Construction and Monitoring of an Instrumented Soil-Bentonite Cutoff Wall: Field Research Case Study

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ABSTRACT: Soil-bentonite (SB) cutoff walls are commonly employed in the US to control groundwater flow and subsurface contaminant migration. In these applications, both the short-term (as built) integrity of the barrier and the potential for degradation in the integrity of the barrier over time are of critical importance. Although many laboratory studies have been conducted to investigate the hydraulic performance of SB cutoff walls and various factors affecting this performance, field investigations are scarce. With support from the National Science Foundation, a 194 m long, 7 m deep, and 0.9 m wide SB cutoff wall has been designed, constructed, and instrumented to serve as a long-term field research site for investigating the in-situ properties and behavior of SB backfill. This paper provides an overview of the project, instrumentation details, and representative post-construction testing and monitoring results. The results presented herein include index and hydraulic properties of the as-mixed SB backfill measured in the laboratory using the grab samples collected during construction, and monitoring data (i.e., inclinometer surveys, backfill settlement monitoring, and backfill state of stress measurements) collected during the construction period and for the first 120 days after construction.

The results indicate that the wall was well constructed and successfully instrumented to yield insightful information regarding the development of stresses and deformations in the backfill. The backfill continues to undergo secondary compression and load transfer to the adjacent formation, resulting in continued decreases in total stress. As expected, total stresses in the wall are considerably lower than would be predicted by assuming a geostatic stress distribution in the backfill. Effective stresses in the backfill are increasing gradually over time as excess pore pressures dissipate, but remain very low (≤10 kPa) after 120 days and are likely to remain low. Inclinometer results show that progressively inward displacements continue to occur adjacent to the wall, indicating progressive lateral squeezing of the backfill. None of the stresses, deformations, or pore pressures have reached equilibrium at the time of this writing, and monitoring will continue in the months and years ahead. Future work will focus on evaluating existing and newly developed models for predicting stress and strain development in SB cutoff walls with comparisons made to the measured field data; in-situ testing and sampling to investigate variability and changes in the hydraulic and strength properties of the backfill as a function of location, depth, scale, and time; and evaluation of electrical resistivity as a viable geophysical technique to non-destructively identify the presence of defects in the wall.
INTRODUCTION
Vertical barriers (i.e., cutoff walls) have been employed for more than 50 years to control groundwater flow and subsurface contaminant transport at thousands of sites in the US and abroad. In the US, the most common type of vertical barrier is the soil-bentonite (SB) slurry trench cutoff wall that takes its name from the nature of the final barrier materials (SB) and the method of construction (slurry trench). While numerous other methods are used to construct vertical barriers, such as the deep mixing method (e.g., Larsson 2005), the trench remixing and deep wall (TRD) method (Evans 2007), and self-hardening slurry methods like cement-bentonite and slag-cement-bentonite (e.g., Opdyke and Evans 2005), these other types of vertical barriers have been used far less frequently than SB slurry trench barriers.

Construction of SB slurry trench cutoff walls occurs in two phases, viz., (1) a vertical trench is excavated and simultaneously filled with bentonite-water slurry to maintain trench stability, and (2) the excavated trench spoils are mixed with slurry and dry bentonite (as needed) to create the SB backfill, which is pushed into the trench to complete the barrier. Properly prepared backfill is homogeneous and sufficiently fluid to fill the entire trench without entrapping pockets of slurry, yet sufficiently dense that the backfill is not too compressible (Evans 1993). The backfill also must exhibit a sufficiently low hydraulic conductivity ($k$) to meet project requirements (typically $\leq 10^{-8}$ m/s or $\leq 10^{-9}$ m/s).

Soil-bentonite cutoff walls continue to be widely used for long-term applications, such as levee repair and geoenvironmental containment, in which the barrier is expected to perform effectively for years, if not decades, after installation. In these applications, both the short-term (as built) integrity of the barrier and the potential for degradation in the integrity of the barrier over time are of critical importance. Short-term integrity of SB cutoff walls typically is assessed based on quality control/quality assurance (QC/QA) testing of field-mixed SB backfill, primarily using laboratory methods to measure $k$ (e.g., Millet and Perez 1981, Millet et al. 1992). Laboratory measurements of $k$ are easy to obtain and are important, but ultimately the in-situ performance is of primary significance. The in-situ $k$ of an SB cutoff wall depends upon the in-situ stress distribution in the wall, which typically is not measured and generally is not fully understood. The vertical stress distribution likely is influenced by arching as a result of frictional forces between
the backfill and the trench sidewalls (Evans et al. 1995), and the horizontal stress distribution is believed to deviate from the at-rest earth pressure due to lateral squeezing of the backfill by the adjacent, native formation (Filz 1996, Filz et al. 1999, Ruffing 2009, Ruffing et al. 2010). Laboratory $k$ values obtained from grab samples of field-mixed backfill specimens may not be representative of the \textit{in-situ} $k$ if the applied stress state in the laboratory test is not representative of the \textit{in-situ} stress state (National Research Council 2007). Moreover, laboratory $k$ tests are insufficient for verifying the absence of high-$k$ construction defects, and only a few such defects can significantly increase the overall $k$ of the barrier (Benson and Dwyer 2006). This latter point was evaluated in a study by Britton et al. (2004) in which laboratory and field methods for evaluating $k$ of a pilot-scale SB cutoff wall were compared. In this study, laboratory $k$ values obtained from undisturbed and remolded specimens of the field backfill were consistently lower than larger-scale $k$ values obtained from \textit{in-situ} measurements (piezocone and piezometer) and pumping tests. Regarding long-term integrity, several factors may cause changes in $k$ of an SB barrier over time, including deformations, desiccation, freeze-thaw, and chemical incompatibility (e.g., see National Research Council 2007, Malusis et al. 2011, Malusis and McKeehan 2013). The significance of these factors on the effectiveness of field-scale SB barriers is largely unknown.

Uncertainties in the state of stress (and thus hydraulic conductivity), time-dependent changes in backfill properties, and variability of hydraulic conductivity under field conditions are all compelling reasons for post-construction monitoring and testing of cutoff walls. The authors are aware of cases in which constructed cutoff walls have failed to provide the required containment due to construction defects or post-construction changes in the wall (as opposed to design deficiencies). In one case, poor wall performance was revealed by groundwater monitoring data and attributed to localized defects such as sand lenses embedded in the wall during construction (in this case, continuous cores were taken to identify and characterize these defects; see Evans et al. 2004). In another case, post-construction property changes and/or inadequate \textit{in-situ} testing procedures resulted in measured $k$ values greater than the required $k$ (Cermak et al. 2012).

Notwithstanding these examples, cutoff walls have been constructed for decades with few reported problems. However, published case histories of field investigations are scarce, due at least in part to trepidations of site owners that \textit{in-situ} testing of completed cutoff walls would be invasive and
potentially disruptive. While there have been a few field studies in which sampling and in-situ testing of an SB wall have been performed (e.g., Evans and Ryan 2005, Ruffing and Evans 2010, Ruffing et al. 2010, 2011, 2012), these studies have been limited in scope and duration due to site access limitations and concerns over potential impacts to the wall. Another layer of complexity is that post-construction groundwater monitoring often is not adequate to detect wall deficiencies. Complex geology and groundwater chemistry regimes may render detection of localized defects in a cutoff wall difficult unless the monitoring is focused specifically on the wall.

For all of the reasons described above, and with support from the National Science Foundation, an SB cutoff wall research site was created in the summer of 2016 near the Bucknell University campus (Lewisburg, PA) for the express purpose of evaluating the behavior of a full-scale SB cutoff wall for comparison to the practical understanding of SB behavior that primarily has been based on laboratory studies. The primary objectives of the field research are three-fold: (1) to investigate the in-situ state of stress in the wall, both at the end of construction and over time; (2) to investigate changes in the in-situ properties of the wall with time, including water content, $k$, and shear strength, with special consideration given to differences above and below the water table; and (3) to investigate the feasibility of electrical resistivity (ER) imaging as an effective geophysical method for detecting variations in homogeneity, including defects, within the wall. In this paper, details regarding the design, construction, and instrumentation of the wall are presented along with monitoring data and field and laboratory test results collected during the first 120 days after construction. Findings related to the short-term properties and behavior of the wall based on these data also are discussed, and plans for future work are described.

BACKGROUND
The SB cutoff wall research site is located on the property of a commercial sand/gravel quarry operated by Central Builders Supply (CBS) in Montandon, PA, approximately 3 km east of the Bucknell University campus in Lewisburg, PA (see Fig. 1). The wall is approximately 194 m long, 0.9 m wide, and 7 m deep, and was installed on a portion of the property that has been set aside in perpetuity as a buffer between the permitted sand and gravel mining area and an adjacent, natural
wetland known as the Montandon Marsh (Fig. 1c). The wall was installed in a primarily alluvial formation within the footprint of a paleochannel of the Susquehanna River. Non-invasive electrical resistance imaging (ERI) and exploratory soil borings, completed approximately every 30-50 m along the wall alignment (see Fig. 2), were used to characterize the subsurface conditions, verify the feasibility of the site for cutoff wall construction, and obtain samples for backfill mix design. The soil profile developed from logging and sampling at the locations of the borings, monitoring wells, and inclinometers, illustrated in Fig. 3, consisted of silty sand and gravel, underlain by sand and clay layers of varying thickness, followed by hard material (presumed to be bedrock) at 6-10 m below ground surface along most of the wall alignment. Borehole refusal depths in Fig. 3 corresponded well with higher electrical resistivity (indicative of bedrock) obtained from the ERI survey at similar depths. The depth to groundwater was approximately 2.5 m at the time of the subsurface investigations, but historic water levels measured in monitoring wells along the perimeter of the wetland indicate that the depth to groundwater approaches 0.5 m during wet seasons. Thus, at least 2 m of seasonal groundwater fluctuation is expected at the site.

WALL CONSTRUCTION AND INSTRUMENTATION

The cutoff wall was constructed by Geo-Solutions, Inc. (New Kensington, PA) and was completed over an 11-day period from July 11, 2016 and completed on July 21, 2016. Prior to construction,
a work platform was prepared by clearing and grubbing a 10-m-wide strip along the wall alignment (5 m on each side of the wall centerline), with topsoil stripped and stockpiled for later reclamation. A pond also was constructed adjacent to the platform for storage and hydration of bentonite-water slurry after mixing. The trench was excavated using a Caterpillar 330 excavator equipped with a 0.9-m wide bucket. Excavation of the lead-in trench began at station 2+07 m (see Fig. 2) and reached the design depth of 7 m at station 2+00 m resulting in a 1:1 starter slope. Excavation
proceeded from station 2+00 m to station 0+06 m, resulting in a completed wall length (excluding the lead-in trench) of 194 m. Bentonite slurry (5-6% bentonite by weight) was used in the trench for hydraulic shoring during excavation and for mixing the backfill. The slurry was mixed in a high-capacity slurry mix plant consisting of a Venturi jet mixer and a high-shear centrifugal pump. Mixing water was pumped from a nearby pond and blended with CETCO (Hoffman Estates, IL) Premium Gel bentonite to obtain the desired Marsh viscosity ($\geq 35$ s). The slurry was stored and circulated in the pond for further hydration and pumped to the trench as needed.

The subsurface conditions encountered during excavation were not entirely consistent with the conceptual profile created from the borings (Fig. 3). Much of the material excavated in the first ~100 m of the trench (i.e., station 2+01 m to station 0+99 m) contained appreciable fractions of gravel and cobbles (and occasional boulders), as illustrated in Fig. 4. Although some oversized materials were expected to be encountered, the abundance of these materials was greater than anticipated from the boring program. In addition, the stiff, gray clay encountered between stations 1+26 m and 0+57 m proved difficult to blend into the backfill. As a result, most of the native spoils were discarded, and the backfill was prepared with stockpiled material excavated from other

![Figure 4. Subsurface profile based on materials encountered during excavation.](image-url)
areas of the mine site. In addition, hard bedrock, cobbles/boulders, and limestone pinnacles limited the excavation depth to less than 7 m at some locations.

The backfill was prepared by mixing the imported base soil with bentonite-water slurry (until the desired slump of 75-150 mm was attained) alongside the trench with either the excavator or a John Deere 700J dozer. The trench was backfilled with the excavator or dozer, starting at the top of the lead-in slope. The backfill was always placed from a point along the trench where the backfill was visible above the slurry level. This point, called the “head” of the backfill, moves forward as the trench is backfilled. The backfill placed at the head slides and slumps down the backfill slope into the trench, thus displacing the slurry (Evans et al. 1985). An example profile showing the migration of the backfill slope and excavation face over a 24-hour period is illustrated in Fig. 5.

![Cutoff wall backfill profile based on trench soundings collected over 24-hour period from 7/15/16 (7 AM) to 7/16/16 (7 AM).](image)

**Figure 5.** Cutoff wall backfill profile based on trench soundings collected over 24-hour period from 7/15/16 (7 AM) to 7/16/16 (7 AM).

Quality control testing was performed throughout the wall construction, in accordance with the frequencies and requirements listed in Table 1. Soundings of the trench bottom and/or top of
Backfill were taken at least twice daily, and slurry taken from the slurry pond (i.e., fresh slurry) and trench (i.e., in-trench slurry) were tested 1-3 times daily for viscosity, filtrate loss, pH, and mud density. Backfill slump was measured on grab samples using a standard ASTM C143 slump cone at a frequency of one test for every 10 m of backfilled trench. When slump samples were collected, grab samples of the backfill also were collected and placed in 20-L buckets for future laboratory testing. During trench excavation and backfilling in the vicinity of the inclinometer locations (i.e., stations 0+30, 0+60, 0+90, and 1+20 m; see Fig. 2), inclinometer readings were taken four times daily. Settlement plates were installed at the locations shown in Fig. 2 once the backfill topped out at these locations, and the settlements were surveyed four times daily.

**Table 1. Summary of testing frequencies during trench excavation and backfilling.**

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Frequency</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soundings (3-m intervals)</td>
<td>Beginning and end of each work shift, and as requested by project personnel</td>
<td>Trench bottom should be clean and at desired depth</td>
</tr>
<tr>
<td>Marsh Viscosity (fresh slurry)</td>
<td>3 per day</td>
<td>35-40 s</td>
</tr>
<tr>
<td>Marsh Viscosity (in trench)</td>
<td>3 per day</td>
<td>&gt;40 s</td>
</tr>
<tr>
<td>Filtrate Loss (fresh slurry)</td>
<td>1 per day from pond</td>
<td>&lt;20 mL</td>
</tr>
<tr>
<td>Filtrate Loss (in trench)</td>
<td>1 per day from pond</td>
<td>None (informational only)</td>
</tr>
<tr>
<td>pH (fresh slurry)</td>
<td>3 per day</td>
<td>6.5-10</td>
</tr>
<tr>
<td>Mud density (fresh slurry)</td>
<td>3 per day</td>
<td>&gt;10 kN/m³ (&gt;64 pcf)</td>
</tr>
<tr>
<td>Mud density (in trench)</td>
<td>3 per day</td>
<td>10-13 kN/m³ (64-85 pcf)</td>
</tr>
<tr>
<td>Backfill slump</td>
<td>1 per 10 linear m</td>
<td>75-150 mm</td>
</tr>
<tr>
<td>Grab sample of backfill (for lab testing)</td>
<td>1 per 10 linear m</td>
<td>N/A</td>
</tr>
<tr>
<td>Inclinometer readings</td>
<td>4 times daily during excavation and backfilling</td>
<td>N/A</td>
</tr>
<tr>
<td>Settlement readings</td>
<td>4 times daily, starting immediately after plate placement</td>
<td>N/A</td>
</tr>
</tbody>
</table>
As illustrated in Fig. 2, the 50-m section of the wall immediately beyond the lead-in slope (i.e., station 2+00 m to station 1+50 m) was designated as the geophysical testing section. This section is being used to investigate the feasibility of ER imaging for detecting defects within the wall. Defects were created in this section by embedding a large limestone boulder (~600 mm diameter) and a series of bundled sand bags (i.e., woven polymeric bags filled with saturated sand and bundled together to create defects of different sizes) within the wall at prescribed locations and depths. Although most of these defects were placed along the bottom of the trench, two bundled sand bags were suspended above the trench bottom within the slurry-filled trench and connected to anchor ropes tied off above the ground surface to minimize their movement during placement of the backfill.

In addition to the monitoring wells and inclinometers installed adjacent to the trench prior to construction, earth pressure cages (RST Instruments Ltd., Maple Ridge, BC, Canada) were installed at two locations (stations 0+75 m and 0+87 m; see Fig. 2) to monitor the three-dimensional state of stress in the backfill over time (see Fig. 6). The instrumentation on each cage included three vibrating-wire stress sensors (“pancake cells”) to measure vertical and horizontal (longitudinal and transverse) total stresses. Each cage also was equipped with a vibrating-wire piezometer to measure pore water pressure, a biaxial tiltmeter and magnetic compass to monitor the in-situ orientation of the cage, and a thermocouple to monitor temperature. The cages were originally designed for use in mine paste backfill and, therefore, have an open structure that allowed the SB backfill to flow through the cage and cover the sensors. All sensors were connected to Kevlar-reinforced cables that extended out of the wall and were routed to an enclosed, weatherproof field station adjacent to the wall (see Fig. 2) for data acquisition.

The cages were installed at each location by first placing a braced steel frame into the slurry-filled trench and anchoring the frame into the clay bottom by pushing the pointed tips of the frame into the clay with the excavator. Metal sleeves were bolted onto the cage corners so that the cages could be placed on the frame (by sliding the sleeves over the steel legs of the frame) and lowered into the trench. At station 0+87 m, three cages were placed on the frame at different depths (i.e., 2.4, 4.4, and 6.4 m below ground surface) with cross-bracing between each cage. At station 0+75 m, a single cage was placed near the bottom of the trench (i.e., 6.2 m deep) to provide replicate
measurements of stress near the trench bottom. Each cage was held with vertical ropes tied off above the ground surface to fix the cage depth during backfilling. Also, the top of each frame was horizontally anchored to minimize rotation of the frame caused by the lateral pressure of the flowing backfill. Once the cages were submerged in the backfill, the vertical ropes were removed so that the cages would be free to move downward along the frame as the backfill settled.

Figure 6. Schematic of earth pressure cage installed in trench on braced frame keyed into underlying clay (left) and photograph of earth pressure cage (right).

All of the data collected from the sensors mounted on the earth pressure cages (i.e., total stresses, pore water pressures, temperatures, and cage orientations), from the vibrating-wire transducers in the monitoring wells installed adjacent to the trench, and from the sensors on the weather station installed on the top of the field station (i.e., precipitation, temperature, relative humidity, and barometric pressure) are being acquired with a Campbell Scientific CR6 data logger located inside
the field station. The sensors on the earth pressure cages are sampled and stored every 15 minutes. Weather data is sampled every 15 minutes and aggregate values are stored hourly.

All data are transmitted wirelessly to a base station on the Bucknell University campus, approximately 3 km west of the site. When a new data file is received by the base station, the data are passed to a web service on the Bucknell network for storage and for visualization with a custom web-based dashboard developed using the Flask microframework (http://flask.pocoo.org/). The dashboard allows the user to graph individual sensor data, display raw data in tabular form, and download data as a comma-separated values (CSV) file. The total time from data being sampled at the site to when the data become visible on the dashboard is much less than the 15-minute sampling interval, allowing site changes to be observed in real-time from any internet-enabled computer.

Approximately six weeks after the construction of the wall was complete, the top of the wall and the adjacent ground surface were leveled and a 3-m wide woven geotextile was placed over the area. All instrumentation was left in place. The geotextile was covered with approximately 0.4 m of topsoil that was graded and seeded with grass. The instruments and data cables protruding from the ground are the only remaining evidence above the ground surface that the wall is present at the site.

RESULTS AND DISCUSSION
The results presented herein are focused primarily on (1) the index and hydraulic properties of the as-mixed backfill measured in the laboratory using the grab samples collected during construction, and (2) the trench and backfill monitoring data (i.e., inclinometer surveys, backfill settlement monitoring, and backfill state of stress measurements) collected during the construction period and for the first 90-120 days after completion of the wall. Water levels are being monitored in the wells adjacent to the wall (see Fig. 2), and limited in-situ testing of the backfill (vane shear and dilatometer testing) also has been performed. In addition, investigation of ER imaging for detection of the defects installed in the geophysical testing section of the wall is ongoing. These aspects of the research are beyond the scope of this paper.
Backfill Properties

Index Properties

Grain-size distribution (GSD) curves for grab samples of the backfill collected at four locations along the wall alignment are shown in Fig. 7 along with the GSD curve for a grab sample of the base soil used to prepare the backfill. The results in Fig. 7 are representative of the gradations of samples collected along the entire wall alignment and illustrate the high degree of backfill homogeneity achieved during construction, due to both the efficacy of the field mixing process and the use of imported base soil that exhibited a consistent gradation throughout the project. The fines contents of the base soil and backfill samples ranged from 44-57 %, and clay contents (fraction finer than 0.002 mm) ranged from 13-20 %. Because no dry bentonite was added to the backfill prior to mixing, the bentonite comprised only ~1 % of the overall solids content of the backfill (i.e., the contribution of bentonite from the slurry). Thus, the base soil was the source of nearly all of the fines in the backfill. Atterberg limits tests on the base soil and backfill samples yielded liquid limits of 16-18 % and plasticity indices of 6-7 %. Thus, the samples classified as either SC-SM or CL-ML based on the Unified Soil Classification System (ASTM D2487).

Figure 7. Grain size distributions of base soil and backfill samples collected during construction. Legend indicates the stations where the backfill samples were collected.
Hydraulic Conductivity ($k$)

Flexible-wall $k$ tests (ASTM D5084 Method C) were conducted on field-mixed backfill specimens prepared from grab samples collected at four locations along the wall alignment (Fig. 8a). The testing procedures and apparatus were the same as those described by Malusis et al. (2009) and Malusis and McKeehan (2013), and involved the use of a custom-fabricated, acrylic cylinder (length = 71 mm, diameter = 71 mm) placed around the flexible membrane to provide lateral support for the backfill prior to consolidation. Specimens were consolidated under back pressure at an effective confining stress, $\sigma'$, of 21 kPa (3 psi) or 35 kPa (5 psi) before permeation with the same water used to prepare the slurry in the field. Final values of $k$ for the four backfill specimens ranged from $7.0 \times 10^{-10}$ to $1.4 \times 10^{-9}$ m/s, with the higher values obtained for specimens consolidated under the lower effective stress (21 kPa). The consistency of these results is another indication of the high degree of backfill homogeneity achieved during construction. Also, although no design $k$ was specified for the backfill on this project, the results indicate that the contribution of native fines from the base soil and the small fraction of bentonite from the slurry (~1 %) was sufficient to achieve $k \approx 10^{-9}$ m/s, which is the $k$ typically specified for SB cutoff walls in long-term geoenvironmental containment and hydraulic barrier applications.

In addition to the flexible-wall tests, a staged falling-head test was conducted on a backfill specimen in a fixed-ring oedometer (Fig. 8b). This specimen was prepared from a post-construction Shelby tube sample collected at station 0+45 m from a depth of 5.2-5.8 m. The specimen was trimmed into the consolidation ring and subjected to normal (vertical) effective stresses ranging from 24 kPa (0.25 tsf) to 383 kPa (4.0 tsf). After primary consolidation was complete for each load increment, the specimen was subjected to falling-head testing to measure $k$, following the same approach as described by Yeo et al. (2005). Excellent consistency is observed in Fig. 8b between the final hydraulic conductivities for the flexible-wall specimens and the oedometer specimen when permeated under similar effective stresses. Overall, the results in Fig. 8b illustrate the dependency of backfill $k$ on effective stress and underscore the importance of conducting laboratory $k$ tests at effective stresses that match the field stresses as closely as possible.
Figure 8. Laboratory hydraulic conductivity results for field-mixed backfill: (a) flexible-wall specimens prepared from field grab samples (legend denotes stations of sample locations and average effective confining stresses, $\sigma'$); (b) comparison of final hydraulic conductivities for flexible-wall specimens and oedometer test specimen prepared from post-construction Shelby tube sample of backfill (collected at a depth of 5.2-5.8 m at station 0+45 m).
**Post-Construction Monitoring and Testing**

*Backfill State of Stress*

Direct measurements of total stress (in three directions) and pore pressure within the backfill at the locations of the earth pressure cages (i.e., Stations 0+87 m and 0+75 m) are shown as a function of time in Fig. 9. While the stresses and pore pressures have not reached equilibrium at the time of this writing, several important observations can be made. First, at the time backfilling was completed, the total stress at a given depth was approximately equal to the weight of the overlying backfill and was reasonably isotropic. These observations are consistent with the fact that the backfill is placed as a thick viscous liquid. Over time, as the backfill undergoes primary and secondary consolidation, the total stress declines as the load is transferred through shear to the sidewalls of the trench (note that the “blips” in the time history relate to stresses caused by site activities such as placement of soil cover, final grading, and seeding). Likewise, pore pressures are dissipating along a similar time line as the total stresses. Just as in a consolidation test when a new load is applied, the load represents a change in total stress, but initially the load is carried by the pore water (resulting in excess pore water pressure). With time, the excess pore water pressure dissipates and the load is transferred to the soil grains, resulting in an increase in effective stress.

Shown in Fig. 10 are the total stresses and pore water pressures at five separate times (i.e., 0, 30, 60, 90, and 120 days after construction) as a function of depth at station 0+87 m. These data show linearly increasing total stresses and pore pressures with depth at a given time, and decreasing total stresses and pore pressures with time at a given depth. As shown in Fig. 10c, all measured vertical stresses are less than the theoretical geostatic vertical stress computed based on the self-weight of the backfill (i.e., $\sigma = \gamma z$, where $z$ is depth and the saturated unit weight, $\gamma$, of the backfill is 18.9 kN/m$^3$) due to load transfer from the backfill to the adjacent formation.

The data in Fig. 10d indicate that excess pore pressures in the backfill continue to dissipate over time. However, effective stresses computed based on the total stresses and pore pressures in Fig. 10 remain very low (i.e., $\leq$ 10 kPa in all directions and nearly zero in the vertical direction in the
Figure 9. Results of total stress and pore pressure monitoring in the backfill at earth pressure cage locations (stations 0+87 m and 0+75 m); \(\sigma_x\) = horizontal (transverse) total stress; \(\sigma_y\) = horizontal (longitudinal) total stress; \(\sigma_z\) = vertical total stress; \(u\) = pore water pressure.
Figure 10. Distributions of total stress ($\sigma$) and pore water pressure ($u$) measured at station 0+87 m immediately after construction ($t = 0$) and at 30, 60, 90, and 120 days after construction.

lower 2-3 m of the wall) after 120 days, as illustrated in Fig. 11. These findings are reasonably consistent with those reported by Ruffing et al. (2011) for another SB cutoff wall constructed with a similar depth. At this site, horizontal effective stresses estimated from dilatometer tests reached ~20 kPa in the backfill after 90 days of service. However, the slow rate of change in effective
stress evident from the data in Fig. 11 suggests that the effective stresses in the backfill at this site will likely remain below 20 kPa long into the future.

Figure 11. Distributions of effective stress ($\sigma'$) measured at station 0+87 m immediately after construction ($t = 0$) and at 30, 60, 90, and 120 days after construction.

Backfill Settlement

Backfill settlement from four settlement plates is shown in Fig. 12 and represent the time rate of backfill consolidation in the field. The shape of these curves is generally consistent with the shape
of those typically obtained in laboratory consolidation tests. The results indicate that primary consolidation is largely complete after two to three weeks, but secondary consolidation (creep) continues. Preliminary calculations of time to 90% consolidation using laboratory data (not shown) from the oedometer test on the aforementioned Shelby tube sample collected from the wall at station 0+45 m compared well with similar calculations based on the field settlement data in Fig. 12. An interesting aspect of this comparison is that the calculations were made assuming the horizontal direction as the shortest flow path and did not consider the possible presence of a bentonite filter cake at the interface between the backfill and the native formation along the trench sidewalls. The extent to which the filter cake remains intact during backfilling remains a lingering question for future consideration.

![Graph of backfill settlements](image)

**Figure 12.** Backfill settlements measured as a function of time (after completion of backfilling) at four settlement plate locations.

*Inclinometer Surveys*

Four pairs of inclinometers were installed prior to trench excavation by drilling 200-mm diameter borings using hollow stem augers advanced with an Acker Soil Scout track-mounted drill rig. After installing the inclinometer tubing through the hollow stem auger, cement-bentonite grout was tremmied into the annulus as the augers were withdrawn. Once the grout had cured a
minimum of 14 days, baseline readings were obtained. Readings were obtained at key times during construction of the wall (e.g., during excavation and backfilling) and continue to be collected over time as the backfill continues to settle and stresses continue to change in the backfill and correspondingly in the adjacent formation. Representative results, shown in Fig. 13 for the pair of inclinometers installed at station 0+60 m, illustrate that lateral displacements in the adjacent soil occur inward (toward the trench) during excavation, then slightly outward (away from the trench) during backfilling.

![Graphs showing lateral displacements](image)

**Figure 13.** Lateral deformations measured in inclinometers at Station 0+60 m: (a) Inclinometer 3 (west side of wall); (b) Inclinometer 4 (east side of wall). Measurements shown were taken prior to excavation (7/11/16), after full excavation depth was reached (7/18/16), after completion of backfilling (7/21/16), and 77 days after backfilling (10/6/16).
during backfill placement, and then progressively inward over time as the backfill undergoes primary and secondary consolidation. After approximately 77 days from completion of the wall (or 88 days from the start of construction), cumulative lateral displacements of 17-20 mm have occurred near the ground surface at station 0+60 m. The displacements generally decrease with depth (notwithstanding the slight bulge in the displacements at depths of 4-6 m) and decrease to nearly zero as the depth approaches 7 m (i.e., the bottom of the wall). These deformation data will be subsequently used to examine the relationship between lateral deformation and stress development in the backfill and to compare against predictions from models developed to simulate stress and strain development in SB cutoff walls (e.g., see Ruffing et al. 2010).

SUMMARY AND CONCLUSIONS
This paper presents the design, construction and preliminary results from a soil-bentonite slurry trench cutoff wall built for the express purpose of conducting research on the short- and long-term behavior of these systems. The wall (approximately 194 m long, 7 m deep and 0.9 m wide) was successfully constructed using quality control measures consistent with professional practice. Substantial monitoring of the wall was undertaken during and after construction using inclinometers, settlement plates, monitoring wells, and sensors to measure in situ stresses and pore water pressures. The results for the first 120 days after construction show that the backfill continues to undergo secondary compression and load transfer to the adjacent formation, resulting in continued decreases in total stress. Total stresses in the wall are considerably lower than would be predicted by assuming a geostatic stress distribution in the backfill. Effective stresses in the backfill are increasing gradually over time as excess pore pressures dissipate, but remain very low (≤10 kPa) after 120 days and are likely to remain low long into the future. Inclinometer results show that progressively inward displacements are continuing to occur adjacent to the wall, indicating progressive lateral squeezing of the backfill.

None of the stresses, deformations, or pore pressures have reached equilibrium at the time of this writing, and monitoring will continue in the months and years ahead. The data will be used to evaluate existing and newly developed models for predicting stress and strain development in soil-bentonite cutoff walls. In addition, an intensive program of in-situ testing and sampling will be conducted to investigate variability and changes in the hydraulic and strength properties of the
backfill as a function of location, depth, scale, and time. Finally, research is ongoing in the geophysical testing section of the wall to investigate the electrical resistivity method as a viable technique non-destructively identifying the presence of defects in the wall. These are the primary areas of focus for the ongoing research at this site.

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