CONSTRUCTION DEWATERING AT SALUDA DAM; DESIGN, TESTING, AND IMPLEMENTATION

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ABSTRACT

The proposed Remediation of Saluda Dam, located approximately ten miles to the west of Columbia South Carolina and owned and operated by South Carolina Electric & Gas Company (SCE&G), consists of a 5,500-foot-long Rock Fill Berm and a 2,200-foot-long Roller Compacted Concrete (RCC) Berm. This combination RCC and Rockfill Berm will be constructed along the downstream toe of the existing 200-foot-high earth embankment dam. Should the existing Dam fail during a seismic event, the combination Rockfill and RCC Berm will serve as a backup dam to prevent an uncontrolled release of Lake Murray. Extensive foundation excavations into the residual soil or to bedrock at the toe of the existing Dam are required to facilitate the construction of the RCC and Rockfill Berm. To maintain an adequate factor of safety against slope instability for the existing Dam during construction, the existing phreatic surface within the Dam needs to be lowered substantially by dewatering. Based on the hydrogeologic conditions at the site, Paul C. Rizzo Associates (RIZZO) determined that the dewatering system should consist primarily of deep wells and eductors. Numerous components of this system that have been installed are currently operating to lower the phreatic surface within the Dam and downstream foundation excavation. Engineering analyses consisting of analytical models and finite element analyses were utilized to estimate the approximate spacing and flow rate required for the deep wells and eductors. Early indications are that the dewatering system will be successful in dewatering the existing Dam so that the construction can proceed without delay.

INTRODUCTION

Saluda Dam, owned an operated by South Carolina Electric & Gas Company (SCE&G), impounds Lake Murray, which is one of the largest man-made lakes in North America. The Dam is a semi-hydraulic fill structure constructed in 1930 following typical “puddle dam” construction technology. This type of construction resulted in significant seepage through the Dam upon filling, which required placement of riprap benches and the installation of an extensive network of seepage collection drains on the downstream slope of the Dam to control seepage after the initial construction of the Dam. The Dam must be remediated to meet changes in earthquake safety criteria as directed by the Federal Energy Regulatory Commission (FERC). The proposed Remediation of Saluda Dam, located approximately ten miles to the west of Columbia South Carolina, consists of a 5,500-foot-long Rock Fill Berm and a 2,200-foot-long Roller Compacted Concrete (RCC) Berm. This combination RCC and Rockfill Berm will be constructed along the downstream toe of the existing 200-foot-high earth embankment dam. These structures

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will impound Lake Murray in the event that an earthquake causes extensive damage to the existing Dam. Accordingly, both the RCC Berm and Rockfill Berm will serve as the primary water retention barrier, and consequently, they must be constructed on competent foundation materials. Therefore, extensive foundation excavations into the residual soil or bedrock encountered at the toe of the existing Dam are required to facilitate the construction of the Rockfill and RCC Berm. To maintain an adequate factor of safety against slope instability for the existing Dam during construction and to provide dry working conditions, the existing phreatic surface (i.e., water levels) within the Dam needs to be lowered substantially by dewatering. This Paper describes the engineering analysis, design, field-testing, and implementation of the dewatering system for the Remediation of Saluda Dam.

**REMEDIAL DESIGN CRITERIA**

The excavations for the Rockfill Berm foundation will extend to competent residual soil as determined from Standard Penetration Test blow counts obtained from borings drilled along the toe of the Dam. As such, the foundation excavations will extend below pipeable and pervious materials to preclude piping and limit seepage, and to preclude foundation liquefaction. The RCC Berm will be constructed on competent rock as typically defined by the elevation where coring operations were commenced in the field.

RIZZO established a criterion to avoid excavation of the sluiced embankment material. Furthermore, no excavation will be made within the limits of the Saluda River. To meet these criteria in the field, foundation excavations will extend into the residual soil at the toe of the existing Dam.

**Slope Stability During Construction**

The length of most of the proposed excavations at the toe of the existing Dam is restricted to about 250-feet to limit the length of the existing Dam subjected to a potential for a reduction in factor of safety against slope instability. These proposed excavations into the residual soil at the toe of the Dam have been designed for a factor of safety against slope instability of 1.5 for local, global, breach, and intermediate failure circles. Slope analyses were performed using shear strength parameters of the residual and embankment soils determined by consolidated undrained triaxial compressive strength performed on undisturbed samples.

Slope stability analyses were performed for Dam cross sections spaced every 100-feet of the 7,800-foot-long Dam to determine the target phreatic levels within the Dam. The analyses considered the pore water pressures within the excavation slope determined from an interpreted phreatic surface. As determined in previous work performed by RIZZO, the factors of safety calculated using pore pressures estimated from seepage analyses are 0.1 to 0.2 higher than assuming the pore pressures are simply proportional to the vertical distance from the phreatic line. Seepage analyses consider the head losses of the water seeping through the Dam, which results in lower pore pressures in most places.
Therefore, the slope stability results used to determine the target phreatic levels have an additional degree of conservatism.

A typical slope stability cross section is shown on Figure 1. This cross section includes both an existing and target phreatic surface along with critical slope stability failure circles.

![Figure 1. Typical Slope Stability Cross Section](image)

A monitoring program will be initiated during construction to ensure the global and local stability of the existing Dam and the excavation slopes. This program will consist of surveying existing monuments at the existing Dam, measuring inclinometers, and evaluating water level data from piezometers, which are installed along the western side of the toe excavation. This monitoring program will include emergency actions that will be triggered in the event that unacceptable displacements of the Dam or excavation slope are detected.

**Dewatering System**

The design criteria for the excavations that terminate in the residual soil (Rockfill Berm) are that the dewatered phreatic surface must be a minimum of five-feet below the proposed bottom of the excavation and that the hydraulic head in the underlying fractured rock should provide a minimum factor of safety of 2.0 against a blowout failure. We postulate that a blowout failure could occur if seepage forces from groundwater flowing from the underlying rock into the residual soil exceed the weight of the overlying soil.

Our design criteria for excavations that terminate at the contact between the residual soil and the fractured rock (RCC Berm) are that the dewatered phreatic surface within the rock must be a minimum of five-feet below the proposed bottom of the excavation. In
addition, the groundwater must be lowered to a level that ensures the stability of the excavated slope during construction as determined by slope stability analyses.

**HYDROGEOLOGIC SETTING**

Saluda Dam is located within the Piedmont region of South Carolina. Igneous and metamorphic rocks, commonly having a mantle of residual soil, characterize the bedrock of the region (LeGrand, 1988). The Saluda Dam site lies within the Modoc Zone, a ductile shear zone within the Carolina Terrane (formerly known as the Carolina Slate Belt) (SCDNR, 1997). Lithologically, the subsurface below Saluda Dam is composed of high-grade metamorphic rocks consisting of quartz-microcline gneiss, and quartz-mica schist. These rocks have been emplaced in low-grade metamorphic rock by tectonic processes. Both of these units show ductile and brittle deformation and have been intruded by numerous pegmatites and dike as well as a small granitic body. These rocks have been subsequently deformed at least twice since their emplacement resulting in folds and additional fractures. These fractures tend to facilitate ground water flow, whereas the dikes and pegmatites tend to inhibit ground water flow. In general, fracture flow within the rock is highly variable, but where ground water is close to the surface, the underlying rock body is tight and where the rock is well fractured, ground water levels are significantly lower. Distinct unfractured and fractured zones in the bedrock occur along the length of the Dam.

A characteristic feature of the region is a mantle of residual soil, which covers the bedrock in most places (LeGrand, 1988). The thickness of the residual soil typically ranges from 0 to 100-feet. The mantle of residual soil is a true hydrogeologic unit that has an important impact on the groundwater conditions.

As shown on Figure 2 (LeGrand, 1988), a composite two-media system characterizes the groundwater flow in the region. The underlying bedrock, which tends to be fractured, is the chief avenue for groundwater flow and the overlying residual soil and weathered rock provide an intergranular medium through which recharge and discharge of water from the fractured rocks occur (LeGrand, 1988). In other words, the operative permeability for the groundwater system is that of the fractured rock and the specific yield is that of the residual soil (Trainer, 1988).
Figure 2. Typical Subsurface Cross Section in Piedmont and Blue Ridge Regions

The residual soil is predominately sandy clay while the underlying fractured bedrock generally grades downward into unfractured rock below a depth of about 325-feet. Typical values for hydraulic conductivities for the fractured bedrock range from $1.2 \times 10^{-4}$ to $2.4 \times 10^{-3}$ cm/sec (Heath, 1988). The sustained yield of most wells within the bedrock ranges from 5 to 106 gpm (LeGrand, 1988).

The old residual soil layer can form an important flow boundary that can significantly affect ground water flow. In places, this clayey material acts as an aquitard. Where this old soil layer is inclined on the flanks of hills of the pre-Saluda Dam topography, it can significantly affect the flow direction of water within the Dam fills. The steeply inclined soils near the old riverbed redirect ground water flow toward the river instead of perpendicular to the Dam. Ground water flow through the Dam is higher in areas where old creek beds exist and less where there are older stream divides.

Alluvial deposits, including sand, gravel and cobbles, were encountered in the river valley. As expected, extracting groundwater from these more permeable materials is easier than from residual soils.

**Previous Work**

An evaluation of the hydrogeologic characteristics of the southern side of the site was prepared by an SCE&G consultant in conjunction with a proposed 115-acre landfill expansion located approximately 2000-feet to the west (downstream) from Saluda Dam on the southern side of the Saluda River. This evaluation indicated that the subsurface conditions of the landfill site are characterized by surficial soils consisting of clayey sands, sand, silt, clay, and fine gravel associated with the local saprolite. This surficial material is underlain by fractured bedrock consisting of a gneiss-schist complex and mantling schist. Two aquifers were identified at the site: a shallow aquifer within the saprolite and a deep aquifer within the bedrock. The bedrock transmits groundwater via...
fractured flow. Furthermore, the saprolite may act as a confining layer to the deep regional aquifer since the hydraulic head in the fractured rock is greater or equal to the hydraulic head in the overlying saprolite aquifer.

A University of South Carolina Master’s thesis prepared by David Herting (1979) provides hydrogeologic information regarding 63 local domestic water wells within a 10 square mile area that includes the southern downstream toe area of the Dam. This study evaluated the effect of different topographic, rock type, and fracture trace locations on total depths, casing lengths, productivity’s, and reported yields of domestic water wells. The reported yield for these wells ranged from 1 to 50 gallons per minute and the total depth of these wells ranged from 55 to 487-feet with an average total depth of 165-feet.

FIELD INVESTIGATIONS

Pumping Tests

Pumping tests were performed by RIZZO at the site at two locations along the downstream slope of the Dam. The purpose of these tests was to determine the appropriate hydrogeologic data necessary to develop a preliminary design of the dewatering system. These tests included the installation of two pumping wells and nine observation wells. Relevant conclusions are summarized as follows: 1) The hydrostratigraphic horizons evaluated in the pumping tests include fractured bedrock consisting of gneiss and schist, residual soil consisting of silty clay, and embankment materials consisting of silty sand to sandy silt; 2) hydraulic communication exists between the fractured bedrock, residual soil, and embankment soils; 3) the fractured bedrock is semi-confined at the location of the pumping tests and behaves like a confined aquifer for all practical purposes; 4) specific capacity values range from 0.28 gallons per minute per foot (gpm/ft) for Pump Well No. 1 to 1.27 gpm/ft for Pump Well 2. The specific capacity values yield maximum sustainable pumping rates ranging from 45 gpm to 206 gpm.

Packer and Falling Head Tests

Packer tests were performed by RIZZO at two locations near the Powerhouse in April of 2000 as part of a hydrogeological study of the site. Testing was performed on 5-foot intervals within the fractured schist encountered in two borings. These tests provided estimates of the hydraulic conductivity of the fractured rock aquifer.

Falling head tests were also performed by RIZZO in April of 2000 at nine piezometers screened within the Dam and underlying residual soils. These tests provided in-situ measurements of the hydraulic conductivity of the residual soil within the screened interval of the piezometer.
GENERALIZED HYDROGEOLOGIC CONDITIONS

Our generalized interpretation of the hydrogeologic conditions at the Dam and downstream toe is presented on Figure 3. To summarize, the conditions at most sections of the Dam can be represented by four hydrostratigraphic horizons: embankment materials, residual soil, fractured rock, and unfractured rock. Representative ranges of hydraulic conductivity values of these horizons as measured in the field by the pumping, packer, and falling head tests are also shown on Figure 3.

![Figure 3. Generalized Hydrogeologic Conditions at Saluda Dam](image)

The radius of influence of a deep well is an important parameter that is used to determine the optimum well spacing for dewatering. Therefore, we have estimated the radius of influence by plotting the observed drawdown in the observation wells as the ordinate versus the log of the distance from the pumping well to the observation well as the abscissa (Driscoll, 1986; Powers, 1991). The radius of influence is calculated as the distance at zero drawdown (i.e., abscissa intercept of the regression line). Calculated radii using regression analyses for the drawdown observed in rock range from 500-feet for Pump Test 1 to 900-feet for Pump Test 2. Calculated radii using regression analysis for the drawdown observed in soil range from 325 to 750-feet.

EXCAVATION GEOMETRY AND PLAN

To provide an adequate foundation for both the RCC Berm and the Rockfill Berm, the excavations will extend down to competent residual soil for the Rockfill Berm and to competent rock for the RCC Berm. The excavation depths range from a minimum of 10-feet to 67-feet below the existing ground surface. Based on the water surface elevations provided by the existing piezometers, the residual soil will need to be dewatered to a prescribed depth for all the excavations made for the Rockfill Berm while the entire thickness of the residual soil and the underlying rock will have to be dewatered for the excavations made for the RCC Berm.

The remedial construction will consist of 25 separate excavations along the toe of Saluda Dam to construct the RCC and Rockfill Berms. These excavations are labeled as N for
the Rockfill Berm to the north of the river, S for the Rockfill Berm to the south of the Saluda River, and C for the RCC Berm. Most of the excavations are limited to a maximum length of 250-feet and excavation widths (measured perpendicular to the longitudinal axis of the Dam) will range from 110 to 315-feet. To minimize the time that an excavation is open at locations deemed “critical” in terms of dam safety, the cells will be excavated and backfilled by construction crews working 24-hours per day, 7-days per week.

DEWATERING ANALYSIS

Engineering analyses were performed to provide ballpark estimates of the expected flow rate and spacing required for the deep wells.

Analytical Models

Using the method of image wells described in Mansur and Kaufman (1962), four analytical models were used to analyze the dewatering of the excavations: line source, confined aquifer; line source, unconfined aquifer; circular source, confined aquifer, circular source, unconfined aquifer. The analytical models relate change in head to flow rate for confined and unconfined aquifers subjected to line or circular sources. Computer spreadsheets were utilized to perform the necessary calculations.

Calculations were performed using all four equations for a line of deep wells spaced at 100-foot on center for a 300-foot-long by 300-foot-wide excavation. A range of equivalent hydraulic conductivities for the residual soil and fractured bedrock aquifer were utilized in the analyses (i.e., 1.0x10^{-4}, 5.0x10^{-4}, and 1.0x10^{-3} cm/sec). Since the analytical models used for a line source do not account for a sloping phreatic surface, we have used an equivalent length from the pumping wells to the line source of 200-feet as an attempt to account for the characteristics of the phreatic surface in the model.

The results from these analytical calculations indicate that the required flow rate to dewater the excavations using deep wells will range from 6 to 130 gallons per minute.

Finite Element Analyses

As an additional check on the flow rates required to dewater the toe excavations and to evaluate the influence of hydraulic conductivity differences between the embankment soil, residual soil, and fractured rock; a two-dimensional finite element analysis of the Dam and toe excavation was performed using the SEEP2D computer program (Biedenharn and Tracy, 1987). SEEP2D is two-dimensional finite element groundwater model developed by the U.S. Army Corps of Engineers Waterways Experiment Station to solve steady state seepage problems. This model is designed to model on two-dimensional groundwater flow problems such as cross sections of earth dams or levees.
A two-dimensional finite element model of the Dam at a cross section approximately 500-feet to the north of the powerhouse was developed as shown on Figure 4. This model includes three horizons: embankment materials, residual soil, and fractured bedrock. In reality, Saluda Dam consists of various zones, which may have different hydraulic conductivities. The embankment material was modeled as a uniform material in this analysis for simplicity. A 300-foot-wide, 40-foot-deep excavation was included at the downstream toe of the Dam to model the proposed excavation at this Station of the Dam. A constant head boundary condition of 358-feet was prescribed along the upstream face of the Dam and along the subsurface materials along the headwater side of the model. A constant head boundary condition of 200-feet was prescribed along the subsurface materials along the tailwater side of the model. A no flow boundary condition was prescribed along the base of the model. To provide an estimate of the flow required to dewater the excavation, an exit face boundary condition was prescribed along the bottom of the toe excavation and the downstream face of the Dam. An exit face boundary condition implies that the total head is equal to the elevation head and that the free surface will exit the model along this face. Therefore, the finite element analysis calculates the groundwater that flows into the excavation.

Sensitivity analyses were performed by varying the hydraulic conductivities of the embankment materials (10⁻³ to 10⁻⁴ cm/sec), residual soil (10⁻⁴ to 10⁻⁵ cm/sec), and the fractured rock (10⁻³ to 10⁻⁴ cm/sec). As expected, the maximum flows into the excavation were calculated assuming a hydraulic conductivity of 10⁻³ cm/sec for the fractured rock. These calculated flows range from 0.0 to 1.2 gpm/foot or 213 to 293 gpm for a 250-foot-long excavation. The minimum flows into the excavation were calculated assuming a hydraulic conductivity of 10⁻⁴ cm/sec. These calculated flows range from 0.12 to 0.42 gpm/foot or 30 to 105 gpm for a 250-foot-long excavation. A typical flow net calculated using the SEEP2D model for semi-confined aquifer conditions is shown on Figure 5.
Interpretation of Results

The analytical models used to determine ballpark estimates of the required flows for dewatering are for idealized conditions as described by the underlying assumptions for each model. These conditions are primarily controlled by the material properties of the subsurface materials as well as the location and geometry of the excavation with respect to the groundwater conditions. Therefore, the “actual” conditions at the toe of the Dam at a single location may be a combination of the conditions required by each analytical model. For example, the fractured rock aquifer may be essentially confined at some areas of the Dam, while at other locations it may be essentially unconfined, or at some areas it may be somewhere in between. This depends on the relative difference between the hydraulic conductivity of the residual soil and fractured rock. Despite the complexities of the “actual” conditions at the Dam, the analyses essentially bound the solution to the dewatering problem for each excavation and provide reasonable estimates to design the dewatering system for the Project.

Based on the results of both the analytical models and the finite element analyses, we estimate that the total flow required to dewater a typical toe excavation will range from 30 to 300 gpm. The actual flow will depend on the hydraulic conductivity of the fractured rock at the location of each toe location. The degree of confinement will also affect the total flow required to dewater an excavation.

DEWATERING SYSTEM DESIGN

The dewatering system utilized for the Project consists of four components: deep wells, eductors, and to a lesser extent, well points and shallow wells. The following paragraphs present a brief description of each component including their primary function in dewatering the excavations for the Saluda Dam Project.
**Deep Wells**

Deep wells are similar in design and construction to commercial water supply wells. The primary difference is that deep wells are designed to maximize drawdown and flow rate while domestic water wells typically minimize drawdown. Accordingly, each deep well is constructed by advancing a borehole into the stratum to be dewatered. A well screen may include slotted PVC pipe within the water-bearing zone with a surrounding sand filter to prevent the infiltration of subsurface materials and to improve the yield of the well. Finally, a submersible pump is placed within the screened interval of the well.

Deep wells are typically used to dewater sand and/or rock formations or to relieve artesian pressures beneath an excavation. They are particularly suited for dewatering large excavations requiring large rates of pumping and deep excavations for dams, tunnels, locks, powerhouses, and shafts (Mansur, 1973). The two major advantages of deep well systems are that they are typically installed outside of the perimeter of the excavation, which eliminates construction interference, and they can handle large volumes of water in one lift. Some of the disadvantages are that they are not as flexible as wellpoints, require subsurface exploration and testing, are relatively expensive, and require intercepting a fracture to provide good yield in fractured bedrock. In addition, deep well pumps are not always effective for hydraulic conductivities less than $10^{-5}$ cm/sec.

The purpose of the deep wells in Rockfill Berm excavations (i.e., excavations that terminate in the residual soil) is to reduce the hydraulic head in the fractured rock to prevent foundation blowout due to “quick conditions” and to dewater the overlying residual and embankment soils to the maximum extent practical. The purpose of the deep wells in the RCC Berm excavations (i.e., excavations that terminate in the fractured rock) is to dewater the fractured rock and to dewater the overlying residual soil to the maximum extent practical. Approximately 95 deep wells were installed at Saluda Dam by the beginning of cell excavation.

**Shallow Wells**

Shallow wells were used in conjunction with eductors and deep wells in two specific areas: 1) where a thick layer of residual soil within the excavation zone of several cells was experiencing recharge from adjacent ash ponds, and 2) in thick alluvial deposits from the old river valley. Shallow wells consist of 6-inch screens with sand packs in 10-inch boreholes. The boreholes were extended to the top of bedrock and a submersible pump is used to remove the well water. A vacuum system was initially added to facilitate flow to the wells, but it was found that adding a vacuum reduced flow to the shallow wells. In practice, most shallow wells were pumped dry before long and the operation of nearby deep wells kept them dry. Approximately 30 shallow wells will be installed in the two areas discussed above. Maximum initial yield is 10 gpm in shallow wells.
**Eductors**

An eductor well point system uses a venturi to draw groundwater into the well screen and up the riser pipe to the header pipe at the surface (Griffin Dewatering Corporation, 2001).

Eductor wells can be installed by a variety of drilling methods including jetting, wash rotary, Barber Rig, and Versa-Sonic Rig. Jetting is the most efficient way to install eductors and produces a well with the highest yield. However, due to concerns about safety, jetting is not permitted into the sluiced embankment materials. Typical eductor well spacing varies from 10 to 20-feet. In a typical single pipe system, larger diameter pipe (2-inch diameter) forms the well casing and a smaller diameter pipe (1.5-inch diameter) forms the return line. Water is pumped under high pressure down the annulus between the two pipes and is forced through a nozzle and venturi causing a differential pressure gradient between the well screen and surrounding soil. Groundwater is then recovered through the well and into the return pipe. Eductor systems are typically used for dewatering soils of low permeability where the volume of water to be pumped is not that great.

The primary advantage that eductors have over wellpoints is that an eductor system can lower water table by as much as 80-feet from the top of the excavation as opposed to approximately 15-feet for a single stage well point system (Griffin Dewatering Corporation, 2001). This is a major advantage at the Saluda site since the water levels for most excavations need to be lowered by more than 15-feet. The other advantage that eductors have is that they can be installed outside of the excavation thus reducing construction interference. The disadvantages of eductor systems are that they can only handle a limited amount of flow and they have lower efficiency than other pumping methods.

The purpose of the eductor systems is to remove the water from the overlying residual and embankment soil that is not dewatered by the deep wells. Approximately 600 eductors were installed at Saluda Dam by the beginning of the cell excavations.

**Wellpoints**

Wellpoint Systems are particularly suited for fine sands and silts that have a low transmissivity (Driscol, 1986). Well points are essentially screened cylinders typically 1.5 to 2 inches in diameter and are about two-feet long. The screen openings are sized to correspond with the predominant grain size of the soil (Werblin, 1960). Each well point is attached to a riser pipe. The riser pipe is attached to a header pipe at the ground surface.

Wellpoints can also be installed by a variety of drilling methods. Jetting is also the most efficient installation technique for well points, and was used at this site. The total drawdown that can be achieved depends on the partial vacuum or suction lift that the pump can maintain. For practical purposes, the maximum drawdown that can be
achieved by a Wellpoint System is about 15-feet (Driscoll, 1986). Therefore, lowering the phreatic surface greater than 15-feet involves multiple stages of well points at different elevations of the excavation. Typical spacing ranges from 3 to 10-feet. For more pervious soils, closer wellpoint spacing is required to obtain the same drawdown.

The main advantages of utilizing a Wellpoint System are their cost effectiveness and their versatility and flexibility. In addition, wellpoints are a proven technology since they have been developed and used for over 50-years. The disadvantages are limited drawdown per stage (15-feet), limited quantity of flow, and potential for construction interference primarily in multiple stage excavations.

The purpose of the Wellpoint Systems used in this Project is to remove any remaining water not dewatered by the deep wells or the eductors, in our case within two excavation cells so far. Approximately 50 to 100 wellpoints were installed at Saluda Dam by the beginning of cell excavation.

DEWATERING SYSTEM TESTING

At the direction of SCE&G, RIZZO developed contract documents for the installation of the Dewatering System for the Project in early 2002. Since dewatering is critical to the success of the Project and considering the complexities of the subsurface conditions at the site, only pre-qualified dewatering contractors were allowed to prepare bids for the Project. SCE&G elected to separate the dewatering contract from the Remediation construction contract to allow a Full-Scale Testing Program of the Dewatering System in the spring and summer of 2002. This testing program included the installation of 26 deep wells and 25 rock and soil piezometers located along the upstream side of the RCC Berm excavation as shown on Figure 6. Both the deep wells and the piezometers will be used in the actual dewatering effort so the full-scale test did not require installing additional dewatering components that could not be utilized for actual dewatering. The primary test area was planned to be Excavation Cell C-5 located behind the Saluda Hydroelectric Powerhouse. At that time, Cell C-5 was planned to be the first excavation of the Project, but the excavation sequence has since changed.

The results of packer tests performed on deep borings drilled into the fractured rock indicated that significant fracturing occurred at depths greater than 250-feet below the top of rock. Therefore, the deep wells were drilled approximately 200-feet into bedrock at a spacing of 100-feet to maximize the potential of intercepting a fracture within the bedrock. The testing program also included two eductor test areas; one located in residual soil and one in alluvial materials located near the Saluda River. SCE&G awarded the dewatering contract to Griffin Dewatering Southeast, L.L.C. in March of 2002 and work began in April of 2002.

A pumping test was performed on each deep well as part of the installation process. The results of these tests indicate that the maximum flow rate for the 26 deep wells ranged from 3 gpm to 120 gpm. After three months of pumping seven deep wells, the maximum
drawdown observed in the piezometers screened in the fractured bedrock aquifer ranged from 1 to over 65-feet and the maximum drawdown in the residual soil ranged from 2 to 20-feet. Very little drawdown was observed in the piezometers screened in the embankment material.

Two eductor test areas were also installed as part of the testing program. Each eductor test area consisted of ten eductors approximately 50-feet deep installed at a spacing of 10-feet. The performance of the eductors in dewatering the subsurface soils was evaluated by measuring the drawdown in soil piezometers located 25 to 50-feet from the eductor line. The eductor line installed in alluvial soils produced a drawdown of 16-feet after 4 to 5 days of pumping. The maximum yield of these eductors was 7.5 gpm. The eductor line installed in residual soils produced a drawdown of 7 to 10-feet after 30 days of pumping. The maximum yield of these eductors was 1 gallon per minute.

While the design of the Deep Well System is relatively simple, testing of the eductor system continued into the full-scale production to determine if yields greater than 1 gpm could be achieved. Variables such as different filter sand, larger diameter casing, length of screened section, addition of vacuum, pumping pressure, drilling methods and finally spacing were tested and evaluated. We eventually determined that a yield of about 1 gpm per eductor in the clayey residual and embankment soils was to be expected for the given site conditions. Groundwater recovery tests were also performed after extended deep well pumping periods. It was determined that groundwater recovery was faster in overburden soils if deep well pumping was shortened. This indicated that long-term pumping of the deep wells in fractured bedrock was needed to prevent recharge of the lower residual soils.

Figure 6. Full-Scale Dewatering Testing Program
The results from the Full-Scale Test Program indicate that water levels in the fractured rock aquifer and residual soil at the toe of the Dam have been lowered substantially; however, the overlying dam soils have not experienced a drawdown similar to the rock. Therefore, eductors in conjunction with deep wells will need to be used to lower the phreatic surface in the overlying dam soils to achieve the target phreatic lines.

**DEWATERING SYSTEM IMPLEMENTATION**

While the Full-Scale Dewatering Testing Program was in progress, SCE&G solicited bids for the Remediation Construction Project (excluding dewatering) from pre-qualified bidders and subsequently awarded the contract to Barnard Construction Company (BCC) in August 2002. Construction commenced at the site in September of 2002.

The construction schedule developed by RIZZO and included in the Contract Documents suggested that the first excavation would be located behind the Powerhouse (i.e., excavation cell C-5) and would occur in March of 2003 to support work associated with replacing the existing circulating water lines. These lines are used to supply cooling water to McMeekin Station, a coal-fired power plant located along the toes of Saluda Dam. Subsequent excavations would be primarily within the RCC excavation cells (i.e., C-1 to C-9). The Full-Scale Dewatering Testing Program was developed to support this anticipated construction schedule. However, the construction schedule proposed by BCC indicated that excavation would begin at both the north and south rockfill sections. To support the actual construction schedule, RIZZO completed the Full-Scale Dewatering Testing Program and refocused the dewatering effort to both the north and south rockfill sections.

Additional drill rigs were mobilized so that deep wells and eductors could be installed simultaneously at both the north and south Rockfill Berm areas of the Dam. Since the results of the Full-Scale Dewatering Testing Program indicated that the target phreatic surfaces could not be achieved by deep wells alone, most of the efforts in the field were focused on installing eductors and optimizing their performance. Drill rigs were also mobilized to install the soil and rock piezometers required for measuring the water levels within the Dam and along the downstream toe.

In general, dewatering wells and eductors were planned at about 100-feet and 10 or 20 feet intervals, respectively, along the upper edge of cell excavation in the Dam. Actual installation of these deep wells and eductors was impeded by the limited available access, as shown on Figure 6, and by the fact that these were separate operations. The deep wells were usually installed first with a typical drilling time of several days by either a Barber rig or air rotary Odex System. Pumping of deep wells could occur within a week of drilling once development, pump installation, and connection to discharge lines were completed. Two rigs were continually installing deep wells for much of the Project, while up to five rigs were installing eductors. While eductor installation was performed with Barber, wash rotary and jetting rigs, most eductors were installed with Sonic or VersaSonic rigs, which use vibration, sound waves and/or water to advance the boreholes.
Typically about 150-feet of eductor casing was installed daily per rig, but pumping took longer than with deep wells because the rigs were on a bench much longer (which delayed development, installation of eductor bodies, and connection to supply/discharge pipes) and the whole length of a particular string of eductors had to be drilled first. At the south portion of the Dam where the cell excavations were deeper and water from the ash ponds were recharging the residual soil, both deep wells and eductors were installed on the downstream edge of cell excavation. However, because the phreatic/groundwater surface within the cell excavation in these areas was responding slower than required to meet the excavation schedule, “sacrificial” deep wells, eductors and shallow wells were also installed. These wells will be lost during cell excavation but deep wells and eductors outside of the cell excavation will maintain the target phreatic surface until the cells are backfilled.

**SUMMARY**

This Paper describes the engineering analysis, design, field-testing, and implementation of the construction dewatering at the Saluda Dam Remediation Project. The approach used to design the system consisted of a thorough site investigation, rigorous engineering analysis, and a full-scale field-testing program. The dewatering system consists primarily of deep wells drilled into the fractured bedrock and eductors drilled into the overlying residual and embankment soils while some shallow wells and well points were used in specific areas. Early indications are that the proposed dewatering system is successfully lowering the water levels to achieve the target phreatic surfaces required to maintain adequate Dam safety and provide dry working conditions.

**REFERENCES**


