Design and Construction of The 
Upstream Geomembrane Facing System for Grindstone Canyon RCC Dam

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Abstract

The Grindstone Canyon Dam near Ruidoso, New Mexico was built using roller compacted concrete (RCC) in 1986, with a height of 123 feet, a 1300-foot-long dam crest, and a vertical upstream face. The pool level in Grindstone Canyon Reservoir had been restricted by the New Mexico Office of State Engineer NMOSE since 2010 to an elevation of 6903.5 representing 11% of the dam height in an effort to mitigate seepage and stability concerns. The primary design objective was to reduce seepage that had been occurring at the dam since first filling so that the full reservoir height could be utilized. To meet this objective, designers selected a flexible PVC geocomposite membrane system for installation over the majority of the upstream face of the dam. The facing area lined for this project was approximately 59,000 square feet, approximately the upper half of the face height or 61% of the total upstream face area.

A geocomposite membrane system was selected as the preferred alternative for a cost-effective and viable waterproofing solution. The selected membrane was a Sibelon CNT 4400, which is a 120 mil (3mm) thick PVC membrane with a 15oz per square yard (500-gram per square meter) non-woven geotextile heat-bonded to the back of the membrane.

Because the RCC was porous and fractured and the fractures reflected through the upstream face to vertical rustication joints, significant interconnected seepage paths across the lower liner seal were likely. A urethane grout curtain was installed behind the dam face to cut off potential seepage paths around the lower seal of the geocomposite system. Also, cracks at the base of vertical rustication joints beneath the liner and where the joints crossed under the lower seal were grouted to cut off potential seepage paths.

A drainage system was installed across the base of the membrane installation just above the lower submersible seal. The system included three drains cored horizontally through the dam, and downstream piping, which carries seepage to the spillway stilling basin.

The subsequent liner drain flow data will be presented to better understand the seepage after installation at Grindstone Canyon Dam to determine liner system effectiveness. From this data, it is apparent that the liner is performing well with drain seepage flow rates approaching zero about one year after the liner was installed.

Introduction

Grindstone Canyon Dam is located in the Village of Ruidoso (VOR or Village), Lincoln County, New Mexico and is owned by the VOR. The dam was constructed of roller compacted concrete (RCC) with a conventional unreinforced concrete face. The reservoir stores about 1550 acre-feet at the
The primary spillway crest, and provides a raw water supply for residents of the VOR and is also used for recreational fishing. Water diverted from the Rio Ruidoso fills the reservoir by gravity flow to a diversion outfall structure just upstream of the left dam abutment, where it flows along the upstream dam face to the reservoir.

The dam is a 123-foot-high RCC structure with a total crest length of approximately 1,300 feet at elevation 6925.3 feet, which includes a 3.75-foot high concrete parapet wall. There are two spillway sections; a 103-foot-wide primary ogee spillway at about elevation of 6919.5 feet and a 248-foot-wide secondary spillway at about elevation of 6921.5 feet. Photo 1 shows the dam and spillway before liner installation.

![Photo 1: Grindstone Canyon Dam before the installation of the geocomposite PVC liner. The primary and secondary spillways can be seen to the right of the gate operators.](image)

The upstream face of the dam is vertical, while the downstream stepped face has a 0.75H:1V slope. A formed conventional concrete layer was placed about 2 to 2.5 feet thick on the upstream face over the RCC. Three-inch deep formed vertical joints (vertical rustication joints) and 1.5-inch formed horizontal joints (horizontal rustication joints) were installed across the upstream face, as pictured below. The horizontal joints are spaced every two feet below El. 6870.5 feet, and every one foot above that elevation and are likely intended to give the dam an aged or masonry aesthetic appearance. Vertical joints are spaced at 16 feet across the entire dam face and are intended to accommodate expansion and contraction of the concrete. Photos 2 and 3 show the rustication joints.
Historic documentation indicates that the dam has had a history of high seepage since it was first filled in 1988. Information provided by the VOR indicated repair projects intended to mitigate seepage through the dam were conducted in 1989, 1996, 2002, and 2009. These repairs, which included filling rustication joints with sealant and placement of a bentonite blanket along the upstream foundation contact, had limited success and the seepage generally increased, leading to a stability study that was completed in 2013 by another engineering company (other than AECOM). This study found the seepage was due to porous zones of partially cemented aggregates and/or transverse thermal cracking, with seepage increasing during colder temperatures. This was likely the result of the opening of cracks in the dam face due to thermal contraction. The stability study led to a recommendation that a liner be installed to reduce seepage and its impact on the dam’s structural integrity.

Results of stability calculations show that the Grindstone Canyon Dam will be stable under anticipated loading conditions. Relatively porous portions of the RCC material detected by seismic geophysical measurements and observed by Ruidoso operations personnel down to 60 feet below the top of the spillway are recommended to be treated with an upstream liner to reduce water losses, and reduce hydrostatic uplift forces on the RCC layer boundaries. The reduction of RCC interlayer hydrostatic uplift forces significantly increases the stability of the dam structure especially during earthquake loading events. (AMEC 2013)

The New Mexico Office of the State Engineer (NMOSE) accepted the AMEC stability analysis report and removed restrictions on storage levels in the reservoir if the liner system was installed as recommended. Although the predominant reason for the partial liner recommendation was increased dam stability, the benefit of reduced seepage would also reduce costs that the VOR was incurring to pump seepage back into the reservoir. Because of limited raw water storage and the Village’s reliance on the water stored in the reservoir to meet municipal demand, the reservoir could not be drained completely to allow for lining the entire dam face. A partial liner was expected to provide some seepage reduction
benefit, although less than would be realized by a fully lined face, but the selected liner would not affect seepage that may be coming through the foundation.

**Design Decisions**

The VOR selected the Carpi geocomposite liner as the preferred method to reduce seepage through the dam based on AMEC’s recommendation. Since the VOR would not be able to completely dewater the reservoir and expose the full upstream face, elevation 6860 feet was chosen to be the lower extent of the membrane system, which is approximately 60 vertical feet below the primary spillway in the mid-portion of the dam. This is consistent with earlier recommendations developed through engineering studies. Because the outlet works operators are mounted on the upstream face just left of the primary spillway, and lining this section would be difficult due to access, the decision was made to leave this area unlined. This created two separate liner areas and drainage systems for the left and right sides of the dam. The unlined area was inspected during construction and it was confirmed that the face was not cracked, and could therefore remain unlined. A profile of the dam and lined area is shown on Figure 1.

![Figure 1: Elevation view of dam and liner limits](image)

**Design Investigation**

AECOM conducted limited pre-design investigations of the upstream dam facing to assess the condition of the vertical rustication joints since the lower liner seal would have to cross them. The existing ground at the upstream face was surveyed and test pits were excavated to expose the foundation contact and lower seal location to confirm its condition and location as shown on Record Drawings. The reservoir was still being drawn down for the anticipated construction project, and was at elevation 6874.2 feet during the investigation in the fall of 2014, which prevented access to elevation 6860 feet, the lowest anticipated elevation for the bottom seal.

Under AECOM’s direction, three, 6-inch-diameter cores were taken from the dam’s left side. All three cores were centered on the intersections of vertical and horizontal rustication joints, and were a
few feet above the ground. All three vertical rustication joints were found to have cracks through the concrete facing. No cracks were noted in any of the horizontal joints observed, but a complete inspection of the face was not conducted due to limited access and the fill that had been placed over the lower portion of the face. The core on a vertical rustication joint was drilled through the concrete facing and into the RCC. The RCC surface, although uneven, was found at a depth of about 26 inches from the upstream face, close to the dimension indicated by the 1987 Record Drawings. The recovery of only larger loose aggregate at the back of the core was considered an indication of segregated and poorly cemented RCC at the concrete/RCC interface. Voids capable of transmitting seepage were noted in the back of the cores, and were assumed to exist intermittently across the upstream face. The core holes were backfilled with dry pack consistency non-shrink grout tamped in 2-inch lifts. The following photograph (Photo 4) was taken of a core drilled through the face at a joint and into the RCC, and shows the deteriorated contact between the face and RCC, as well as the crack that propagated through the face.

![Photo 4: A core sample taken from the upstream face of Grindstone Canyon Dam. Note the formed vertical rustication joint on the right side, and the deteriorated RCC disbonded from the conventional concrete facing on the left side.]

AECOM also excavated test pits along the dam face to 1) observe the condition of the concrete face below the existing ground surface near the foundation contact, where the lower seal of the liner would be located, 2) determine if the horizontal and vertical joints extended to the foundation contact, and 3) determine if the elevation of the foundation contact was consistent with the 1987 Record Drawings.

At the foundation contact, the excavation encountered a bentonite layer placed as part of the 1989 repair project. An upper bentonite blanket near the top of the backfill and against the face (shown in the 1989 report) was not discernable in the excavation. The vertical rustication joint terminated above the flat portion of the face, and there was no indication of a crack extending down from the vertical joint termination.

After exposing the foundation, AECOM requested an additional test pit at another joint to see if conditions varied. At this location, the vertical rustication joint extended to within a few inches of the
foundation contact and a crack extended below the vertical rustication joint through the leveling concrete to the foundation contact. Based on a taped measurement from the parapet wall to the foundation contact, the foundation contact elevations were confirmed to be within +/-1 foot of the elevations shown on the 1987 Record Drawings at these joints.

Carpi’s original design intent was to line the dam face to the foundation contact of the dam in the abutments. During the design investigation, it was noted that on the Record Drawings, the elevation of foundation contact in the abutment areas above elevation 6860 feet (bottom elevation of the liner) appeared to be about 20 to 25 feet below the existing ground surface. The current ground surface was later confirmed with a survey that, when combined with test pit information, showed that 20 to 25 feet of soil would need to be excavated upstream of the face to install the liner to the foundation contact. AECOM, Carpi, and the VOR agreed that it was prudent to install the liner as Carpi had originally intended—to the foundation contact in the abutments. The VOR planned to maintain the reservoir level during construction at about elevation 6855 feet to provide working room and freeboard for the bottom seal installation.

The information collected during the design investigation led to the recommendations to install a grout curtain behind the conventional concrete facing. This grout curtain would be installed along the lower seal to cutoff seepage that could be transmitted through the porous RCC along the back of the face, and through cracks in the face near the lower seal. These cracks and other defects in the face, including along the foundation, would be in contact with reservoir water pressure, and if left unchecked, would be a direct source of seepage into the RCC. This seepage could potentially end up between the upstream face and the proposed liner. Crack grouting in the abutments was also added to the program to further reduce communication from the reservoir to the RCC dam body.

**Carpi Geocomposite Liner System**

The installation of a Carpi geocomposite membrane creates a continuous seepage barrier. The geocomposite membrane consists of a 120 mil polyvinyl chloride (PVC) geomembrane that is heat-bonded to a (15 oz/yd²) non-woven geotextile. The heat-bonded geotextile increases the stability of the geomembrane and provides a cushioning layer between the geomembrane and the upstream face surface, and provides additional puncture resistance.

Beneath the geocomposite is a drainage system that is designed to help direct water that collects behind the liner to designated drainage points that carry seepage through and downstream of the dam. Four drainage points were initially chosen at Grindstone Canyon Dam, two in the left membrane area and two in the right membrane area to provide redundancy; however, due to drilling difficulties, one drain in the right liner area was eliminated during installation. After inspecting the upstream face, a 59 oz/yd² (2,000 g/m²) non-woven geotextile was chosen for the drainage system due to the irregularities of the upstream face from the many horizontal and vertical rustication joints, as well as the geotextile’s ability to transmit water and act as a drainage layer. The geotextile acts as additional protection to keep the waterproofing geocomposite from being pressed into these rustication joints once hydrostatic pressures are applied. Photos 5 and 6 show the geocomposite liner and the cushioning geotextile.
The drainage system through the dam body consisted of 3-inch diameter holes core drilled through the dam to the downstream face. The core barrel is made of stainless steel, and was left in place once drilling penetrated the dam. The annular space between the drain conduit and dam was then grouted to direct seepage into the drain pipes at their upstream openings near the membrane, and to prevent water within the dam body from exiting through the annular space. To measure seepage from the liner drains, the drains were designed to daylight on the downstream face instead of into the drainage gallery inside the dam body, which required about 50 feet of drilling. After daylighting on the downstream face, the stainless steel piping continues as buried conduit to prevent freezing until the pipes discharge into the stilling basin.

Once drilled, each of the drain holes had a stainless steel protection plate installed overtop to prevent any debris from entering and clogging the drain pipes. A tri-planar geonet was then installed along the bottom of the liner to the drain holes to direct water horizontally towards the drainage locations. Spacers hold the plate off the dam face to allow seepage to flow behind the plates from the geonet and into the drain pipes. The drainage plate detail is shown on Figure 2.
Figure 2: Drainage plate detail

The geocomposite liner is anchored to the upstream face using a vertical stainless steel anchor system typically installed about every 19 feet. Actual spacing length is determined based on a minimum wind loading of 100 MPH and other factors for specific locations. This anchor system consists of an internal profile that is fixed directly onto the upstream face prior to installing the geocomposite or drainage layer. These profiles are installed by drilling stainless steel anchors directly into the concrete of the upstream face. Once the geocomposite has been installed over the internal profiles, an external profile is attached to the internal profile by extending the anchor with a coupler. Holes are cut into the liner at each anchor bolt so the anchor bolt can penetrate the liner and overlying external profile, and a screw is placed over the external profile into a hole that is countersunk so the screw does not protrude beyond the face of the external profile. A geomembrane cover strip is then welded over the external profile, producing a water tight seal over each profile line. Once fully installed, the external profile pulls the liner to the dam face and tensions the geocomposite liner between anchorage profile lines, creating a taut system across the upstream face. Once completed, these profiles are the primary means of resisting wind uplift during the system’s service life. The top and bottom ends of the profiles are tapered from full height to nothing to avoid a sharp angle that could puncture the liner. Figure 3 shows a detail of the tensioning profile system. Photo 7 shows the liner being installed between profiles and how it becomes taut once tension is applied.
Figure 3: Tensioning profile detail

Photo 7: The tensioning profile anchorage system in progress. In the center of the photo are multiple external profiles installed along the top portion of the upstream face. The geocomposite is pulled taut in the area where the external profiles are complete and is loose along the bottom where external profiles are still needed. The gap in the white cushioning geotextile on the right side of the photo is the internal profile. Note the liner wrapped around the swing stage ready for deployment down the internal profile.

Along the perimeter of the geocomposite liner system is a seal that can be either submersible or non-submersible depending on the location. These liner seals attach the edges of the liner to the dam face, where tensioning profiles previously discussed are not used. Any area that will be
underwater would need to have a robust submersible seal, while any area that will be a safe distance away from the water level could be non-submersible. The submersible seal (tested to over 2,800 feet of head) creates a waterproof barrier through the use of compression applied on the geocomposite liner from a stainless steel flat profile. To create a smooth surface against concrete, and to remove possible voids, a two-part epoxy resin is placed underneath the geocomposite liner prior to compression being applied. The submersible seal is shown in Photos 8 and 9, and the non-submersible top perimeter seal is shown on Figure 4.

Photos 8 and 9: The submersible seal detail involves multiple layers of material in compression. The left photo shows the rubber gasket that is placed between the geomembrane and the stainless steel plate. The right picture shows the submersible seal completed. The completed seal has an epoxy resin placed underneath the geomembrane to make a smooth surface once the anchors are tightened and the compression is applied. The epoxy resin helps fill holes and other irregularities in the concrete surface to provide a better seal.

Figure 4: Non-Submersible Top Seal Detail

Where the seals crossed rustication joints, a low-modulus epoxy joint-filling compound was used to fill
the void space. This epoxy compound was formulated with ceramic microspheres as a filler to reduce shrinkage. This was done because the depth of the joints was more than most epoxy manufacturers recommend for the use of their products. To avoid excessive shrinkage during curing, a specialty epoxy manufacturer was contacted, who was able to formulate a product particularly for this project.

**Lower Submersible Seal and Crack Grouting**

Because only the top 65 feet of the dam height was lined (including parapet wall), the approximate bottom half of the upstream face was left exposed to the reservoir. As a result, leakage through the unlined portion of the face could still occur. The elevation of the lower submersible seal varied between El. 6861.0 and 6861.5 feet. Where the seal meets the abutments on either side, the lower seal generally followed up the abutments along the dam foundation contact. In order to address seepage bypassing the lower seal, hydrophobic polyurethane chemical grout was injected behind the concrete facing in the immediate vicinity of the lower seal between the abutments to construct a grout curtain. The intent of this grout curtain was to cut off and block potential seepage pathways behind the upstream face that would allow water to flow into the dam body and potentially between the liner and dam face, where it would end up discharging through the liner drains. Depending on the location, different grouting procedures were implemented. Between the abutments, a grout curtain was installed behind the dam face in the location of the bottom seal, and additional crack grout holes were installed at each joint to inject grout at each vertical joint that was crossed. Where the lower seal meets the abutments and the seal turned upwards to follow the dam foundation contact, the grout curtain was eliminated and a different procedure was used along the abutment contact, which involved grouting only at the vertical joints that were crossed. Vertical rustication joints in the abutments were grouted below the bottom seal and slightly above to seal cracks that might transmit water behind the seal through the crack. In addition, the face was inspected for visible cracks outside of the joints, and if observed, these were grouted as well.

Holes were drilled into the face for grout injection using both rotary hammer and pneumatic rock drills. After the hole was completed to full depth, it was flushed with water to wash out any remaining drill cuttings and then a mechanical packer was inserted into the hole. The packer was then tightened sufficiently to expand it into the side of the hole, and a zerk fitting was screwed into the threaded tube on the end of the packer. The zerk fitting has a small ball valve on the end that is opened under pressure when the grouting hose tip is placed over it, and when the grout hose is removed, the ball valve closes to prevent grout from being ejected out of the hole under pressure. Before grout was injected, the hole was first flushed by injecting water through the zerk fitting. This provides water to react with the grout, causing the hydrophobic grout to foam and expand into the drill hole and any void connected to the drill hole. The zerk fitting was left off of adjacent un-grouted holes, allowing air to escape as grout was injected. Frequently, grout was observed traveling from the injection hole to adjacent holes.

Grout and water were injected into the drill holes via the zerk fittings using airless paint sprayers as grout pumps. The pump pressure was adjustable and monitored with a digital pressure display to help control grout pressures. One pump was dedicated solely for water injection, and another solely for grout to avoid mixing the reactive components prematurely. Grout and water injection pressure, as measured at the pump discharge, was typically in the 800 to 900 psi range, which is typical for this type of work.
Curtain holes were grouted in groups. Each group of holes was grouted sequentially, which allowed time for the grout to react with water, and expand into the voids but minimizing travel beyond the injection ports and grout curtain line. After about one-half gallon to one gallon of grout was injected, water was then injected to react with the grout and to flush the packer and zerk fitting before moving on to the adjacent hole. After grouting the last hole in a group, grout injection was resumed on the previous holes, which by then had sufficient time to react and reach final set for the injected material. This helped minimize grout travel and injection volume, but allowed further injection of grout until the hole refused to take more grout, at which time it was considered completely grouted.

Different polyurethane grout formulations were used for the curtain and crack grouting. Grout mixing and injection procedures were similar for each type of grout, but the depth and orientation of the drilled holes were different. Curtain holes were drilled straight into the face every 12 inches to the contact with RCC at a 90 degree angle. Crack holes were angled into the crack at a 45 degree angle to intersect the cracks at about 12 inches behind the face. Crack holes typically alternated from one side to another and were offset above or below each other.

The grout used for curtain grouting was DeNeef CUT PURE, a hydrophobic urethane grout that reacts quickly with water. Reaction time increases with higher temperature and the addition of more catalyst. At 5 to 7 percent catalyst by volume, the time to initial foaming reaction at about 60°F was about 40 seconds, with final set time just over 3 minutes. Grout take was variable across the length of the curtain and ranged from 0 to 5 gallons, with a typical hole taking 0.5 to 2 gallons. A total of 754 grout curtain holes were drilled for the project, and a total of 912.2 gallons of chemical grout was injected for an average take of about 1.2 gallons per hole.

The grout used for crack grouting was DeNeef Flex LV PURE, a lower viscosity hydrophobic urethane grout. A total of 135 crack holes along elevation 6861.5 feet +/−, and 71 abutment crack holes were drilled for the project. A total of 27.1 gallons of grout was injected into the 206 crack grout holes. Grout volume injected into the vertical joints was typically small, and ranged from essentially 0 to about 0.31 gallons per hole, and averaged about 0.09 gallons per hole. Grout volume injected into abutment holes was typically greater, ranging from essentially 0 to 1.75 gallons per hole, and averaged about 0.5 gallons per hole. All crack holes took at least a small grout volume to fill the drill hole, and abutment holes tended to take more grout because visible cracks were more prevalent there. Visible cracks typically took more grout, and at times grout was observed seeping from the cracks in the dam face or from below the dam foundation.

Completed grout holes were eventually filled with a non-shrink grout. Typical grouting details are shown on Figures 5 through 7. A typical example of grout hole layout can be seen in Photo 10.
Figure 5: Typical crack grouting elevation view along lower seal between abutments

Figure 6: Typical crack grouting in plan view
Figure 7: Typical crack grouting in abutments

Photo 10: Typical grout hole pattern at a vertical rustication joint showing injection fittings used for curtain and crack grouting. Note the angled holes used for crack grouting (arrows) and perpendicular holes used for curtain grouting (circles).
Drainage System and Downstream Piping Installation

Historically, seepage has been measured downstream of the dam at weirs situated on the drainage creek downstream of the primary spillway stilling basin. Gallery seepage has been directed over-ground from the gallery entrance, through an opening in the right spillway wall and into the spillway stilling basin. Because the seepage has been measured in weirs downstream of the dam, additional flow from sources such as over-land flow due to rain and snow events or groundwater is able to enter the channel. As a result, seepage data measured at the weir locations may be at times erroneously high.

The liner drain system also feeds into the seepage measuring weirs described above, but liner seepage is also measured before it enters the channel. Liner seepage is measured at the outflow points in the left and right spillway stilling basin walls. Since no membrane is installed in the area of the gate operators and equipment, the membrane system is separated into two isolated compartments to the left and right of the gate operators. The drains, which were cored through the dam lead to welded stainless steel pipes that carry the water down the stepped RCC downstream face and over to the stilling basin walls where the pipes discharge. The left membrane area has two drains that discharge through the left stilling basin wall, and the right membrane area has one drain that discharges through the right stilling basin wall. Photo 11 shows one of the drains on the left side, and Photo 12 shows the two drains penetrating the left spillway wall.

Photos 11 and 12: Downstream stainless steel piping protruding from downstream RCC steps and installed in a trench before backfilling (left photo). Two drains from the left side of membrane area daylight through the left spillway wall. Direct measurements using a bucket and stopwatch are taken here for left side drains (right photo).
Photo 13: Grindstone Canyon Dam with 59,000 square feet of membrane installation complete. Note the area to the left of the primary spillway where membrane was not installed at the gate operator locations. Two separate membrane areas were created in this installation.

**Construction Challenges**

**Ogee Crest Swingstage Access**
In the area of the dam where the ogee-crest primary spillway is located, access for installing anchors, and designing the beam system to support the swing stages used to install the liner on the dam face proved a challenge. The anchor system was designed specifically for the shape of Grindstone Canyon Dam’s ogee weir crest. An independent fall arrest life-line system was installed for worker access to this area. Note the fall-protection cable running in the top left corner of the above picture. This cable is structurally anchored at each end, and is designed for safe access when moving swing stages to new locations. After installation was complete, a decision was made to leave the life-line system in place for any future needs. Photo 14 shows the installation of the beam system used to support the swing stages.

Photo 14: Workers install anchorage for swing stages that access the upstream dam face in the ogee-crest area of Grindstone Canyon Dam. Specially designed beam anchorages were installed across the crest to accommodate moving the swing stages.
**Drilling drain holes**

During design, four drain holes, each approximately 50 feet long, were planned for installation. Since the membrane system is separated into two isolated compartments. There were to be two drains per compartment for redundancy in case one drain developed problems in the future. Photo 15 shows a drain being drilled.

![Drilling drain holes](image)

**Photo 15: Core drilling the right most drain in the left membrane area. In this case, core drilling is conducted from a swingstage over the water.**

The drill string was specially fitted with a cutting head that could be removed after the drill string daylights on the downstream RCC steps. The stainless steel drill string stays in place and functions as the drain pipe in order to prevent collapse of the drain. During drilling of the left drain in the right membrane area, voids and loose material caused the drill string to break and the loss of 8 feet of the stainless steel pipe. This drain hole was abandoned by filling with grout, and discussions of how to proceed led to the elimination of this fourth drain pipe. Since the right side membrane area is smaller, fewer drain locations are necessary.

**Bentonite layer at foundation and backfill placement**

During excavation of the backfill that had been placed against the dam face, the existing bentonite layer that had been placed at the foundation contact along the upstream toe was generally encountered as expected near the bedrock foundation contact. However, the 4-inch thick blanket that was expected near the foundation contact was thicker in places and the location varied vertically in relation to the foundation contact. The existing bentonite layer was not fully excavated to bedrock in some places. Bedrock was generally observed just upstream of bentonite that was left in place, indicating the bentonite layer was placed over the bedrock contact in many areas. The remaining bentonite was also probed with a metal rod to confirm the underlying material was hard, with all indications being consistent with it having been placed on bedrock. After it was confirmed that bedrock was present, the last of the excavation was generally completed with a smooth-edged excavator bucket to minimize damage to the underlying bentonite and bedrock. The tooth-edged
bucket that was used for most of the excavation tended to break up the bedrock beneath the bentonite, and the loose bedrock would need to be removed before replacing the bentonite.

After cleaning the excavation surface to remove any loose material overlying bedrock or the existing bentonite, the bentonite layer was replaced before replacing the backfill against the lined dam face. The design called for a 4-inch thick bentonite blanket to be placed about 2-feet wide along the entire upstream foundation contact that had been exposed. The blanket was typically deeper and wider than this since extra bentonite had been ordered for the project at a nominal cost to avoid running out and incurring additional shipping costs. The powdered bentonite was shipped in 50-pound bags, which were broken and placed by hand by laborers. Before placing the bentonite, the foundation was cleaned of any loose material down to clean bedrock or to the top of the existing bentonite blanket.

Backfill was placed back to the same level as it was before the liner project began, with placement of the first few lifts being done carefully to avoid disturbing the bentonite blanket. A protective layer of cushioning geotextile and or excess geocomposite liner material was placed over the completed liner on the dam face to protect it from puncturing during backfill placement. Compaction of the backfill was done with the excavator bucket in the lower portion, and when enough width was available, tracking with a dozer or excavator was used. Photo 16 shows the bentonite layer and first lift being placed. Photo 17 shows the dozer working fill up to the liner and protective cover.

![Photo 16: View of the completed liner covered with a protective layer. The bentonite layer is also visible as the lighter colored material upstream of the liner. The first lift of fill is also shown on the right side of the photo.](image-url)
Post Construction Performance

Since the year 2000, the reservoir level had typically been above elevation 6,890 feet; however, starting in late 2010, the reservoir level started dropping due to water availability and hit a low of about elevation 6862 feet. It recovered to about elevation 6876 feet in September 2014 before it was lowered for the liner construction project starting in October 2014. The reservoir remained below about elevation 6,855 feet throughout construction in the spring of 2015, with the low water elevation of 6,852 feet occurring in May of 2015.

Post-construction filling of the reservoir began in June 2015 and on about September 15, 2015, the reservoir storage level reached the lower watertight seal at elevation 6,860 feet and started submerging the liner system. Following the installation of the liner system, seepage observations were made of the liner drains. Starting in November 2015 when the drains started to seep, measurements were collected from the liner drain outfalls on the downstream face using a bucket and stopwatch. To date, there has been up to 27 feet of water impounded above the submersible seal representing 35% of the total liner area. The peak liner drain seepage discharge of 7 gallons per minute (GPM) from the liner drain system was observed in December 2015 with 21 feet of water impounded above the lower seal. Reservoir levels generally increased through January to about 27 feet over the lower seal before decreasing, but liner drain seepage rates decreased during this period of rising reservoir levels to around 4 GPM, at which time the lower seal was under about 25 feet of water. Possible explanations for the decrease in seepage through the liner drains from higher initial rates with increasing reservoir head may be explained by the dewatering of the RCC dam body through the upstream face, or possibly a decrease in seepage rates under the dam foundation corresponding to the hydration of the bentonite layer installed just below the lower seal at the dam foundation contact in the groins. As the bentonite expands and its hydraulic conductivity decreases, seepage around the submersible seal through the foundation contact could decrease.
Currently, at the time this paper was written in August 2016, the liner drain seepage has continued to decrease and is currently at 0.1 GPM with about 18 feet of head on the lower seal. The Village of Ruidoso continues to collect seepage data and monitor instrumentation at Grindstone Dam, and to work closely with the New Mexico Office of the State Engineer. The goal of sealing the upper reaches of Grindstone Dam has been achieved, with a typical seepage through the lined area being less than 1 GPM total for the three liner drains. This flow rate amounts to a water height behind the liner of less than 2 feet, which is just enough to reach the drain inverts above the lower seal. Therefore, there is a height of nearly 63 feet of upstream face behind the liner that is dry after lining.

Project Acknowledgments

The geocomposite liner installation began on March 4, 2015 and the project was substantially completed by June 20, 2015, at which time the reservoir began storing water. Carpi and their subcontractors worked safely and efficiently, with no accidents or lost time on the project to complete the liner installation on budget and on schedule. The authors wish to thank the Village of Ruidoso for their cooperation and participation as a member of a successful project team.

References