonation and leaching by permeating water are missed. For water retaining structures, carbonation and leaching actually may be the most significant processes affecting barrier performance as the groundwater adjacent to many water bodies can be otherwise rather benign (as regards bentonite and cementitious systems). These and other reaction processes have been modelled in the arena of nuclear waste repositories and procedures are well recognised. However, such procedures have yet to be widely adopted for the assessment of other barrier systems.

Keywords: durability, cement-bentonite, barrier walls, permeability, hydraulic conductivity, geochemical modelling

INTRODUCTION

Over the last few decades there has been an enormous amount of laboratory research on the durability of barrier materials or more specifically papers reporting the results of permeability tests. Unfortunately there is a much smaller literature on scaling test results to barrier dimensions in the field or to required design lives. Although in the geotechnical industry durability research has been largely laboratory based, in the nuclear industry where relevant timescales are multi-millennia rather than decades, research has been heavily underpinned by geochemical modelling. A significant purpose of this paper is to examine the tools that may be used to analyse the results of tests on barrier materials used in geotechnical engineering, for example, for water retaining structures and contaminated land and draw the attention of the construction industry to the potential of the modelling procedures developed for nuclear repository assessments. Modelling can help to identify underlying processes and so augment information from laboratory testing focused on changes of physical properties such as permeability and strength.

A further issue with barrier systems is that there has been too little use of field data despite the fact that there are so many barriers systems currently in service and being monitored. Back analysis of field performance data could give valuable information on material performance. The majority of work that has been done, for example for landfill liners, has focused on immediate effects rather than long term durability. A notable exception for cut-off walls is the work of Tedd et al. (1997), which showed, amongst other important insights, that degradation is very rapid when a slurry wall is excavated through soils containing damaging chemicals (in this case including sulfates) which became entrained in the slurry.

Savage (2011) gives an example of the use of field data with a review of industrial and natural analogues of reactions over short- and long-timescales between cementitious barrier materials and surrounding rocks. This work was carried out in relation to barrier performance in nuclear waste repositories.

© 2016 Deep Foundations Institute

PROPERTIES OF BARRIER MATERIALS

Permittivity

The key parameter for a barrier system is its permittivity (the reciprocal of resistance to flow), that is (k/d) where k is the overall hydraulic conductivity (permeability) of the barrier material and d the effective thickness of the barrier (in soil-bentonite walls the filter cake on the upstream and downstream walls can make an important contribution to the barrier permittivity, in cement-bentonite walls these filter cakes will again make a contribution but their effect may be more modest). For a cement-bentonite cut-off wall d may be 600 mm or greater whereas for a bentonite geocomposite liner (GCL) it may be over 100 times less at about 5 mm. Thickness is also of course of major significance when considering diffusive transport.

Permeability testing

Typically permeability tests are done using flexible wall permeameters (i.e. specimens sealed within a confining membrane in a triaxial type set-up) rather than fixed wall permeameters where edge leakage can be a problem – though it also can be an indication of an important phenomenon, material shrinkage as a result of reaction with the permeant. Laboratory permeability tests are typically done under relatively high gradients, perhaps of the order of 10 to 20 to minimise test time. Diffusion will make a contribution to the rate of advance of reaction processes through a test sample though the boundary conditions may be poorly defined in laboratory testing with a triaxial cell set-up.

Strength

Strength may not be a major criterion for in-situ barrier performance. However, for cemented systems, it may be used as a control parameter during construction and later as an indicator of impacts from chemical reactions. Typically these are assessed in the laboratory by crushing samples after immersion tests.

Volume change

Volume change and cracking as a result of chemical / physical impacts on barrier materials can produce localised high permeability zones. Expansion and shrinkage may be caused by specific reactions and thus can be amenable to predictive chemical modelling. Immersion tests can help to identify chemically induced cracking as a result of induced volume changes. Less often adopted but providing useful information are immersion tests with displacement transducers to measure changes in dimensions during immersion. Such tests may be particularly useful for processes where there is shrinkage without observable cracking in laboratory tests.

Expansive reactions from sulfate attack are widely recognised and can lead to disintegration of some cement containing barrier materials (Tedd et al. 1997 and Jefferis 2003).

Volume change also can occur as a result of temperature change, for example shrinkage during cooling after the heat release from hydration of the cementitious content (Cernak et al., 2012, Evans and Jefferis, 2014). Such shrinkage of barrier materials in-situ may lead to the opening of cracks – though many of which may be sufficiently fine that they can self-heal by carbonation.

Strain at failure

The ability to withstand strain without failure will be important for barriers such as dam cores which may be subject to strain in service (for example, from movements on impoundment). Unfortunately strain at failure is an inherent property of materials which is difficult to vary except by selecting a different material.

© 2016 Deep Foundations Institute

For a barrier system failure implies an unacceptable increase in permeability and so the strain at failure should be understood as the strain at which there is an unacceptable increase in permeability. Assessment is difficult.

Changes in cement formulation / chemistry

Cement manufacturers are seeking to reduce the carbon footprint of their cements, for example, by modifications to manufacturing processes and the addition of materials such as ground granulated blastfurnace slag, fly ash and limestone powder to their products as supplied. Cements containing these additions are typically formulated for use in structural concrete but high water/cement ratio cut-off systems can be more sensitive to changes in cement formulation than structural concretes and the impacts of changes are unlikely to have been considered by the manufacturer (except for special cements). Experience derived from the behaviour of low water/cement ratio concrete mixes can be a poor indicator of the performance of high water/cement ratio cut-off and grout mixes.

Materials such as ground granulated blastfurnace slag and fly ash are regularly used to modify the fluid and set properties of cut-off materials. However, it is important to note that:

- Portland type cements, as supplied from the manufacturer, may contain significant amounts of one of the following: blastfurnace slag, fly ash and limestone powder. Allowance must be made for this when formulating blended systems;
- supplies of fly ash are becoming limited as coal burning power stations are closed to reduce carbon dioxide emissions.

RATE OF MOVEMENT OF CHEMICAL REACTION FRONTS

When a barrier material is subjected to permeation there may be chemical interactions between the material and species in the permeant and importantly there also may be dissolution of the barrier material by the permeant. Many of the chemical reactions associated with damage to barrier materials are relatively fast, for example, compared with the transit time through a 600 mm thick barrier of permeability 10^{-9} m/s or a 5 mm GCL with a permeability of 10^{-11} m/s. It follows that the rate of supply of reactants is typically the controlling factor for degradation or other chemically induced change of behaviour of barrier materials and not the underlying rate of the chemical reaction. In a permeability test the rate of supply of reactant is controlled by the applied hydraulic gradient and the overall permittivity of the test specimen with a contribution from diffusion.

Jefferis (1992; 2003) used a simple single reaction/dissolution/process model to derive a formula for the overall permeability of a sample as a function of the time under permeability as a reaction front moved through the sample (for permeation alone excluding diffusion). From the model, the overall permeability, k_o of a specimen at time, t ($t < t_f$) was derived as:

$$\frac{1}{k_o^2} = \frac{1}{k_u^2} + \left(\frac{1}{k_r^2} - \frac{1}{k_u^2}\right)\frac{t}{t_f} \qquad [1]$$

and the time, t_f for the reaction front to penetrate through the whole specimen was:

$$t_f = \frac{(V_t+n)d^2}{2H}\left(\frac{1}{k_u} + \frac{1}{k_r}\right) \qquad [2]$$

Where V_t is the volume of permeant required to fully react with (or dissolve species from) one cubic metre of material according to the process which changes the permeability from k_u (unreacted

© 2016 Deep Foundations Institute

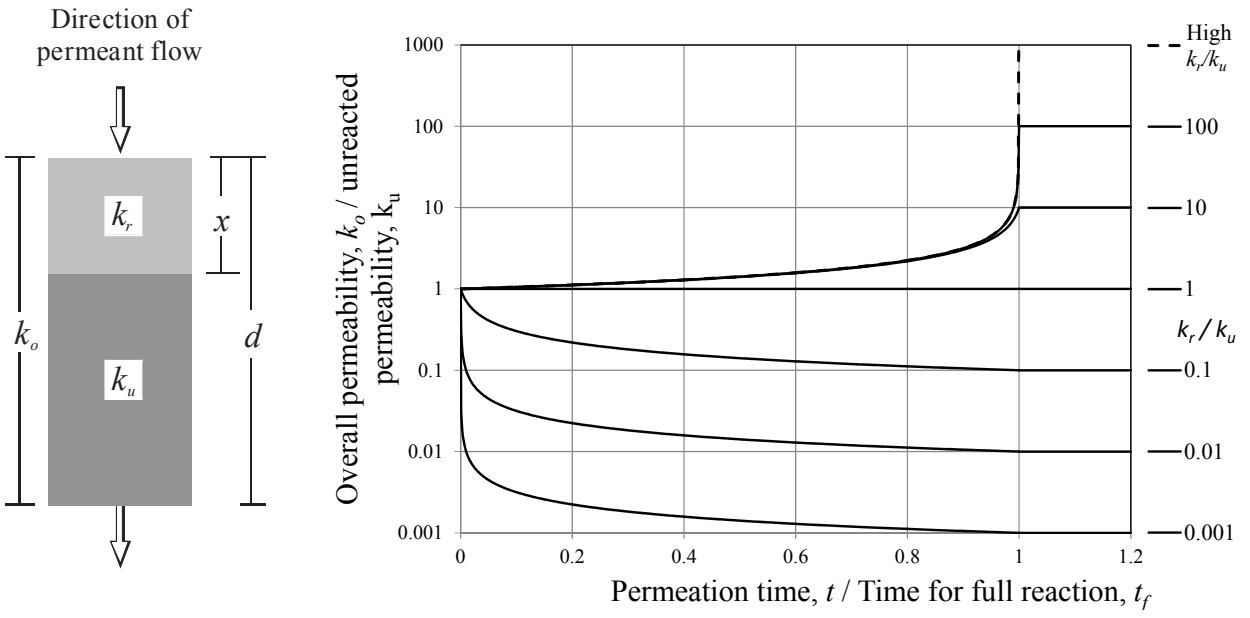


Fig. 1. Single reaction front model and predicted permeability as a function of time

permeability) to k_r (reacted permeability), H is the driving head of permeant and d the initial flow length through the specimen and n the porosity of the material after the reaction. It may be noted that $(1 + V_t /n)$ is thus the retardation of the reaction front and that for reaction rather than simple linear sorption processes, if such exist in reality, retardation is a function of the concentration of the reactive species in the permeant.

Damaging reactions

An interesting conclusion from the above, albeit very simple, model is that the permeability test gives effectively no information on the reacted permeability k_r of the specimen until breakthrough is almost achieved. For example, if:

$$k_r^2 \gg k_u^2 \left(1 - \frac{t}{t_f}\right)^{-1} \quad [3]$$

that is when $k_r^2 \gg k_u^2$ and until the permeation time t approaches but is still less than the time for full reaction t_f, then

$$\frac{k_o}{k_u} = \left(1 - \frac{t}{t_f}\right)^{-0.5} \quad [4]$$

Equation 4 is plotted as the dotted line in Fig. 1, it almost completely overlies that for 10 and 100-fold increases in permeability until t approaches and is very close to t_f. Thereafter the behaviour depends on the change in permeability induced by the reaction.

It follows that the observed permeability, k_o is independent of the permeability after reaction k_r until breakthrough is almost achieved if the permeability after reaction is as little as 10 times greater than the initial permeability ($k_r^2 \gg k_u^2$ x 100).

© 2016 Deep Foundations Institute

Thus when there is an increase in permeability, the situation of most concern for barrier performance, the test is not a measure of effect of the permeant on the specimen (i.e. an assessment of k_r) but of the permittivity of the remaining material.

However, a laboratory permeability test can give useful information if it is continued for a time that exceeds the required design life of the barrier in the field – scaled for the relative dimensions of the sample and the field barrier and the driving heads, noting that in Equation 2, for a constant head H, the time for full reaction is proportional to thickness squared. When scaling from laboratory to field conditions, it is important to make allowance for diffusion which will be significant at field timescales.

An interesting feature of the permeability-time plot when $k_r^2 \gg k_u^2$ x 100 is that from Equation 4:

$$\frac{dk_o}{dt} = \frac{k_u}{2t_f}\left(1 - \frac{t}{t_f}\right)^{-1.5} \quad [5]$$

and thus the time for full reaction t_f can be estimated from the slope of the plot when there is a single process resulting in a substantial increase in permeability.

Equation 5 also offers potential for an early indication of the time for full reaction. If, for a linear-linear plot of k_o versus time t, a tangent is drawn to the plot near time zero and extended to a permeability of $1.5k_u$ (an increase of $0.5k_u$) then the corresponding time will be an indicator of t_f, a result which can be updated as further data become available as the permeation time increases.

Reactions producing a significant reduction in permeability

For a reaction which produces a significant reduction in permeability and for times t where:

$$k_r^2 \ll k_u^2\, t/t_f \quad [6]$$

the model gives

$$\frac{k_o}{k_u} = \frac{k_r}{k_u}\left(\frac{t}{t_f}\right)^{-0.5} \quad [7]$$

and thus no information can be obtained on the final permeability of the material until the reaction front has propagated through the whole of the test specimen.

MEASURED PERMEABILITY OF CEMENT-BENTONITE MATERIALS AS A FUNCTION OF TIME

Laboratory tests on samples of cement-bentonite typically show permeability reducing with time under test, for times under test from a few weeks to over a year. Typically the laboratory data can be fitted by a relationship of the form:

$$\frac{k_2}{k_1} = \left(\frac{t_2}{t_1}\right)^{-n} \quad [8]$$

Where k_1 and k_2 are permeabilities at times t_1 and t_2. The exponent, n typically has a value in the range 1 to 2.2. Whereas Equation 7 although not strictly comparable, might imply a value of 0.5 at longer times.

© 2016 Deep Foundations Institute

Figure 2 shows a typical permeability-time plot for a ground granulated slag-cement-bentonite cut-off wall mix continued to over 300 days. The reduction of permeability with time is often attributed to continuing hydration of the cut-off material. Whilst continuing hydration may make a contribution it is possible that there was also chemical reaction, for although distilled water was used as the permeant, the water was pressurised with compressed air without a gas impermeable membrane between water and the pressuring gas. The permeant water therefore will have contained some dissolved carbon dioxide from the air (though at low concentrations, natural waters if used as permeants are likely to have much higher bicarbonate ion concentration). This will have led to carbonation of calcium hydroxide in the cut-off mix to precipitate calcium carbonate at the pore walls / in the pore spaces, a reaction which is recognised as slowing the carbonation of structural concrete by pore blocking.

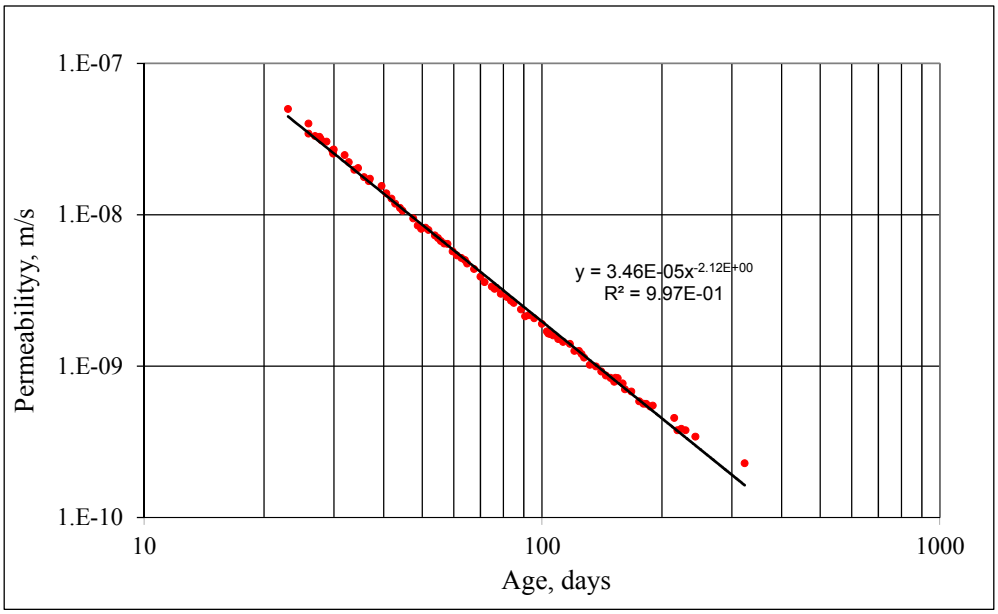


Fig. 2. Permeability-time plot for a ground granulated blastfurnace slag-cement-bentonite mix with best fit line

It is important to note that carbon dioxide may diffuse through the confining membrane in a triaxial-type permeability set-up. This will produce more rapid carbonation of the specimen and hence reduction in the measured permeability if the cell water is not de-aired and / or if the cell is pressurised via compressed air without a gas impermeable membrane between air and water.

A single reaction such as carbonation could lead to an apparently power law relationship as shown in Equation 7, for if Fig. 1 is re-plotted as a log-log plot it can be seen that there is an approximately linear relationship between the logarithm of the permeability and the logarithm of time – though interestingly the plots show a slight downward trend with time whereas the laboratory data show a slight upward trend so that a log-fit slightly under-predicts permeability (i.e. over-predicts permeability reduction at any time).

Further evidence for reaction process(es) contributing to permeability reduction can be drawn from the observation that samples under continuous permeation typically show faster reduction in permeability than those cured under water and tested only at late ages.

© 2016 Deep Foundations Institute

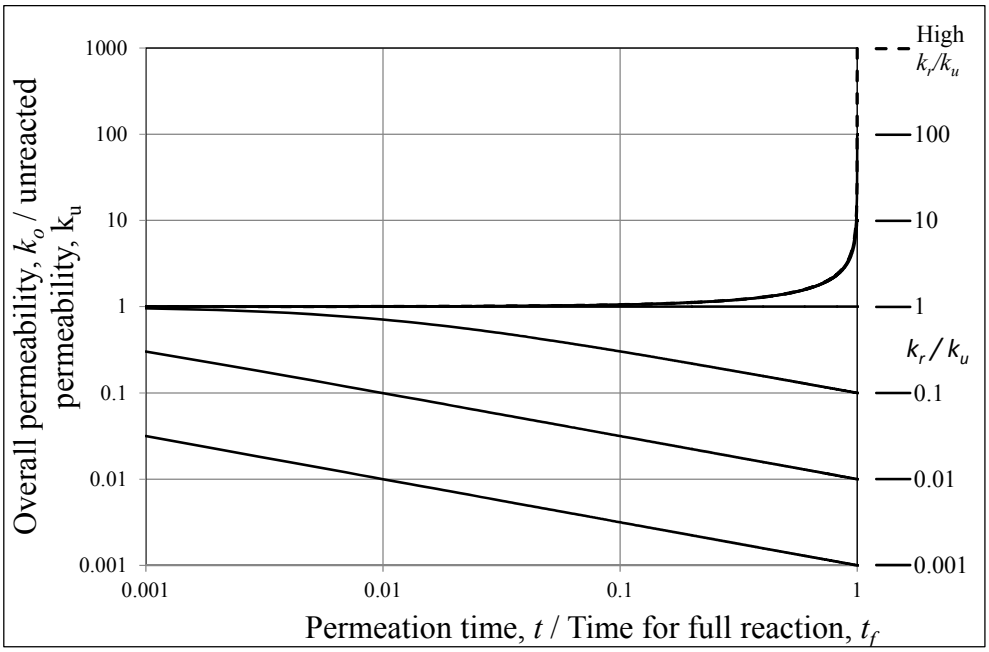


Fig. 3. Fig. 1. with Time axis plotted to a logarithmic scale.

CAVEATS

The model set out above and the derived equations not only assume that there is a single reaction front but also that it progresses uniformly through the test specimen. This type of behaviour is regularly seen for laboratory permeability tests on cement-bentonite materials. However, under certain reaction conditions the front may finger so that the flow is concentrated in a small region leading to much more rapid advancement of the reaction front, an issue amenable to analysis but outside the scope of this paper.

Whilst the analysis presented above gives useful insights into what can and cannot be deduced from a single reaction front model, the challenge with real systems is that there will be many simultaneous processes in play. Some of these, for example, carbonation of cement-bentonite materials, may produce a reduction in permeability so masking permeability increasing processes unless testing is continued for very long times. The analysis of multiple reaction front processes is beyond the scope of this paper.

BARRIERS FOR DAMS, DYKES AND LEVEES

For many impoundments retained by cement-bentonite barrier materials, the most reactive species in the water may be dissolved anions such as sulfate and bicarbonate which can react with calcium species in the cementitious material, though at the concentrations typical of many reservoir waters the sulfate contents may be sufficiently low that no precipitation occurs.

Cations such as calcium and magnesium that can affect the dispersion of bentonite are often assumed to be damaging but will have minimal effect because in cement-bentonite cut-off materials with significantly more cement than bentonite in the mix, the alkaline conditions will have removed or altered the bentonite to other minerals (Savage, 2011, Oda et al., 2004, Jefferis, 2012). Magnesium ion can of course damage the cementitious materials.

In addition to carbonation of the bulk of cement-bentonite materials during permeation by waters containing inorganic carbon species such as bicarbonate, carbonation can very effectively seal small

© 2016 Deep Foundations Institute

cracks in cementitious barrier materials – the white deposits sometimes seen at cold joints in retaining walls are a manifestation of such sealing by carbonation.

Jefferis (2001) reports long-term permeability tests on a cement-bentonite material deliberately designed to be of high permeability (by adding the bentonite to the mix water and then adding the cementitious materials, Portland cement and ground granulated blastfurnace slag almost immediately afterwards so that the bentonite had minimal time to swell). The tests showed that there is significant leaching of alkaline materials by water alone.

CHEMICAL REACTIONS AND MASS BALANCES

A major problem with the assessment of the durability of barrier materials is the identification of the chemical reactions that drive physical changes. As a result, many studies of attack on barrier materials focus on physical effects with minimal attention to the underlying chemistry.

However, if the chemistry of the reactions that can occur between the barrier material and the permeant is not known then it is not possible to undertake a mass balance to determine the degree of reaction achieved per unit volume of permeant and in the absence of such information it is not possible to model the rate of advance of the reaction front through a barrier – except possibly by direct observation in permeability tests (preferably taken to completion, i.e. penetration of the reaction front through the whole sample). Furthermore, if the chemistry is not studied significant reactions may be missed.

An indication of the chemical reactions that are occurring can be obtained by analysis of the immersion solution after immersion tests, especially if this solution is regularly replaced with fresh solution. It is of course essential that a record is kept of the liquid: solid ratio for each immersion and that the immersion solutions are well characterised before as well as after immersions.

It is important to add a caveat to this discussion of chemistry. Knowledge of the chemistry of a reaction will not provide information on its effect on the permeability of a material except in the more extreme cases, for example, where there is a substantial loss of volume by dissolution.

MODELLING

For nuclear repositories where the timescales are millennia, an understanding of reaction processes becomes essential and this has led to the development of sophisticated modelling tools. However, geochemical modelling appears to have been rather little used in barrier durability assessments outside the nuclear repository industry. There are now geochemical databases that are internally consistent for Portland type cements, bentonite and rock minerals (Giffaut et al., 2014) which can be used in conjunction with computer codes such as PHREEQC (Parkhurst and Appelo, 2013). These codes can provide useful insights into some of the basic processes of cement maturation such as carbonation and sulfate reaction with an indication of the evolution of mineral products appearing / disappearing as reaction processes evolve. Geochemical modelling of cement-bentonite mixes indicates that minerals produced by alteration of bentonite vary depending on alkalinity and other chemical conditions. Zeolites such as analcime are likely (Oda et al., 2004) but sheet silicate minerals may also form. The variety of alteration minerals might have implications for durability of the barrier system. It is hoped that geochemical modelling coupled with mineralogical and petrological examination of materials exhumed from barriers during service will lead to new insights on material selection and service life prediction.

The validity of geochemical modelling for assessing the durability of these materials is dependent on assumed data for reaction kinetics and thermodynamic properties. There are significant uncertainties in values for these data in relation to chemical compositions of solids and pore waters. Modelled

© 2016 Deep Foundations Institute

geochemical and mineral evolution then needs to be coupled with the distribution of porosity and of other material properties that affect cement-bentonite performance. Coupled modelling is at an early stage of development for these types of applications.

Alkaline alteration of the bentonite component and the timescale over which this occurs depends on the leaching and movement of solutes through the cement-bentonite mixture. Homogeneous diffusive solute movement will produce alteration that is different in terms of both timescale and spatial distribution from heterogeneous advective movement in fissured material. The results of experiments and modelling indicate that there is mass transfer, and associated porosity change, from cementitious solids to newly-formed minerals associated with bentonite particles. Assumptions about the mechanism of bentonite dissolution, the resulting alteration minerals, and the physical and geochemical factors involved are sources of uncertainty in geochemical modelling (Watson et al., 2009). Laboratory experiments and studies of natural systems have provided insights of how the products of bentonite alteration are determined by the stage of cement leaching and the pore water-to-solid ratio (Savage et al., 2007). Other factors that affect the pattern of alteration are the ion compositions on bentonite exchange sites and accessory minerals in cement and bentonite. Sensitivity to these uncertainties can be assessed with existing geochemical and reactive transport models; likely developments of coupled models will extend capabilities further towards the potential impacts on material properties.

CONCLUSIONS AND RECOMMENDATIONS

a) Permeability (hydraulic conductivity) test results with potentially reactive chemicals are often presented solely to show that there was no significant increase in overall permeability within the timescale of the laboratory tests. No attempt is made to identify what was happening within the test specimen or that the test was continued for sufficient time taking account of the chemistry.

b) Test results are seldom interrogated using either reaction models such as that shown in Fig. 1 or by chemical modelling. Physical impacts may be identified but chemistry is largely ignored. As a result the understanding of barrier performance may be little advanced and useful insights likely lost.

c) The assessment of time effects is fundamental to durability assessment. Without some assessment of the time(s) for full reaction, and/or scaling to field conditions, tests may be too short to identify processes which will dominate service life in the field.

d) This paper shows that further information can be obtained from permeability test results if they are interrogated using reaction models such as that shown in Fig. 1 and/or by chemical modelling.

e) At the present time specifications for cut-off walls and other barriers for water retaining structures and the containment of contamination typically require the contractor to design a system to provide a barrier effective against challenges from an array of chemical species. Little or no time is allowed for laboratory compatibility testing and yet service lives of decades may be specified. Whilst it may be possible to identify underlying chemical reactions (especially with the help of geochemical codes) without laboratory testing it will be almost impossible to assess the impacts of the reactions on permeability and other barrier properties. The problems will be particularly acute when the only data in the literature on relevant chemicals show damage but at unrealistically high concentrations or no damage at unrealistically low concentrations.

f) Back-analysis: some cement-bentonite barriers and soil-bentonite barriers have been in service for over 60 years. It is time for a concerted programme of sample exhumation and physical, chemical and mineralogical examination.

© 2016 Deep Foundations Institute

REFERENCES

Cernak, J., Evans, J.C. and Tamaro, G.J., 2012. Evaluation of soil-cement-bentonite wall performance – effects of backfill shrinkage, Proceedings of the Fourth International Conference on Grouting and Deep Mixing, Deep Foundations Institute and Geo-Institute of the American Society of Civil Engineers, New Orleans, pp502-511.

Evans, J.C. and Jefferis, S.A., 2014. Volume change characteristics of cutoff wall materials, Proc. 7th International Congress on Environmental Geotechnics, Melbourne.

Giffaut, E., Grivé, M., Blanc, Ph., Viellard, Ph., Colàs, E., Gailhanou, H., Gaboreau, S., Marty, N., Madé, B., Duro, L., 2014. Andra thermodynamic database for performance assessment ThermoChimie. Applied Geochemistry, 49, pp225-236.

Jefferis, S.A., 1992. Contaminant - grout interaction, ASCE Specialty Conference, Grouting, Soil Improvement and Geosynthetics, New Orleans, February, pp1393-1402.

Jefferis, S.A., 2001. Permeability a dynamic property of barrier materials, International Containment and remediation technology conference and exhibition, Orlando, Florida, June.

Jefferis, S.A., 2003. Long term performance of grouts and the effects of grout by-products, ASCE Specialty Conference, Grouting and Ground Improvement, New Orleans, February, pp1141-1152.

Jefferis, S.A., 2012. Cement-bentonite slurry systems, Proceedings of the Fourth International Conference on Grouting and Deep Mixing, Deep Foundations Institute and Geo-Institute of the American Society of Civil Engineers, New Orleans, pp1-24.

Oda, C. Honda, A. and Savage, D., 2004. An Analysis of Cement-Bentonite Interaction and Evolution of Pore Water Chemistry in Metcalfe, R. and Walker, C. (eds), Proceedings of the International Workshop on Bentonite-Cement Interaction in Repository Environments, Tokyo, Japan, ppA3-74-A3-79.

Parkhurst, D.L., and Appelo, C.A.J., 2013. Description of input and examples for PHREEQC version 3—A computer program for speciation, batch-reaction, one-dimensional transport, and inverse geochemical calculations: U.S. Geological Survey Techniques and Methods, book 6, chap. A43, 497 p., available only at http://pubs.usgs.gov/tm/06/a43/.

Savage, D., Walker, C., Arthur, R., Rochelle, C., Oda, C. and Takase, H., 2007. Alteration of bentonite by hyperalkaline fluids: A review of the role of secondary minerals. Physics and Chemistry of the Earth, 32, pp287-297.

Savage, D., 2011. A review of analogues of alkaline alteration with regard to long-term barrier performance. Mineralogical Magazine, 75 (4), pp2401-2418.

Tedd, P., Holton, I. R., Butcher, A. P., Wallace, S. and Daly, P. J., 1997. Investigation of the performance of cement-bentonite cut-off walls in aggressive ground at a disused gasworks site. Proc. of the 1st Int. Containment Technology Conf. St Petersburg, Florida, USA. pp 125-132. Also published in Land Contamination & Reclamation Vol 5, No 3, pp217-223.

Watson, C., Hane, K., Savage, D., Benbow, S., Cuevas, J. and Fernandez, R., 2009. Reaction and diffusion of cementitious water in bentonite: Results of 'blind' modelling. Applied Clay Science, 45, pp54-69.

© 2016 Deep Foundations Institute

DIAPHRAGM WALLS FOR SEEPAGE CONTROL AND REMEDIATION: THREE CASE HISTORIES

Raffaella Granata, Trevi S.p.A, Italy, +39-3479727588, rgranata@trevispa.com
Mario Mauro, Treviicos, MA, USA, 617-241 4800, mmauro@treviicos.com
Fabio Santillán, Treviicos, MA, USA, 617-241 4800, fsantillan@treviicos.com

ABSTRACT

This paper presents the utilization of slurry walls methods for the recent remediation of three high hazard active dams in the United States. Tuttle Creek Dam, located in Manhattan, KS, required seismic retrofit. To this extent, transverse shear walls were installed on the downstream slope using self-hardening slurry (SHS) to reinforce the liquefiable foundation soils. Herbert Hoover Dike, located around Lake Okeechobee, FL, is subject to potential failure because of internal erosion, piping and slope instability at high water levels. The innovative use of the hydromill equipment with SHS allowed the successful construction of the cutoff wall through heterogeneous soils and rock. At Bolivar Dam, located in Bolivar, OH, under-seepage negatively affects the structural stability of the dam, and a partial seepage barrier wall is required through the upstream slope of the dam. In this case, the Contractor selected the plastic concrete seepage barrier, excavated by a combination of clamshell and hydromill equipment. The main features of the three projects are summarized in Table 1.

Keywords: Cutoff Wall, Seepage Barrier, Self-hardening Slurry (SHS), Plastic Concrete

Table 1. Main Features of the Three Projects

	Tuttle Creek, KS Shear Walls		Herbert Hoover Dike, FL Task Orders 3, 7G, 8H Cutoff wall		Bolivar Dam, OH Seepage Barrier	
Total length	4801 m	15,750 ft	11,659 m	38,250 ft	1377 m	4519 ft
Max depth	24 m	80 ft	26 m	84 ft	44 m	144 ft
Total area	117,058 m^2	1,260,000 ft^2	238,640 m^2	2,568,700 ft^2	55,277 m^2	595,000 ft^2
Thickness	1.2 m	4 ft	0.6 m	2 ft	0.9 m	3 ft

TRANSVERSE SHEAR WALLS AT TUTTLE CREEK DAM, KS

Introduction

Tuttle Creek Dam was constructed by the U.S. Army Corp of Engineers (USACE) on the Big Blue River in North-East Kansas in the 1950s for flood control. Tuttle Creek Dam is a zoned earthfill embankment, 7,500 ft long and about 137 ft high with a crest width of 50 ft (Fig. 1). It is located in an area of moderate seismicity associated with an old continental rift zone located just to the east of the dam. The USACE, under the Dam Safety and Assurance Program, determined the need for remediation. Site investigations and geotechnical analyses identified the upper alluvial foundation sands as being susceptible to liquefaction when subjected to the Maximum Credible Earthquake.

In 2005, USACE awarded a construction-manager-at-risk/early Contractor involvement contract. The USACE and the Contractor closely cooperated to identify the most effective technologies, materials and configurations to be used in the remediation. The original remediation design required both the upstream and downstream slope stabilization of liquefiable foundation. Results of refined seismic deformation analyses revealed that the stabilization of the downstream slope and toe was needed to protect the downstream seepage control system and to provide assurance that a liquefaction-induced flow slide would not occur (Stark et al., 2012).

© 2016 Deep Foundations Institute



Fig. 1 Aerial View of Tuttle Creek Dam and Worksite

Initially, downstream slope stabilization was planned to be performed by jet grouting. However, given the technical concerns (proximity of the horizontal sand drain in the downstream dam shell) and economics, another technology was sought. Ultimately, the construction of 351 transverse shear walls was selected as appropriate to reinforce the liquefiable foundation soils. The shear walls had to be installed through the embankment and foundation soils near the downstream toe of the dam. They are oriented perpendicular to the crest of the dam, to resist deformations induced by slope instability following the design seismic event. Indeed, deformations at the downstream toe may jeopardize the fragile pressure relief well system, which provides essential under-seepage pressure relief during operation of the reservoir. Typical shear wall is 45 ft long, 4 ft wide, 62 to 80 ft deep, spaced at 14 ft on center along the downstream toe for a replacement ratio of about 29%. The permeable portion of soil between the walls is vital to allow continued through seepage in the drain and foundation soils, so as to control the pore pressures.

SHS Mix Design and Transverse Shear Wall Construction Method

The transverse shear walls were installed using self-hardening slurry (SHS). The standard SHS is made of water, bentonite and cement. Using the SHS, the wall is constructed by the single-phase method: (1) the trench is excavated to the full depth under the SHS, which operates as stabilizing fluid; (2) when the excavation is completed, the SHS sets and hardens; (3) after setting, the self-hardening slurry acts as a backfilling material, providing strength and waterproofing.

The USACE performed seismic deformation analysis, using a calibrated FLAC model, to evaluate the impact of the transverse shear walls on downstream slope deformation (Stark et al., 2012). Based on these analyses, to achieve the desired structural capacity of the walls, USACE specified that the completed walls had to be characterized by an unconfined compressive strength (UCS) $\geq$ 300 psi. This UCS value is an unusually high value to be obtained by SHS.

Therefore, several mix designs were studied and tested to obtain a SHS able to meet the required performances criteria. The grout mixes' components and their relative proportions were selected after an extensive laboratory test: the water was from a site well; the cement was a 50/50 blend of Portland and blast furnace slag; the bentonite was a Wyoming type. The relatively high cement content, necessary to comply with the UCS requirement, required the use of a special admixture to simultaneously grant adequate viscosity, cohesion, and stability (both vs. bleeding and filtration under pressure) during the whole excavation phase. Four SHS mixes were trialed also in field tests, to assess the workability and stability of the SHS, the stability of the trench and the ultimate strength. Finally, the selected SHS was characterized by 0.5 cement/water, 4.5% bentonite/water, and 1.3% of admixture (all ratios by weight).

A work platform was constructed on the downstream slope of the dam by removing the existing fill (to expose the pervious drain fill), and placing sand road sub-base for a working surface, at elevations varying

© 2016 Deep Foundations Institute

between 1038 and 1043 ft ASL. The depth requisites were met by reaching the bottom elevation (El. 978 ft ASL). Foundation soils to be excavated below the drain of the dam were typical sequences of alluvial deposits consisting silt and clay, underlain by sand deposits with various contents of silt, gravel and lenses of clay. Typical stratigraphy of the area is presented in Fig. 2.

The transverse shear walls were excavated by means of a mechanical clamshell supported by a hydraulic crawler crane. Each wall was installed by excavating three 13-ft-long primary bites, followed by two overlapping secondary bites (Fig. 2). A special temporary steel frame was used to guide the clamshell into the trench being excavated.

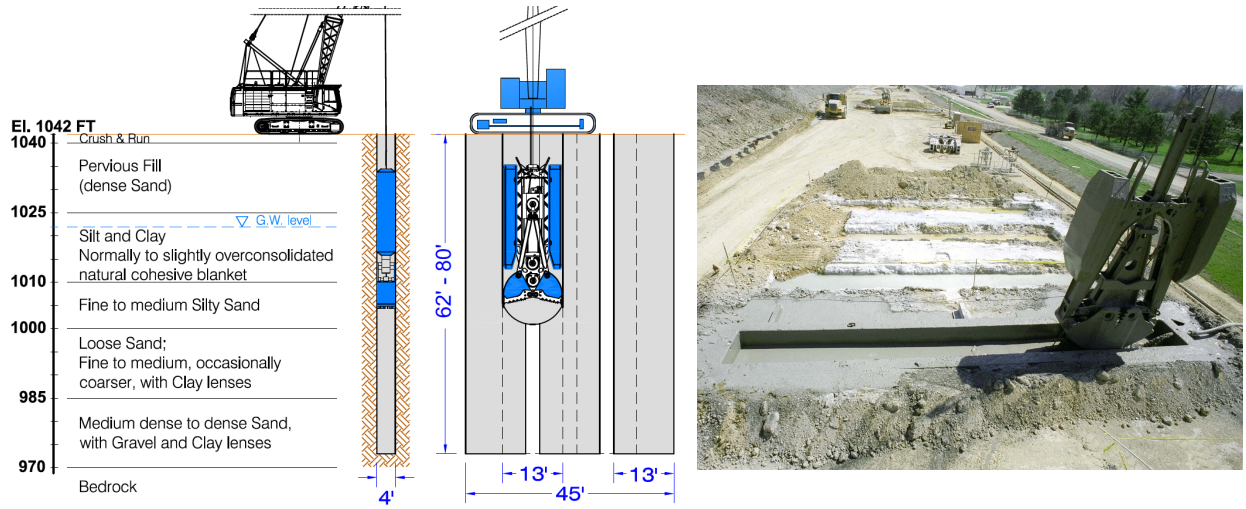


Fig. 2 Site soil profile; Wall size and construction sequence; Wall being excavated by clamshell

The SHS was prepared at the plant facility and pumped to the trench via steel pipes. The water-bentonite mud was prepared in a dedicated high speed mixer, and allowed to hydrate for 8 to 12 hours. The admixture and cement were then mixed with the bentonite slurry, in high speed mixers. All through the construction of the transverse shear walls, the SHS was maintained under control by testing its density, viscosity and stability. Samples were routinely taken both at the mixing plant and from the trench. Cylindrical specimens were prepared and delivered to the laboratory to determine the strength characteristics after curing periods of 28 and 60 days.

Acceptance criteria established in the technical specifications were solely based on the results of the verification coring, and of the UCS tests performed on cored specimens at a minimum of 90 days of age. The requirements of the QC testing program led to the perforation of 101 coring holes. As stated in the technical specifications, each coring had to demonstrate a core recovery ≥ 90% for each run, and no soil inclusions larger than 3 inches in any direction. At least 21 samples for each coring had to be tested for UCS; the required average UCS on core samples was ≥ 300 psi, with at least the 90% of individual UCS tests ≥ 200 psi. The overall average measured UCS on cored samples was 357 psi. A typical coring run is shown in Fig. 3.



Fig. 3 Tuttle Creek Dam. Core Run in the Transversal Shear Wall

© 2016 Deep Foundations Institute

In addition, slurry permeation borings were performed just outside the wall limits to determine whether or not permeation would impact under-seepage flows. In general, these borings showed global permeation of SHS into the surrounding soil of only a couple of inches or less throughout the depth of the walls, with farther permeation along isolated coarse grained lenses (Stark et Al., 2013).

Summary and Conclusions

The system adopted proved that the SHS trenching procedure could be very effective in the foundations soils encountered at the site. Moreover, transverse shear walls constructed using slurry trench techniques and SHS appear to be an efficient technique for slope stabilization in a liquefiable foundation, if under-seepage flows are not inhibited (Stark et Al., 2013).

SHS CUTOFF WALL AT HERBERT HOOVER DIKE, FL

Introduction

The Herbert Hoover Dike (HHD) is a 143-mile-long earthen embankment around Lake Okeechobee, FL. The levee was originally constructed by farmers in the early 1900s for flood protection. Severe hurricanes caused massive flooding and loss of life, and the levee system was upgraded in several steps, from the late 1920s to the 1940s. In 2007, the USACE placed HHD on the top 6 list of dams in the nation needing repair, since it was recognized as subject to potential failure because of internal erosion, piping and slope instability at high water levels. In the same year, the USACE awarded three Multiple Award Task Order Contracts for the remediation in the most vulnerable section (Reach 1), which extends from Belle Glade to Port Mayaca, for a total of about 22 miles.

The rehabilitation project required the construction of a cutoff wall within the levee to eliminate the existing seepage through the levee foundation and limestone layers (Fig. 4). The USACE provided performance specifications and acceptance criteria for the cutoff wall. The Contractors were allowed to select the installation method.

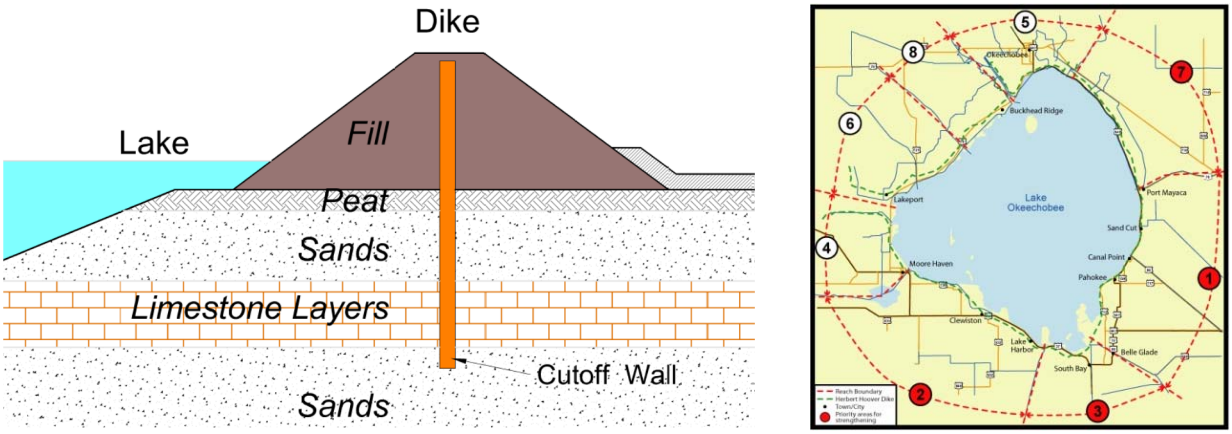


Fig. 4 Herbert Hoover Dike – Rehabilitation Concept and Soil Profile (left). Location (right)

SHS Mix Design and Cutoff Wall Construction Method

In Task Order 3, Treviicos proposed the SHS method for the cutoff wall, taking in consideration that the presence of the limestone layers required the use of the Hydromill. Usually, the SHS method is performed by excavating by clamshell or long reach excavator. In normal conditions, due to the unique slurry circulation system, the Hydromill cannot work under SHS, due to the cement in the slurry.

© 2016 Deep Foundations Institute

Nevertheless, the proposed construction procedure foresaw:

(1) the use of hydraulic and mechanical clamshells to completely remove upper soils including peat, topsoil and miscellaneous fill material;
(2) followed by a Hydromill cutter to excavate to a final depth of about 80 feet through limestone of variable continuity and hardness (UCS up to 14,000 psi).

The Contract Documents required the installation of a 500-ft Demonstration Section in order to verify that the means and methods were adequate to properly construct the cutoff wall in accordance with the performance requirements of the contract. Following the approval of the Demonstration Section, the installation of the cutoff wall started. After completion of Task Order 3, two further Task Orders (7G and 8H) were completed by Treviicos with the same construction method.

The cutoff wall was installed from a temporary work platform (Fig. 5) constructed on top of the existing dike. The work platform moved with cutoff wall construction as it progressed. The cutoff, 2-ft thick, was installed within concrete guide walls, which were necessary to lead the excavation tool and provide a fixed reference for panel location, depth and verticality measurements.

The "Panel Method" was used to construct the continuous cutoff wall, with overlapping primary (25 to 30 ft long) and secondary panels (9.2 ft long). The Hydromill inclinometer provided verticality information in real time and allowed for corrections of any deviations if necessary. Cutoff wall centerline, panel locations (joints in between primary and secondary panels) and positioning of verification borings were all located using GPS.

The SHS was transported from the batch plant to the trench by concrete mixers, which continuously supplied the trench as the excavation advances. The material excavated by the clamshells was dumped directly into off-road trucks and disposed offsite. With the system adopted at HHD, as the Hydromill cutter excavates the limestone and sand, the SHS loading the cuttings is circulated through a screened desanding unit, which removes rock particles in excess of ¼ inch. After passing through the desander, the SHS is gravity fed back into the trench. At the completion of the excavation, the slurry hardens and becomes the wall backfill material.

In this process the SHS works in unique condition, since it encapsulates sand and fines, and quickly increases its density and viscosity as soon as the Hydromill starts the excavation. Hence, the SHS mix had to be carefully designed, taking into account both the special installation needs in terms "workability" and specification requirements in terms of strength and permeability.



Fig. 5 Overview of Herbert Hoover Dike Worksite

© 2016 Deep Foundations Institute

As part of the construction quality control and quality assurance (QC/QA) testing, the SHS produced at the mixing plant was constantly monitored before being sent to the site (temperature, density, bleeding and Marsh viscosity), and two samples per shift were collected for UCS and permeability testing. In addition, for every 200-ft interval, in situ samples were grabbed within the wall, at 35, 55 and 70 ft depth, to be tested for 7 and 28-day UCS (ASTM 2166) and permeability (ASTM 5084), (Fig. 6).

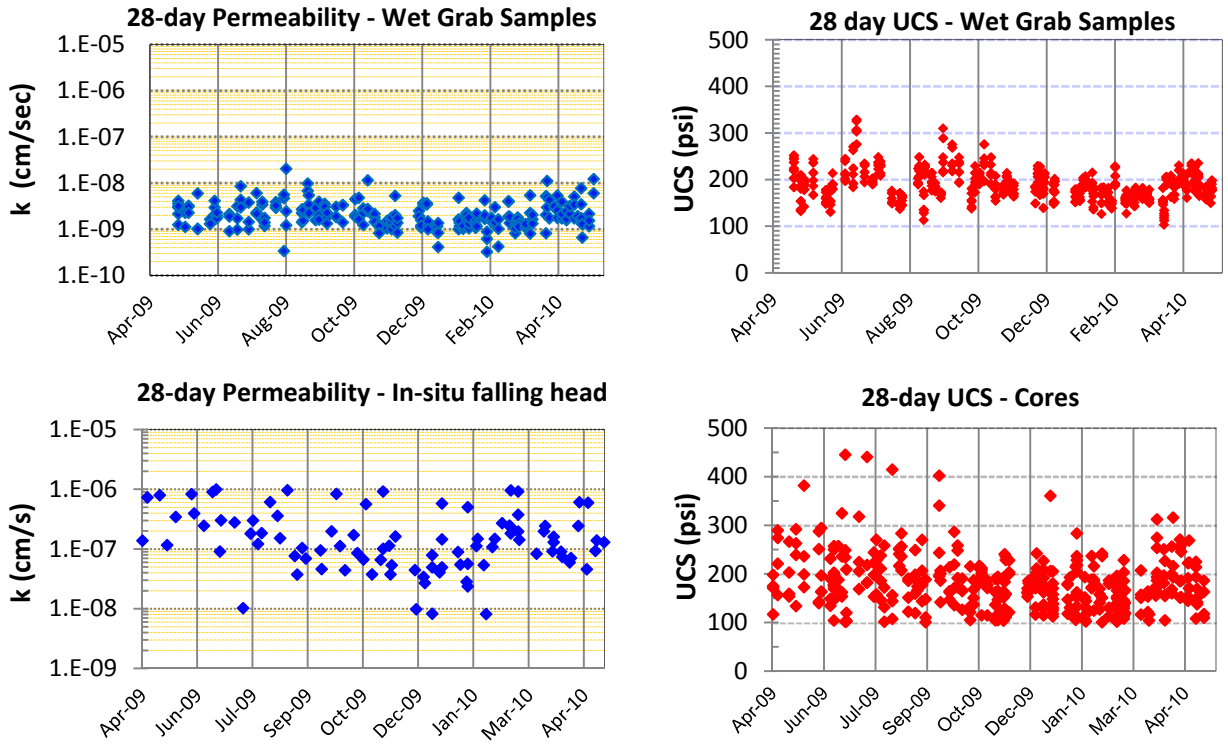


Fig. 6 Herbert Hoover Dike, Reach 1, Task Order 3. Permeability and UCS Lab and In-Situ Tests

The cutoff wall was validated via a combination of onsite and laboratory testing performed on verification borings 28 days after panel installation. The verification borings were spaced approximately every 170-200 ft along the centerline of the cutoff wall, at center of panels and at joints between primary and secondary panels. The verification borings had to be cored from top of the wall, down to 2 ft above bottom of cutoff wall. Each coring had to demonstrate a core recovery $\geq 95\%$ for each run and provide the necessary samples for testing.

Rotary coring was performed using a wire line system, with a double tube type core barrel, and water as drilling fluid. A core drilling 4-inch diameter was selected to retrieve 2.5-inch diameter cores (Fig. 7). The verification borings were surveyed using a High Resolution Optical Televiewer (Hi-OPTV), to record a 360-degree view of the borehole surface. The OPTV record shows the image as a "flat core" with description of depth and orientation. All the collected data and info (from cores, logs, video-logging) were analyzed to certify that wall is homogeneous and continuous, without inclusions, voids and cracks.

The permeability characteristics were assessed by in-situ falling head test performed in one stage along the entire length of the verification boreholes, using transducers to record the water level versus time. The tests, performed 28 days after panel installation, had to confirm that a permeability of less than $1x10^{-6}$ cm/s was achieved.

The UCS characteristics were assessed on core samples (Fig. 6 and Fig. 8). At least four samples for each coring had to be tested for UCS, two of which from depth 10 to 20 ft, one below 30 ft, one

© 2016 Deep Foundations Institute





Fig. 7 HHD - Typical Cores and Core Specimen at Panel Joint

within 10 ft of the bottom. The Technical Specification required UCS ranging from 100 to 500 psi (evaluated by a 10-point moving average), with no value below 75 psi.

All controls and testing required to validate the cutoff wall were always witnessed by USACE representative. In addition to the acceptance criteria required in the specifications, a section of approximately 200 ft of wall was exposed (to a depth of 4 ft) to allow further direct inspection by the USACE. The wall exposed was continuous and homogeneous, without inclusions and/or open cracks (Fig. 8). Further information about this Project and tests are reported in Blackwell et al., 2010.





Fig. 8 Herbert Hoover Dike. From left to right: UCS testing at Panel Joint, Permeability test on wet grab samples; Lakeside of cutoff wall being inspected by the USACE.

Summary and Conclusions

Full replacement of the upper soils in the wall mitigated potential issues associated with the organic content and high degree of variability of those soils on backfill performance. Removal of the miscellaneous embankment fill materials from the wall and the use of a manufactured/engineered SHS permitted better control of the final wall backfill composition.

The Clamshell/Hydromill construction method under a carefully designed and tested SHS provided a homogeneous and continuous cut-off wall, which meets strength and permeability performance criteria, with good bonding between primary and secondary panels. Also, it was proved that the SHS trenching procedure (due to the SHS high density and cement content) is very effective in the foundations soils encountered at the site, whereas, for instance, the excavation under bentonite slurry may present stability

© 2016 Deep Foundations Institute

issues. The innovative use of the Hydromill cutter allowed successful construction of the SHS cutoff wall through heterogeneous materials and rock over 7.2 miles of dike, for a total of over 2,500,000 sqft of wall.

DEEP SEEPAGE BARRIER AT BOLIVAR DAM, OH

Introduction

Bolivar Dam was built by the USACE in the 1930s to provide flood control and water conservation in the Muskingum watershed in South-East Ohio. In 2014, the USACE awarded Treviicos a contract for the construction of a deep Seepage Barrier Wall (SBW) through the upstream slope of the dam on the basis of performance specifications. The purpose of the project was to address problematic seepage at Bolivar Dam, which negatively affects the structural stability of the dam resulting in increased risks to the public safety.

The foundation soil consists predominantly of sand with variable amounts of silt, and several gravel (both poorly and well graded) strata with variable thickness and extent. At the left abutment, the foundation bedrock includes siltstones and claystone, coal, shales, sandstones, and limestone units.

The contract required the construction of a partial-depth and partial-length seepage wall. The design of the SBW was developed by the USACE to ensure that the SBW is embedded a minimum of 10 feet into the dam's impervious core and includes the grouting of the left abutment of the dam. The SBW has a length of about 4518 ft, 735 ft of which are partially or fully embedded in the bedrock. The SBW depth ranges between a maximum of 144 ft and a minimum of about 25 ft (Fig. 9). As required by the Technical Specification, a campaign of integrative exploratory drilling was carried out in order to better define the top of bedrock surface and to allow a proper barrier key-in.

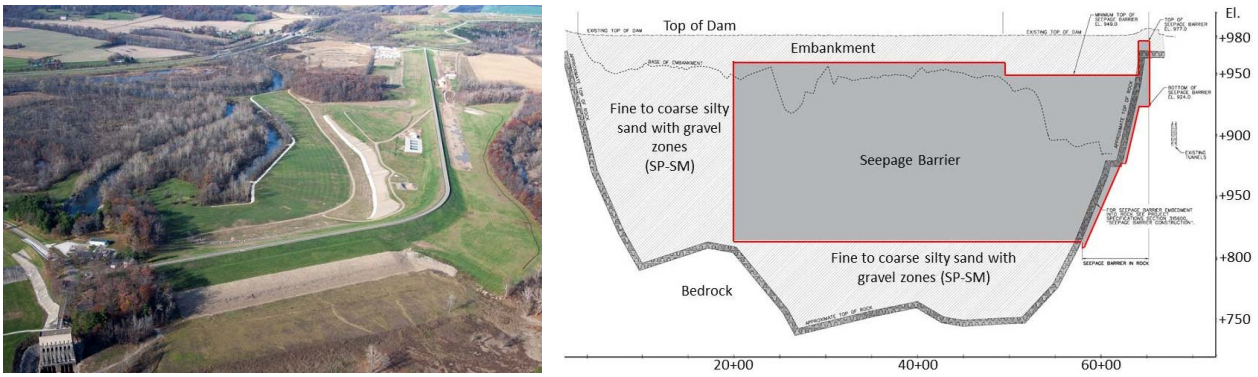


Fig. 9 Bolivar Dam. Aerial view (left) and SBW profile (not to scale)

The Technical Specification required the seepage barrier to be continuous, homogeneous, with an in-place permeability $\leq 1 \times 10^{-6}$ cm/s and an average UCS ≥ 750 psi (as defined by a 10-point moving average over consecutive tests on core samples), with no single test below 500 psi, at 28 days.

In its review of the solicitation documents, the Contractor considered three methods for the construction of the SBW, i.e., (1) deep mixing, (2) self-hardening slurry wall excavated in panels with clamshell and hydromill equipment, and (3) the plastic concrete panel wall method. Due to the depth of the SBW and the strict acceptance requirements (such as homogeneity, continuity, verticality, strength and permeability), the Contractor excluded the first two methods and selected the plastic concrete panel wall method.

© 2016 Deep Foundations Institute

The deep mixing method was disregarded because the Contractor believed that it would not guarantee the homogeneity and verticality required to meet the acceptance criteria established in the contract specifications, due to the considerable depth of the seepage barrier (144 ft). Moreover, deep mixing was not applicable in the portion of the seepage barrier where the embedment into the rock was required. Also the method successfully utilized at the Herbert Hoover Dike was excluded. Although this method allows verticality control and excavation of hard rock (with the hydromill equipment), it was not applicable for the construction of the SBW at Bolivar Dam, due to the high strength required for the seepage barrier backfilling material. In order to reach such strength, the composition of the self-hardening slurry would require a high cement content, which in turns would render the slurry unmanageable because of the high density, generated hydration heat, and the resulting short setting time. This factor would become critical for the seepage barrier at Bolivar Dam because of the excavation depth and the presence of rock, with unfortunate consequences, including a high risk of getting the excavation equipment permanently stuck and "cemented" in the trench.

Plastic Concrete Mix Design and Seepage Barrier Wall Construction Method

The construction of the SBW by was done by a combination of clamshell and hydromill equipment, and installation with plastic concrete panel wall method. With this method, the positive seepage barrier is composed of a series of rectangular shaped panels, each installed in two phases. In the first phase, each panel is excavated to full depth under bentonite slurry, in the second phase the panel is backfilled with plastic concrete placed by tremie method.

At Bolivar Dam, the clamshell was utilized to excavate the upper portion of the panel, where cobbles, boulders, and silty-clayey soil were present. The hydromill continued panel excavation through soil and rock embedment to the required depth. The hydromill inclinometer provided verticality information in real time and allowed for corrections for any deviations, if necessary. Afterward, the deviation from verticality and the actual geometry and location of each panel were accurately surveyed by several measurements with Koden ultrasonic echo-meter. All measurements were correlated and an extensive study of the geometry of the panels performed to ensure compliance with the geometry and continuity requirements of the design

The technical specifications required a minimum thickness of 24-in and an overlap of 6 in between adjacent panels at any depth. To comply with these requirements, the seepage barrier was composed of 36-in thick panels, with overlap at the work platform ranging from 12 in (for shallow panels) to 16 in (for 144-ft deep panels). Typically, the primary panels were excavated with a length of 30 ft, with multiple bites of the excavation tools (9.2-ft long). The secondary panels were excavated in a single bite. The stability of the 30-ft panel length, which was the maximum allowed, was verified during the demonstration phase of the project.

The interlocking between primary and secondary panels was obtained by the hydromill, by milling the edges of the two primary panels already constructed. Special care was spent in the mix design and cleaning of the bentonite slurry; special care was remarkably spent in the cleaning of joints, performed both with hydromill and special brushes before pouring.

The SBW panels were backfilled with plastic concrete, poured by tremie pipes. Plastic concrete was made of water, bentonite, cementious material, sand and aggregates. A series of studies and tests were performed to select proper components and tune the mix design, in order to obtain the best possible performances. A blend of blastfurnace slag and Portland cement was selected as cementious material. The aggregates were imported from controlled sources, in order to maintain consistent rheological properties of the concrete. Special additives were used to grant high slump (8 in to 10 in) and high slump retaining for the all the time necessary to complete the panel casting (some hours).

© 2016 Deep Foundations Institute

The validation of the SBW was accomplished by testing cored samples and by in-situ permeability tests. Therefore, the mix design was conceived taking into consideration that coring operations generally induce significant disturbance, both on the core samples and in the borehole itself. In turn, these disturbances tend to produce poorer results, both in terms of UCS and permeability, than those actually existing in-situ (Bruce and Mauro, 2010). Accordingly, to ensure the required UCS $\geq$ 750 psi from core samples (including joints), the "design" target UCS was 1300 psi; similarly, to ensure an in-situ permeability $k \leq 1 \times 10^{-6}$ cm/s during falling-head water tests, the "design" target permeability was $k \leq 1 \times 10^{-8}$ cm/s.

The plastic concrete was manufactured in a dedicated on-site plant. The plant included storage covered areas and silos for aggregate and cement, scales for dosing and batching liquid and powder components, electronic moisture tester, electric switch board and an electronic control panel. The bentonite mud was prepared in a separate high speed mixing plant. The pre-hydrated bentonite slurry was added and mixed to the other components. The plastic concrete homogeneity was assured by the efficient mixing in the robust twin shaft mixer. The temperature of the plastic concrete was always maintained in the range between 40° and 90°F, by regulating the temperature of water and bentonite slurry, and by heating the aggregate storage during the winter season. During pouring, the plastic concrete was routinely tested for slump and temperature. Additionally, for each panel twelve cylindrical specimens were cast, to be tested after 7, 14 and 28 days of curing for UCS (ASTM D4832) and permeability (ASTM D5084); (Fig. 10).

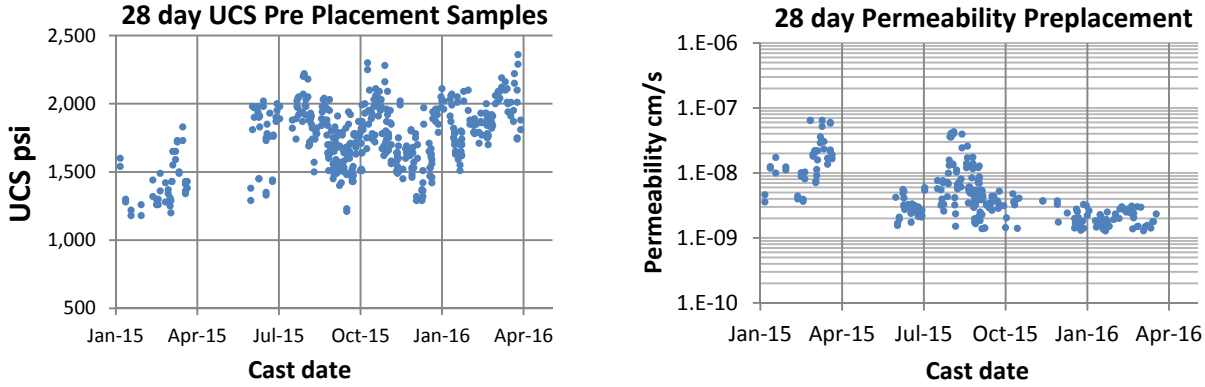


Fig. 10 Bolivar Dam. 28-day UCS and permeability of pre-placement samples.

The plastic concrete was delivered to the trench by means of truck mixers, at a rate of about 60-80 cy/hr, to allow for expeditious and proper concreting operations. The casting was accomplished using three sets of tremie pipes for the primary large panels, one set for secondary panels. The concrete placement was carried out without interruption, until the concrete has reached the required final elevation. During placement, the concrete level was checked by sounding the concrete depth both in the panel and tremie pipe, to control the actual panel volume versus the theoretical volume and consistency of poured concrete.

Within one day after backfill placement, the top of the seepage barrier was protected from cold and drying. Upon approval a section of the SBW, the work platform was reshaped and the upstream slope of the dam brought to its final configuration.

The acceptance of SBW was based on verification boreholes drilled by continuous coring a minimum of 28 days after panel installation. The verification borings were to evaluate the SBW continuity and homogeneity, to sample cores for UCS tests and to run in-situ permeability test. Verification boreholes were drilled vertical or inclined (20-22° from vertical) along the SBW centerline, from the top of the SBW to 2 ft above the SBW base. Several of the vertical boreholes were drilled full length at the joints, the inclined ones so as to intersect more than one panel and joint. The rotary coring was performed using a wireline system, with a triple tube type core barrel (6-in diameter), using water as drilling fluid.

© 2016 Deep Foundations Institute

Continuity and homogeneity of the in-place SBW was evaluated based on the quality of the recovered coring and video image survey performed by a High Resolution Optical Televiewer. All the collected data (from cores, logs, video-logging) were thoroughly analyzed to ensure the SBW is homogeneous and continuous, without inclusions, voids and/or cracks (Fig. 11).

For each core, at least five samples were selected for UCS testing. The samples were selected at various depth, from top to bottom (the deepest being within 10 ft from the panel base), and tested according to ASTM D4832.

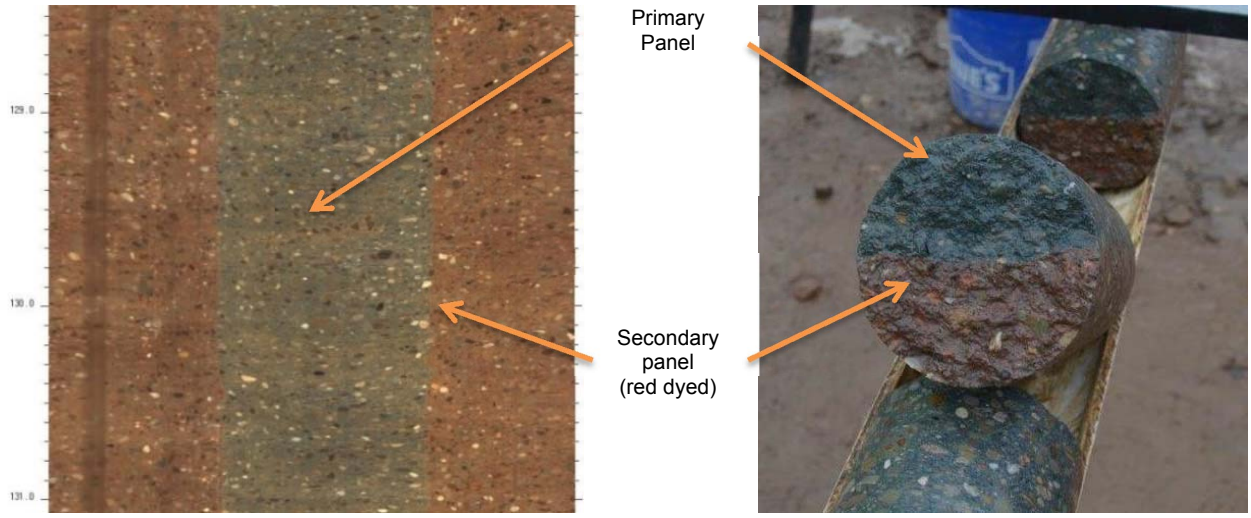


Fig. 11 Bolivar Dam. Left: Hi-OPTV at panel joint (detail at 128-131 ft depth); Right: Piece of Panel Coring. In both photos gray/green is the primary panel, red is the secondary panel (dyed).

All verification boreholes were tested for permeability. The tests were performed by falling head method, along the entire borehole depth. In each test the water level was recorded over a period of 2 hours at 15-sec intervals, using an automated, electronic water level transducer. The concurrent manual recording was done at 10-min intervals utilizing a water level indicator.

Over 30 verification borehole were drilled and tested for permeability; a total of over 170 cores were tested for UCS. The UCS results were always in compliance with contract specification requirements, with an overall average of 1300 psi and a minimum 10-point moving average of 880 psi.

The in-situ permeability was always between two or three orders of magnitude lower than required, when measured in the panel body and in the inclined boreholes. In addition, the refinement of the construction methods developed during the demonstration phase allow the utilization of a consistent process throughout the production phase, which provided permeability values ranging between 10^{-7} and 10^{-8} cm/s in vertical boreholes drilled along the joints. Further information about this Project and tests are reported in Elgouhary et al., 2016.

Summary and Conclusions

This methodology provided a) reliable and verifiable control of verticality, dimensions and continuity of the wall, through the real-time monitoring system and hydromill steering capability; b) a consistent quality and homogeneity of the backfill material, prepared in a dedicated on-site batching plant and fully tested before its placement in the wall; c) an effective way to construct the seepage barrier into the rock, due to the capabilities of the hydromill equipment.

© 2016 Deep Foundations Institute

The end result of the project proved that the selection to use plastic concrete as a backfill material for the Bolivar Dam produced a very high quality seepage barrier that exceeded all the performance requirements. The on-site batch plant provided the opportunity and flexibility to test, adapt and optimize the original mix design to the variable needs of the project along its construction.

The extensive and stringent QC/QA measures implemented throughout the project provided excellent control of the construction processes but above all great certainty as to the overall performance and consequent resiliency of the seepage barrier at Bolivar Dam.

CONCLUSIONS

Each project has its own unique features, with different stratigraphic characteristics, different requirements in terms of QC/QA and final performance. Even though the construction methods appear to be similar, each one was characterized by an unusual and innovative approach, which allowed meeting different construction needs and results in terms of performance.

Based on the Author's experience, it can be stated that the "the-one-solution fits all ideal system" does not exist: the construction method must be selected and tailored to each project, taking into account the site conditions and design requirements. In all these cases, performance specifications were the right vehicle which, together with the hard work and dedication of all USACE and Treviicos personnel, brought these projects to success.

DISCLAIMER AND ACKNOWLEDGMENTS

The contents of this paper summarize the authors' opinion and do not necessarily reflect those of the represented entities. The authors acknowledge the personnel of the U.S. Army Corps of Engineers Kansas City, Jacksonville and Huntington Districts for their professionalism and cooperation.

REFERENCES

Blackwell, D., Morales, C., Nicholson, P. and Taylor, J., 2010. "Cutoff Wall Construction By The Self-Hardening Slurry Method At The Herbert Hoover Dike, FL", Proceeding of Association of State Dam Safety (ASDSO) Officials Annual Conference, September 2010, Seattle, Washington.

Bruce, D.A. and Mauro, M., 2010. "Quality Control & Assurance & Performance Verification for Cutoff Walls", Proceeding of Association of State Dam Safety (ASDSO), Officials Annual Conference 2010, September 2010, Seattle, Washington.

Elgouhary, M., Koziol, D., Taylor, J. and Tasillo, C.L., 2016. "Seepage Barrier At Bolivar Dam", DFI International Conference on Deep Foundations, Seepage Control and Remediation", October 2016, New York, NY.

Stark, T.D., Beaty, M.H., Byrne, P.M., Castro, G.V., Walberg, F.C., Nicholson, P.J., Perlea, V.G., Axtell, P.J., Dillon, J.C., Empson, W.B., and Mathews, D.L., 2012. "Seismic Deformation Analysis of Tuttle Creek Dam". Canadian Geotechnical Journal, 49(3), 323-343.

Stark, T., Walberg, F., Byrne, P., Castro, G., Nicholson, P., Axtell, P., Beaty, M., Dillon, J., Empson, W., Topi, J., Mathews, D., and Bellew, G. (2013) Soil Improvement for Seismic Retrofit of Tuttle Creek Dam. Geo-Congress 2013: pp. 2069-2079.

© 2016 Deep Foundations Institute

UNIFIED SPECIFICATIONS FOR SOIL-MIXED SEEPAGE CUTOFF WALLS

Erik James PE, GE, PG, US Army Corps of Engineers, Sacramento District, Sacramento, California, USA, 916-557-5259, erik.w.james@usace.army.mil

Jose R. Gomez PE, US Army Corps of Engineers, Sacramento District, Sacramento, California, USA, 916-557-7147, jose.r.gomez@usace.army.mil

Wayne S. Smith PhD, PE, GE, US Army Corps of Engineers, Sacramento District, Sacramento, California, USA, 916-557-5381, wayne.s.smith@usace.army.mil

ABSTRACT

Soil mixed seepage cutoff walls are increasingly being used in levee and dam modification projects nationwide. An important aspect of their overall quality depends greatly on the language included in the specifications. Depending on the method (e.g. vertical axis auger mixing, cutter soil mixing, or trench mixing techniques) the specifications are a blend of prescriptive and performance based contract language with differences in element construction, element connections, quality control/quality assurance, and work area restrictions. Many of the existing specifications were initially developed by contractors in whole or in part, and can contain restrictive provisions favoring the authoring contractor. In response to these challenges, basic soil mixing fundamentals were revisited and recent lessons learned incorporated, resulting in the development of a new specification for soil mixed seepage cutoff walls that provides adaptability for the various construction methods. The new specification addresses quality control, quality assurance, equipment independence (elimination of restrictive contract language), rig instrumentation.

INTRODUCTION

Soil mixing has been used in the construction of seepage barriers for over 30 years. Most soil mixed project primarily use the wet-rotary-shaft, wet-rotary-end or wet-vertical panel methods (reference FHWA Deem Mixing Manual Figure 137). One of the first major soil mixing seepage barriers was constructed in the Jackson Dam in Wyoming. Since then the use of soil mixing for seepage barriers has increased significantly and this specialized domain of construction which was solely procured by RFP methods, is now increasingly being specified in projects issued for bid. The Bid documents require more careful specification writing than RFP methods because the RFP methods allowed detailed proposals by the contractor. Therefore the development of soil mixing specifications began with contractors preparing the specifications. This paper addressed some of these issues, and proposes methods to address these resulting concerns. The resolution of these issues considers several aspects: 1) Assessment of binder injection, 2) Evaluation of mixing index, and 3) Verification testing.

BACKGROUND

The motivation for this paper is in part the result of the rapid acceptance of soil mixing to be used as a seepage barrier. Secondly, following the recent publication of the FHWA Soil Mixing Manual, the US Army Corps of Engineers has embarked on the development of an Engineer Manual for Seepage Barriers, with a specific chapter focusing on soil-mixed seepage barriers. Part of that effort is to develop guide specifications for use in cutoff wall construction using this method. Defining the standard approach to quality assessment of the seepage control barrier, and the specifications to achieve high quality walls goes hand-in-hand, and this paper lays out the critical recommendations for specification writers. While many projects have been written with limited specs suitable for an RFP type solicitation, faster processes such

© 2016 Deep Foundations Institute

as lowest cost-technically acceptable methods, which evaluate the contractor's qualifications and experience, and make selection based on cost have become more common. For this method, more prescriptive specifications are required, but they must be open enough to permit the contractor to use his means and methods.

DISCUSSION

Assessment of Binder Injection

One aspect of ground improvement is assessing the quantity of binder injection. Tracking the mass of binder injected into the foundation soils is a necessary part of the quality control process. Typically this starts at the truck delivery stage and is measured by weigh tickets from inbound trucks delivering the binder components. The quantity of materials brought to the site is the first estimate. The second stage is the weights of the batches as the material comes in from the storage silos. For most contractors, these two first measurements of binder are straight forward.

Subsequent measurements of mass of binder as it moves from the batching plant through pipes/hoses and out through the injection ports is important specifically when targeting or correlating binder quantities to specific soil layers. While there are many ways the binder mass can be measured rig specific plumbing and pumping can cause conflicts in specifications that favor one technology over another. Thus specifications should permit reasonable differences in measurement techniques.

Evaluation of Mixing Index

The mixing index relates the degree of mixing for each unit volume. In general, there are two factors, rotational mixing and advancement rate. Several mixing indices have been developed for various techniques, such as the Blade Rotation Number (BRN) for deep soil mixing using augers equipped with paddles. Nicholson Construction presented a mixing index for a trenching type method. While standardize mixing indices are useful for comparing work across site conditions, for most cases, a mixing index that relates the degree of rotation of the mixing device, and the penetration or advancement rate can be developed and used for site specific conditions. This is true when using a demonstration section, where the establishment of the mixing index would take place.

The mixing index, once established in the demonstration section, and in conjunction with the mix design, and grout injection rates, can be used to measure quality of the cutoff wall during production. Deviations from the accepted mixing index (with some allowed variance) may indicate anomalies in the subsurface conditions or construction effort.

The mixing index for wet-rotary-shaft methods, where the mixing apparatus does not pass through the entire element volume:

$$Vertical\ Element, Vertical\ Axis: \quad I_m = BRN = M\left(\frac{N_p}{V_p} + \frac{N_w}{V_w}\right)$$

For vertical or horizontal mixing, where the entire soil mixing apparatus passes through the entire element, a simpler index can be used relating only rotation and advancement speeds:

$$Vertical\ Element, Horizontal\ Axis: \quad I_m = \left(\frac{N_p}{V_p} + \frac{N_w}{V_w}\right)$$

© 2016 Deep Foundations Institute

$$Horizontal\ Continuous\ Trench: \quad I_m = N_c\left(\frac{V_c}{V_a}\right)$$

Rig and Grout Plant Instrumentation

Since the monitoring of soil mixing parameters and binder placement are necessary to evaluation of soil mixing work, rig instrumentation is required.

While some contractors may elect to monitor these parameters differently than others, specifications should be written to require monitoring at multiple locations along the binder injection process.

With regard to rig sensors, different contractors may elect to monitor parameters in a different location or manner than others. The specifications should state minimum requirements. For instance, to develop the mixing index, rotation speed and advancement rates must be known by the engineers.

Verification Testing

Verification testing for the demonstration section should be more rigorous then the verification testing of the production wall because the contractor needs to show that the construction method will produce a cutoff wall in accordance with the project specifications. In the demonstration section the cutoff wall may be cored and an optical televiewer used to ensure the cutoff wall meets the design criteria. The verification testing in conjunction with the binder injection rates, and the mixing index would define the baseline for the monitoring of construction in the production wall. For the production wall the binder injection rates, the mixing index, and in situ wet grab samples should indicate if the production wall has any variations in quality as compared to the demonstration section. Consideration to the use of an independent means of collection of samples can be used. For instance lowering a metal beam with a single bailer-type device has been shown to be successful on projects. This sampling equipment is not limited to soil mixing rig type.

Element Construction

Since elements are formed by soil mixing rigs of various configurations, consider generic provisions in the specification, such as overlap should be equal to the width of the element.

RECOMMENDATIONS

The resulting recommendations for ensuring that specifications are as open as possible but still ensuring that quality considerations are met for cutoff wall specifications are as follows:

1. Consider the assessment of binder mass injection in assessing the quality of the cutoff wall. Require measurements of binder injection at various places along the binder injection process.
2. Use a mixing index appropriate to the rig and soil mixing device. The specifications should be flexible to allow for various types of equipment. Use established mixing indices where possible. For methods that are not easily adaptable to an existing mixing index, develop a mixing index during the demonstration section that relates rotation and advancement. For methods that do not mix or pass through the entire zone of the element, consider the blades or paddles similar to the BRN for multi-axis augers.

© 2016 Deep Foundations Institute

3. Ensure that quality control processes are independent of the method. Test the demonstration section heavily, including coring with use of OPTV. Then restrict coring for times of quality disputes. Tie quality in the wet-grab samples to the demonstration section. Use of a rig independent means of sampling the in situ samples is an option.

SUMMARY AND CONCLUSIONS

Projects benefit from having specifications that allow contractors to have reasonable expectations of requirements. They are also benefitted by permitting as many qualified contractors to bid on the project. Preparing specifications that allow flexibility in rigs type benefits projects by reducing cost and risk to owners, if the specification still have means to detect and enforce quality issues when they arise.

© 2016 Deep Foundations Institute



Deep Foundations Institute
326 Lafayette Avenue
Hawthorne, NJ 07506
USA
Tel: 973-423-4030 | Fax: 973-423-4031
staff@dfi.org | www.dfi.org