Jet Grouting and Safety of Tuttle Creek Dam

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ABSTRACT

Jet grouting has increasingly become a ground improvement technology used to address seepage concerns and provide strength improvement to soils underlying dams. The technique of jet grouting uses high pressure/volume jet fluids to erode existing soil, evacuate some or most of the soil, and mix the remaining cuttings with cement slurry to form soilcrete. While considered a useful technology, this paper discusses some of the problems that can develop while jet grouting in or below a dam with an operational reservoir and seepage condition. Jet grouting experience at Tuttle Creek Dam indicates concerns with respect to ground fracture; spoil return, column diameter consistency, and homogeneity of resulting soilcrete. Recommendations are presented to increase monitoring of downhole parameters during jet grouting to better understand the downhole pressures and soil response during jet grouting.

JET G ROUTING TECHNOLOGY

Use of jet grout walls as a barrier to water flow under dams and other sites has been used increasingly in recent times. (Croce and Modoni, 2007; Burke, 2007, Martin, et al., 2004; Yilmaz, et al., 2007; and Fang, et al., 2006). Jet grout columns have the potential to provide a continuous wall of relatively low hydraulic conductivity material; however, the effectiveness of jet grout construction can be difficult to assess, particularly when the project involves an operational dam. Usually the reservoir is operating at non-critical conditions (normal pool or non-seismic state) during jet grouting and after construction. However, the cutoff wall is designed for a critical loading condition, and is not fully tested until that condition is experienced. This reduces confidence in the constructed elements for the design event(s).

Jet grouting has been in commercial use since approximately 1975 (Kauschinger, 2008). Currently, the three main jet grouting systems are single, double, and triple fluid systems. In the single fluid system, grout slurry is ejected under high velocity through a horizontal nozzle which works as a cutting or erosion fluid to mix and evacuate soil. The grout slurry mixes with non-evacuated soil and then hardens to create a soilcrete column. Single fluid jet grouting is most effective in loose coarse-grained/cohesionless soils. The double fluid system usually uses air and grout ejected from two different nozzles that are placed opposite each other on the drill rod. However, some double fluid systems use one nozzle and air surrounds the grout to increase cutting/erosion. Double fluid jet grouting is most effective in loose to medium dense coarse-grained soils and some soft fine-grained soils. In the triple fluid system, air and water are used as cutting fluids above the grout nozzle. Separation of the erosion and grout mixing processes is thought to yield more uniform columns. However, Stark et al. (2009) show considerable soil inclusions may be present in large diameter triple fluid columns. Triple fluid jet grouting is believed to be best for eroding and evacuating dense coarse-grained and some fine-grained soils.

For all jet grouting methods, field trials are usually required to establish site-specific jet grout parameters, energy correlations, achievable column diameters, and assessment of quality of treated soil. Because jet grouting does not use positive displacement or a known mechanical excavation and mixing tool, it is more technically demanding and less forgiving than other ground improvement methodologies, such as excavated slurry walls and non-jet assisted soil mixing.

Recent Dam Jet Grouting Trends and Precedence

Initially jet grouting systems were used to create soilcrete columns with diameters of 1 to 3 m (3 to 10 ft). Recent modifications and procedures are being used to construct soilcrete columns with diameters up 5 m (16 ft). To create these larger diameter columns, high velocity jets, extremely high fluid pressure and volumes, slow rotation of the drill string, and slow lift rates are being used to erode and excavate a larger volume of soil. These high pressures can cause ground fracturing of the insitu soils because they exceed the borehole resistance to fracture. With either double fluid or triple fluid methods the use of air is crucial because it is readily mixed with the spoil cuttings and reduces the weight of the borehole column of spoil. The bottom hole pressure is thus only a fraction of the induced fluid pressures, and is only that pressure necessary to lift spoil to the ground surface. However, ground fracturing can occur nearly instantaneously if the annulus between the drill rods and the borehole or casing becomes blocked because air, water, and/or grout are continuously being injected at extremely high pressures during jet grouting.

TUTTLE CREEK DAM

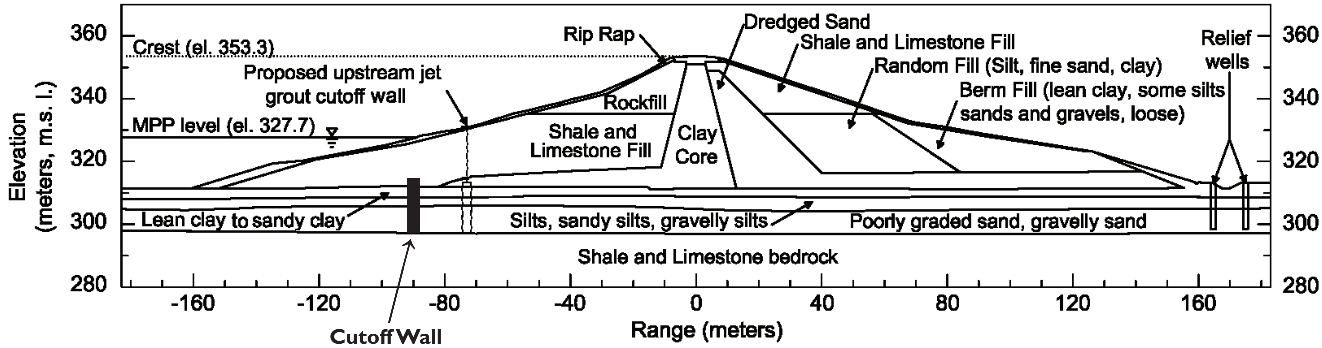
Tuttle Creek Dam is a U.S. Army Corps of Engineers, Kansas City District (USACE) project located on the Big Blue River near Manhattan, Kansas, 200 km (125 miles) west of Kansas City. Tuttle Creek Dam is 41.8 m (137 ft) high earth and rockfill embankment with a length of about 2,288 m (7,500 ft). Details of the fill zones and construction of the dam are available in Lane and Fehrman (1960) or Walberg et al. (2012).

Tuttle Creek Dam was originally designed and constructed in the 1950's prior to the development of recent earthquake engineering technology which accounts for the behavior of materials subjected to seismic shaking. Shortly after the upstream slope failure caused by soil liquefaction at Lower San Fernando Dam in 1971, the USACE began a program to evaluate all of its dams based on evolving technology. Investigations to assess the seismic stability of Tuttle Creek Dam were conducted in the 1980's and 1990's and concluded that seismic rehabilitation (specifically an upstream cutoff wall and slope stabilization and downstream slope stabilization) was required to assure the project could withstand the design ground motion without an uncontrolled release of the reservoir towards downtown Manhattan, Kansas.

Tuttle Creek Dam Foundation Conditions

The dam is founded on native alluvial soils consisting of 2.4 to 8.2 m (8 to 27 ft) of silt (ML) and clay (CL, CH, and OH) underlain by sand, silty sand, and gravely sand to a depth of 12.2 to 24.4 m (40 to 80 ft). The silt and clay deposit immediately below the dam forms a natural low hydraulic conductivity fine-grained blanket that facilitates seepage control by dissipating some of the hydraulic head imposed by the reservoir. This material is referred to as the fine-grained blanket herein. Below the fine-grained blanket are layers of loose, fine to medium sand to silty sand, and sand with silt (SP, SW, SM, SM-SP, and SM-SW) and medium dense to dense, coarse to gravelly sand to bedrock. The fine to medium sand and coarse to gravelly sand deposits typically vary in thickness from about 7.6 to 18.3 m (25 to 60 ft). Interspersed with the sands are occasional relatively thin layers or lenses of clay and silt typical of recent alluvial deposits. The ground water surface is typically located at a depth of 2.7 m (9 ft) or Elevation 310.2 m (1017 ft) for Stations 30+00 to 50+00 at the downstream toe, but is dependent on reservoir elevation. Walberg et al. (2012) presents a detailed description of the foundation conditions.

Fig. 1 presents a typical cross-section of Tuttle Creek Dam showing the seepage control system which consists of an extended upstream impervious embankment zone, the natural fine-grained soil blanket underlying the embankment, and a downstream relief well system. The natural fine-grained blanket dissipates 13.7 to 15.2 m (45 to 50 ft) of the reservoir head upstream of the impervious zone before it enters the permeable foundation sands. Once the seepage reaches the permeable foundation sands, seepage continues to the downstream toe with less hydraulic head loss because of the high hydraulic conductivity of of the sands and gravels. The majority of underseepage is eventually intercepted by the relief well system along the downstream toe of the dam. Relief well flow discharges into an adjacent collec-



[FIG. 1] Typical dam cross section around Station 50+00 and proposed upstream cutoff wall (thick dark line) and slope stabilization

tor that runs along the downstream toe with several lateral ditches that take the water further downstream. Flumes placed in the lateral ditches show a continuous flow rate of 2,200 gpm (295 ft^3/min = 4.9 ft^3/sec = 0.14 m^3/sec) at multipurpose pool (MPP) elevation of 327.7 m (1075 ft). Relief well flow rate is also pool-level dependent. The relief wells at the downstream toe are critical to protect the dam against foundation erosion and piping. The large earthquake induced deformations predicted for the downstream toe would likely disable the downstream relief wells. With loss of, or damage to, the relief wells, an internal erosion/piping failure of the foundation soils could occur even at the MPP.

TUTTLE CREEK DAM FOUNDATION MODIFICATION PROJECT

The selected seismic retrofit alternative for Tuttle Creek Dam was to stabilize foundation soils without drawing down the reservoir. The initial upstream stabilization included jet grouting to stabilize the liquefiable foundation silty clays and sands below the upstream slope and to install an upstream cutoff wall (depth of approximately 36 m (120 ft) on average) to reduce seepage and piezometric levels to acceptable levels at the downstream toe in case the relief wells were damaged during an earthquake. Downstream slope stabilization was required to stabilize the liquefiable foundation silty clays and sands. Downstream stabilization was to include either soil mixing or jet grouting to reduce downstream slope movement and relief well damage. In September 2005, the USACE entered into a contract with Treviicos South, a ground improvement contractor, to construct the upstream cutoff wall and slope stabilization and downstream slope stabilization/foundation improvement.

The cutoff wall objective was to dissipate sufficient hydraulic head (11.9 m or 39 ft) so the pressure relief system along the downstream toe of the dam was not necessary at MPP because it could become inoperable during or after the design ground motion. The upstream cutoff wall was to be constructed to a minimum thickness of 3.0 m (10 ft) using multiple (at least two) rows of full or partial jet grout columns. The wall was to penetrate a minimum of 0.3 m (1 ft) into the bedrock except in the area of a deep buried channel where a 3 m (10 ft) bedrock socket was required because of the presence of slump blocks overlying the alluvial sands and gravels in the channel. The contract did not specify how the bedrock embedment of 0.3 m (1 ft) and 3 m (10 ft) was to be obtained and did not require predrilling. In addition, it was unclear how the depth of embedment would be measured. It was expected that the depth of embedment would be determined by observations and/or recordings obtained from the Lutz instrumentation system used with the jet grouting equipment. It also was not clear how effective against seepage the cutoff wall/bedrock contact would be because the air-water jets might not be effective in cutting the limestone and hard shales in the bedrock. This seal at the bottom of the cutoff wall is important because an improper seal can lead to reduced efficiency of the cutoff wall with time (Rice et al., 2009a and b). Finally, it should be realized that the depth is usually measured from the depth of the nozzles. Therefore, the drill bit may have to penetrate 1 to 1.2 m (3 to 4 feet) below the bedrock interface to achieve a wall penetration of 0.3 m (1 ft).

One of the major challenges of the upstream jet grouting for both the cutoff wall and slope stabilization was the reservoir had to be maintained, meaning the active foundation seepage previously described (2,200 gpm or 0.14 m^3/sec) would be present during construction and is significantly greater than the seepage in the downstream test area. Treviicos South had recently constructed a cutoff wall under similar conditions at Paso de las Piedras Dam in Argentina which facilitated their understanding of the project. The jet grouting pressures used at Paso de las Piedras Dam (Treviicos, 2007) are presented herein for comparison purposes with the Tuttle Creek Dam pressures.

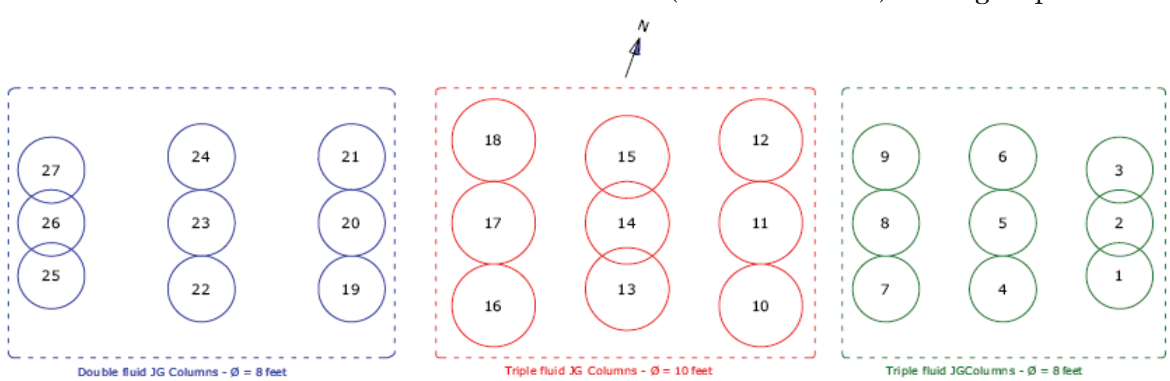
Paso de las Piedras Dam Jet Grout Cutoff Wall

Paso de las Piedras Dam has a central impervious core, a vertical chimney drain, and horizontal drainage blanket downstream of the core (Bustinza et al., 1999; Rattue, 2005, and Treviicos, 2007). A cutoff wall through and below the core was designed because the dam had a history of seepage related issues that resulted in the dam operating at a reduced pool level for many years. The dam is about 30 m (100 feet) high with upstream and downstream slopes of 2.75H:1V and 2.5H:1V, respectively. The foundation soils consist of gravel and silt/sand. A jet grout cutoff wall was constructed between 1998 and 2001 a distance of 12.2 m (40 ft) upstream of the dam axis. The cutoff wall columns are an average of about 22.9 m (75 ft) long, but in deeper areas of the foundation the columns average 39.7 m (130 ft) in length. Approximately 42,000 m^2 of triple fluid jet grout columns were completed (Rattue, 2005). Primary column diameter is about 1.5 m (5 ft) with columns spaced about 2.4 m (8 ft) apart. After the primary row of columns was installed, secondary (2.4 m/8 ft diameter) and tertiary (1.5 m/5 ft diameter) columns were installed at various locations to close windows/gaps in the wall. The top of the wall extends 3.0 to 3.7 m (10 to 12 ft) into the impervious core of the dam. A total of 1,299 jet grout holes were drilled. Only 12 holes had to be abandoned due to verticality, drilling difficulties, obstructions, or other reason.

DOWNSTREAM TEST PROGRAM

In 2006 the contractor and the USACE initiated a test program downstream of Tuttle Creek Dam to prove the viability of jet grout and jet assisted soil mixing technologies and develop appropriate site-specific parameters before beginning production of the upstream and downstream foundation modification elements. The test program site is located about 152 m (500 feet) downstream of the dam and is approximately 56.4 m (185 ft) wide and 103.7 m (340 ft) long). The jet grout and soil mix columns were surrounded by a perimeter cement-bentonite cutoff wall constructed to bedrock which allowed the test section area to be dewatered so column excavation could occur below the groundwater surface after construction. Stark et al. (2009) and Walberg et al. (2012) present additional details on the Downstream Test Program.

The jet grouting test program consisted of twenty-seven jet grout columns in three groups of nine. Columns were installed using both double and triple fluid jet grouting systems. The double fluid jet grout system was used to create columns 19 through 27 with a target diameter of 2.4 m (8 ft) (see Fig. 2). The triple fluid jet grout system was used to create columns 1 to 18 with target diameters from 2.4 to 3.0 m (8 to 10 ft). In each group of double fluid and triple fluid columns, one set of three columns was overlapped to assess column overlap strength and integrity. For example, triple fluid columns 1, 2, and 3 (8 foot diameter) and 13, 14, and 15 (10 foot diameter) are the two groups of triple fluid columns that were overlapped. The double fluid columns 25, 26, and 27 (8 foot diameter) is the group of double fluid



[FIG. 2] Final layout of jet grout columns for Tuttle Creek Dam test program. (Treviicos, 2006)

columns that were overlapped. The water-to-cement ratio for the columns varied from 0.75, 0.9, and 1.0. The columns were about 11 m (36 ft) in length with the top of the columns at about elevation 309.9 m (1016 ft) (3 m/10 ft below ground surface) and the base was at about elevation 298.9 m (980 ft).

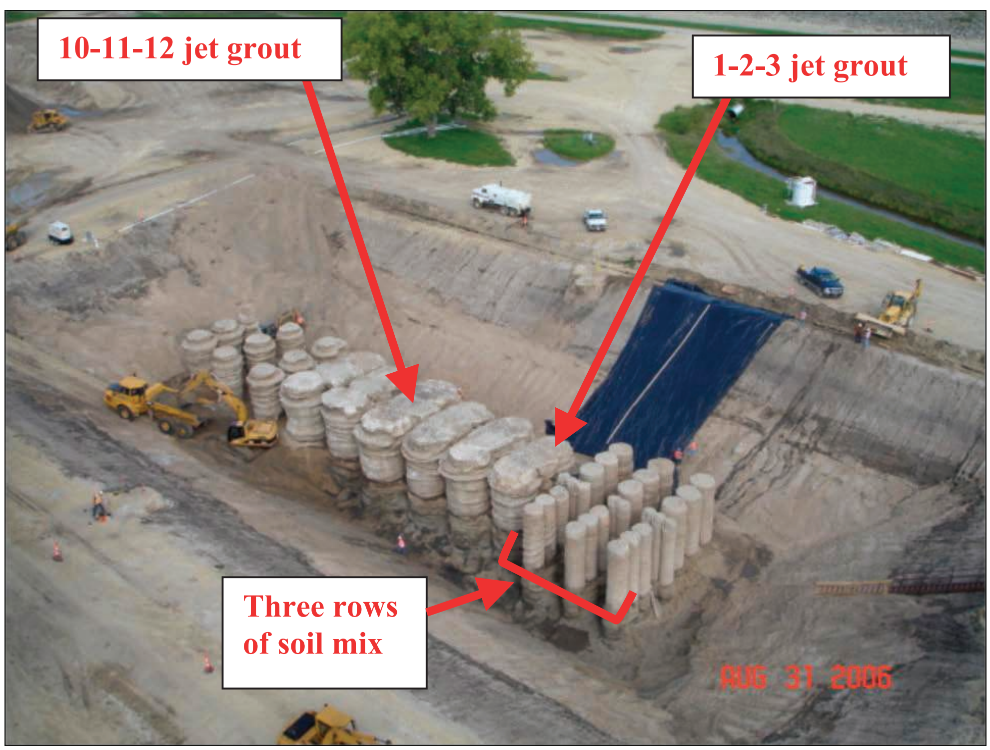
Soil Inclusions in Completed Columns (Performed following Jet Grouting on Dam)

After the jet grout columns were completed and subsequent coring of the columns had been conducted, the groundwater within the cement-bentonite slurry wall was lowered to 11.3 m (37 ft) below ground surface (b.g.s.), i.e., elevation 299 m (989 ft) utilizing two (2) dewatering wells and monitored by six (6) observation wells inside and around the slurry wall. Once the water level was reduced to the target depth of 11.3 m (37 ft), excavation to expose the columns proceeded. Fig. 3 shows the test site near the completion of excavation. The triple fluid columns are closer together because the actual column diameters are much greater than the 2.4 and 3.0 m (8 and 10 ft) target diameters (see Fig. 3). For example, triple fluid jet grout columns 1, 2, and 3 had diameters that ranged from 3.0 to 3.7 m (10 to 12 ft) and columns 10, 11, and 12 had diameters that ranged from 3.7 to 4.3 m (12 to 14 ft).

Following these observations, nine (9) columns were chosen to be sectioned which would expose the inside of the columns to determine column integrity and homogeneity. The upper 4.9 m (16 ft) of the columns were to be removed to allow sectioning of a 3 m (10 ft) segment of the selected columns between about elevation 305 m (1000 ft) and elevation 302 m (990 ft). Column groups 1-2-3 and 10-11-12 of the jet grout columns were chosen for sectioning. Fig. 3 shows that columns 1-2-3 correspond to 2.4 m (8 ft) diameter columns that were overlapped while columns 10-11-12 correspond to 3 m (10 ft) diameter columns that were not overlapped. Both groups were constructed using triple fluid technology. The columns were cut along the line of the group, effectively cutting each column in half.

The sectioning revealed that the cross-section of the jet grout test columns contained more than 40 to 50% native soil that was not broken up and evacuated during the jet grout process (Stark et al. 2009). In other words, 40 to 50% of the sectioned column consisted of chunks of native soil and about 60 to 70% of the sectioned triple-fluid column consisted of native soil. The inclusions were encountered throughout the entire diameter of the column. Most of the inclusions were greater than 75 to 100 mm (3 to 4 inches) which could lead to blockage of spoil return. This large amount of soil inclusion could also have impacted the hydraulic conductivity of the resulting column.

The observed inclusions in the completed jet grout columns included significant amounts and large pieces of both fine-grained (silts and clays) and coarse-grained (fine sands and sands) soils. The following are two explanations for the inclusions: (1) fine grained soils can be difficult to erode and break up into small enough



[FIG. 3] Downstream parametric jet grout and soil mix column groups during excavation and columns to be sectioned

particles that can be evacuated to the surface through the drill rod annulus, especially at low natural water content, and (2) when large diameter columns are being excavated, the excavated roof of the cylindrical cavity may be unstable and may collapse introducing large pieces of soil into the unhardened slurry (Stark et al. 2009). This roof instability probably allowed large slabs of the natural fine-grained blanket material to break off, fall into the slurry, and then not be reduced by the cutting and mixing action of the rotating jets.

In summary, the significant amount and large size of the soil inclusions found in the completed columns suggests that blockage of the annulus between the boring casing and jet grouting drill rod should be expected. This may lead to accumulation of high pressures in the subsurface and likely induce ground fracturing.

Downstream Jet Grouting Parameters and Ground Fracturing

Table 1 presents the triple fluid jet grouting parameters used to construct the two clusters of columns (1, 2, and 3 and 10, 11, and 12) that were sectioned. Table 1 shows that high water, grout, and air pressures were used to construct these columns. This resulted in some air bubbles being observed at the ground surface during downstream jet grouting. In addition, there was anecdotal evidence of ground fracturing during the downstream jet grouting including grout being found in the bottom of the two dewatering wells inside the cement-bentonite cutoff wall. It is recognized that other possible occurrences could have contributed to this condition, such as migration of lean grout through the coarse-grained soil directly above the bedrock, among others.

UPSTREAM CUTOFF WALL CHALLENGES

At the outset, it was recognized that construction of the upstream cutoff wall posed many challenges including column diameter consistency, verticality, variable stratigraphy, drilling through the upstream embankment materials (rock fill), and securing the top of the cutoff wall to the extended impervious embankment material. The following paragraphs briefly discuss these challenges.

Column Diameter Consistency

Based on the Downstream Test Program, it was not certain that the required column diameter and adequate column consistency could be achieved, especially in the dense sands and stiff cohesive layers, to ensure the proposed cutoff wall would have a thickness of 3 m (10 ft) thick, would not have gaps, and could achieve the required 11.9 m (39 ft) head drop to create a safe condition if the relief wells were ren-

[TABLE 1] Triple fluid jet grouting parameters for sectioned downstream parametric columns

Jet Column Group	Group # 1, 2, 3		Group #10, 11, 12	
Column Diameter (m/ft)	2.4/8		3.1/10	
Nozzles (# & diameter [cm/inches])	1 & 0.64/0.25 (w)	1 & 0.76/0.3 (c)	1 & 0.64/0.25 (w)	1 & 0.76/0.3 (c)
Water Pressure (MPa/psi)	45.0/6,525		44,988/6,525	
Grout Pressure (MPa/psi)	25.0/3,626		24,994/3,626	
Air Pressure (MPa/psi)	1.0/145		1,000/145	
Grout Flow Rate (gal/min)	112.3		112.3	
Station time (sec/four cm)	17.5		32	
Rotation Speed (rpm)	4-8		2-4	
Water Quantity ([gal/m] / [gal/ft])	800/244		1,417/432	
Grout Quantity ([gal/m] / [gal/ft])	814/248		1,444/440	
Cement Quantity ([kg/m] / [lbs/ft])	1,848/1,239.5		3,297/2,211	
Specific Energy (MJ/m)	130-140		240-250	
Grout Mix (C/W & B/W)	0.75 & 0.01		0.75 & 0.01	

dered inoperable or damaged. The option of smaller diameter columns and additional rows (allowed in the contract) were viable options, but not options the contractor adopted due to cost concerns.

Variable Stratigraphy

Another jet grouting challenge at Tuttle Creek was the variable foundation stratigraphy and varying grain size distributions. Because of its cohesiveness, clayey material is difficult to evacuate from the boring during jet grouting. The downstream test program demonstrated that the columns contained numerous inclusions, but also that column diameter can be reduced significantly when a cohesive layer is encountered. Brill et al. (2003) show that plastic clays erode as chunks and pieces rather than small particles and that the chunks often lead to clogging of the drill rod annulus and subsequent spoil return blockage. Even within the sand layers there is considerable variation in density/Standard Penetration Test (SPT) blow count values which can cause column variability. However, SPT or density variation in the sands will have less of an influence than the presence of clays because of the difficulty in eroding and breaking-down cohesive material which can result in annulus blockage. Successful jet grouting performance is a challenge with highly variable stratigraphy because jet grout parameters may have to be adjusted to provide different energy levels required to erode and grout different materials.

Securing the Top of Cutoff Wall to Embankment Material

Observations and coring of completed jet grout columns in the Downstream Test Program indicate that a void was usually created at the top of a jet grout column due to "bleed" of the unhardened grout and settlement of the materials involved. It was estimated that 75 to 90% of the completed jet grout columns had a void at the top. All of these voids were filled with grout following construction by topping-off the column. If a void develops at the top of a completed column and is not filled, this can result in a preferential flow path over the top of the column. This flow path would exhibit a larger gradient than is currently being experienced in the foundation sands because a smaller amount of hydraulic head would be dissipated in the impervious zone and/or the natural fine-grained blanket. This could result in unacceptable gradients and flow at the downstream relief well system. Additionally there was concern that the presence of voids could result in progressive erosion back to the reservoir creating an open flow path from the reservoir, across the top of the cutoff wall, into the foundation sands, and to the downstream toe. Such a condition could lead to an unstable seepage condition that also could not be controlled by the pressure relief well system. A similar concern occurred if ground fractures remained open and allowed direct communication between the pervious foundation and the reservoir. These scenarios could occur at high flood pools when implementation of a remedial measure would be difficult resulting in a serious threat to dam safety.

Field observations also suggested that the presence and/or quality of the extended impervious zone was in question. Historic as-built drawings suggest the presence of this zone and refer to the material as "impervious fill". Several investigative borings indicate this zone may have actually been shale "rock fill". Original embankment construction specifications required different placement techniques and compactive efforts for the two different types of fill. This discrepancy was recognized and considered a possible contributor to some of the observations presented herein and made sealing of the cutoff wall to the embankment more suspect.

UPSTREAM JET GROUT CUTOFF WALL PARAMETRIC COLUMNS

The contractor continued the jet grout field trial program on the upstream face of Tuttle Creek Dam to confirm that the jet grouting parameters developed in the downstream parametric columns were applicable to the upstream soils, higher hydraulic head imposed by the adjacent reservoir, and significantly increased confining stresses because of the overlying embankment. This upstream work initiated concurrent with excavation and complete evaluation of the downstream test columns so some of the findings previously discussed in this paper were not yet fully understood as the upstream jet grouting commenced.

Pre-Drilling through Embankment Material for Jet Grouting

Fig. 1 shows the cutoff wall was to be located near the upstream extent of the extended impervious zone. To install the cutoff wall the contractor pre-drilled borings through the upstream rock fill shell (see vertical jagged line above cutoff wall in Fig, 1). The borings extended to a depth of about 20.1 m (66 ft) to the proposed top of the cutoff wall. The initial diameter of the boring was 0.4 m (1.3 ft) and the hole was maintained using temporary steel casing and grout. Before the grout hardened in the boring, a 254 mm (10 inch) diameter PVC casing with no end cap (to prevent the casing from floating back up) was inserted in the boring. Spacers or “spiders” were attached to the PVC casing at various depths in an effort to maintain casing verticality and its location in the center of the boring. These spacers had to span the 75 mm (3-inch) annulus in the 406 mm (16-inch) borehole. It became apparent that the flimsy and variable spacers aligning the casing in the large hole would not be able to maintain casing verticality.

After setting the PVC casing in the unhardened grout, a 203 mm (8 inch) drill bit was used to clean out the PVC casing and advance the boring and jet grout tools to bedrock. If perfectly centered, this results in a 25 mm (1-inch) wide annulus of hardened grout in the casing prior to drilling the hole to rock for the jet grouting of the column. The hardened grout tended to shrink or detach from the casing wall which resulted in relatively large pieces of the grout annulus falling into the hole created for jet grouting. Evidence of the grout annulus falling into the hole is large pieces of curved grout were returned to the surface in the jet grout spoil.

Subsequently, using either sonic or augering drilling methods the contractor also installed ungrouted 254 mm (10 inch) diameter steel casing to the proposed top of the cutoff wall, elevation 314.2 m (1030 ft). A 203 mm (8 inch) drill bit was used to advance the jet grout boring from the bottom of the 254 mm (10 inch) diameter steel casing to bedrock. The steel casing provided a larger return annulus in the casing because hardened grout was not present in the casing, avoided the issue of hardened grout falling into the column, and could be withdrawn upon completion of the column and grout backfilling to the work platform.

The jet grout drill rod diameter ranged from 60 to 114 mm (2.4 to 4.5 inches) depending on the jet grout system being used, e.g., double v. triple fluid jet grout system. The triple fluid system utilized a drill rod diameter of 114 mm (4.5 inches). This created an annulus of about 44.5 mm (1.75 inches) in both the cased and uncased portion of the boring if the 25 mm (1-inch) wide annulus of hardened grout in the casing was still present. If the grout broke from the casing, the triple fluid annulus within the casing increased from 44.5 to 70 mm (1.75 to 2.75 inches). Thus, a soil clump with any dimension of 44.5 to 70 mm (1.75 to 2.75 inches) could cause a partial blockage of the drill rod annulus. The sectioned columns previously described show sand and clay chunks much larger than these dimensions in the completed columns. The annuluses of 44.5 to 70 mm (1.75 to 2.75 inches) correspond to an annulus area of 0.022 to 0.041 m^2 (34.3 to 62.6 in^2), respectively. Thus, the spoil had to be evacuated through an annulus area of less than 0.041 m^2 (62.6 in^2). Of course larger column diameters require a larger volume of material to be eroded and evacuated through the annulus.

It is also recommended that the PVC or steel casing should extend to the bottom of the borehole, i.e., top of jet grout column, to provide the best connection possible to the top of the cutoff wall. Extending the casing to the bottom of the hole would provide the best conditions for a stable hole at the critical connection between the embankment materials and the top of the wall. This connection is important because if the connection is lost, leakage during jet grouting can occur at this location due to the high pressure air, water, and/or grout. This is discussed below as a possible cause for the observed air and possibly grout observed in Tuttle Creek Reservoir during upstream jet grouting. Due to elevation variations, equipment must be available to adjust the length of the PVC or steel casing. For example, the PVC casing was furnished in 6.1 m (20 ft) sections so additional 1.8 m (6 ft) of casing had to be added to reach a depth of 20.1 m (66 ft). If not, a gap of 1.8 m (6 ft) was created which could serve as a release point for high pressure air, water, and/or grout during grouting.

After installation of the casing to a depth of 20.1 m (66 ft) and drilling an 203 mm (8 inch) boring inside the 254 mm (10 inch) diameter PVC or steel casing to accommodate the jet grout drill rod, the grouting drill rod would then drill past the bottom of the cased hole to the desired depth of the column bottom, i.e., to bedrock, a depth of approximately 36 m (120 ft) on average for the bottom of the cutoff wall. The jet grouting would then start from the bottom of the column and move upward to create the soilcrete column.

Observations during Upstream Jet Grouting of Triple Fluid Column S2-A3

Jet grout construction of triple-fluid column S2-A3 was attempted on Tuesday, 11 July 2006 using the jet grouting parameters in Table 2. The jet grouting started at 14:30 and ended about 145 minutes later at 16:55. Jet grouting was to occur from elevation 37.6 m (123.4 ft) to 27.9 m (91.5 ft). The contractor started stroking the hole to regain spoil return but only partial return was restored. At 15:20 the contractor reduced the water and grout pressure to 100 bars (1450 psi) and continued stroking the hole. At 15:35, the contractor reduced the water and grout pressure to 70 and 30 bars (1015 psi and 435 psi), respectively, and continued stroking the hole with the nozzles just below the PVC borehole casing. At 15:50, the contractor regained spoil return. However, spoil return was lost, but regained return shortly after, at the following times and depths: 16:15 (31.2 m), 16:30 (29.9 m), 16:45 (28.5 m) and 16:50 (28.3 m).

At 16:55, contractor and USACE personnel noticed vigorous air bubbling in the reservoir about 15.3 m (50 ft) upstream of the work area. After this observation, the contractor stopped jet grouting, pulled the drill rods, and grouted up the drill hole around 17:00. The arrows in Fig. 4 indicate large areas of intense air bubbling in Tuttle Creek Reservoir in the vicinity of jet grout column S2-A3. The bubbling air in the reservoir appeared from approximately Station 65+00 to 67+00 and suggested that ground fracture may have occurred. The air bubbling continued for almost 48 hours after jet grouting indicating a large volume of air was stored, probably in the pervious foundation materials, prior to release into the reservoir.

The loss of spoil return and the presence of high air, water, and grout pressures can fracture insitu materials as pressure builds up. For triple-fluid column S2-A3, water and grout were being injected at the rate of about 341 and 448 liters/minute, respectively (see Table 2), while the air pressure was being maintained at 9 bars (130.5 psi). While bubbles of air escaping in the reservoir may have indicated damage to the embankment and/or the natural fine-grained blanket, the first manifestation of damage, e.g., increased seepage or gradient, can occur long after fracturing has initiated, particularly when the pool level increases. As a result, the USACE carefully monitored the downstream relief wells and toe area for signs of erosion, sand boils, and increased seepage for three weeks after S2-A3. Piezometers were also monitored to identify any changes in the hydraulic heads in the foundation soils. No abnormal reading or observations were observed so the heightened surveillance was reduced after three weeks. No higher than expected piezometric levels have been observed since completion of all construction at the dam up to a pool level approximately 9.2 m (30 feet) above MPP.

Table 2 also presents the specific energy induced by the jet grouting (EJG) which can be related to column diameter (Schlosser, 1997). For double-fluid jet grouting the specific energy is calculated by adding the energy imparted by the grout and air (see Equation (1)) (Pagliacci et al., 1994). For triple-fluid jet grouting the specific energy is calculated by adding the energy imparted by the water and air (see Equation (2)).

$$E_{JG-Double} = \frac{\left(P_G * Q_G + P_A * Q_A\right)}{V_S} \quad [1]$$

$$E_{JG-Triple} = \frac{\left(P_W * Q_W + P_A * Q_A\right)}{V_S} \quad [2]$$

where P_G, P_W, and P_A are the grout, water, and air pressures in MPa, respectively, Q_G, Q_W, and Q_A are the grout, water, and air flow rates in m^3/hour, respectively, and V_S is the withdrawal speed in m/hour. Use of these units will yield values of E_{JG} in MegaJoules/meter.

Restart of Upstream Jet Grouting and Double Fluid Column S2-A7

Because of the critical nature of the project, the contract specifications required that the work had to be performed without inducing ground fracturing. Suspension of jet grouting was directed by the USACE for assessment of hydraulic fracturing and to develop techniques for controlling downhole pressures. Discussion with the contractor revealed no means to accurately measure downhole pressure in the columns nor could they demonstrate that triple fluid jet grouting with increased erosive capability would not suffer blockage and cause hydraulic

[TABLE 2] Triple fluid jet grouting parameters for upstream parametric column S2-A3 and Paso de las Piedras Dam

Jet Grout Parameters	Tuttle Creek Dam S2-A3 in Foundation Sands	Paso de las Piedras Dam Primary Columns in Silt and Sands
Air Pressure (bars)	9	12 - 20
Air Pressure (MPa)	0.9	1,200 – 2,000
Air Pressure (psi)	130.5	174.0 – 290.1
Air Flow Rate (liter/min)	7,700	12,000 –18,000
Air Flow Rate (m^3/hour)	462.0	12,000 –18,000
Water Pressure (bars)	440	450
Water Pressure (MPa)	44.0	45,000
Water Pressure (psi)	6,381.7	6,526.7
Water Flow Rate (liter/min)	341	170
Water Flow Rate (m^3/hour)	20.5	170
Grout Pressure (bars)	300	200 – 220
Grout Pressure (MPa)	30.0	20,000 – 22,000
Grout Pressure (psi)	4,361.1	2,900.8 – 3,815.8
Grout Flow Rate (liter/min)	448	180
Grout Flow Rate (m^3/hour)	26.9	180
Withdrawal Rate (m/hour)	8.4	
Specific Energy (MJ/m)	156	32 - 71
Nominal Column Diameter (ft)	Minimum 8.5	5.2 – 7.9
Nominal Column Diameter (m)	Minimum 2.6	1.6 – 2.4

fracturing. A revised plan of action for spoil blockage allowed a loss of spoil return for 30 seconds before action to remove the blockage, e.g., stroking the hole with the jet grout drill rod, had to be undertaken. Upstream jet grouting resumed Friday, 14 July 2006 with double-fluid column S2-A7. During jet grouting of column S2-A7, air bubbles again were observed in the reservoir just upstream of the S2-A7 location. The air bubbles occurred shortly after spoil return became intermittent. After observation of the air bubbles, jet grouting was terminated and the hole grouted. Spoil return was regularly lost for as much as 10 seconds but never for 30 seconds. Thus the plan of action was not implemented before air bubbles were observed in the reservoir because the maximum blockage time was less than 30 seconds. This necessitated development of another response plan that would not allow hydraulic fracture before remedial actions were implemented.

The experience of S2-A7 where lack of spoil return was not sufficient to trigger the action plan was troubling because it provided no means for detecting the onset of possible fracture until the presence of air bubbling was present in the reservoir and potential damage had already occurred.

Second Restart of Upstream Jet Grouting and Triple Fluid Column S2-A3B

Vibrating wire piezometers and open tube devices were installed to monitor the effects of jet grouting during a moratorium period of about two months to determine longer term effects of the observed air release. No adverse conditions were noted, and jet grouting for additional upstream parametric columns was resumed on 19 September 2006 with the vibrating wire piezometers and open tube devices providing real-time monitoring of jet grouting pressures. Some of the adjustments made before jet grouting resumed are use of a larger annulus, i.e., use a 254 mm (10 inch) casing and 228.6 mm (9 inch) drill bit so the grout annulus was reduced from 25 mm to 12.5 mm (1 inch to 0.5 inch), starting air and fluid circulation in

[FIG. 4] Photograph of air bubbling in Tuttle Creek Reservoir during jet grouting of upstream column S2-A3 with reservoir turbidity curtain in background

the casing instead of at the bottom of the borehole, and starting jet grouting at the column top and moving to the bottom instead of starting at the bottom and moving to the top. This allowed better evacuation of spoil by not operating under a full column of spoil, especially near the top of the column after starting. The triple fluid system utilized a drill rod diameter of 114 mm (4.5 inches) which created an annulus of 57.2 mm (2.25 inches), instead of 44.5 mm (1.75 inches) if the 12.7 mm (0.5 inch) wide annulus of hardened grout in the casing was still present. The first three jet grout columns, S2-A3B, S2-A6B, and S2-A4B, were completed on 19, 20, and 21 September 2006, respectively, without a visible air release into Tuttle Creek Reservoir.

Table 3 provides a comparison of the jet grouting parameters used at Paso de las Piedras Dam and upstream triple fluid column S2-A3B, which did not experience reservoir air bubbling. The triple-fluid system used at Tuttle Creek involved grout pressure and water and grout flow rates that exceeded those used at Paso de las Piedras Dam. However, the air pressure and flow rate used at Paso de las Piedras did exceed those at Tuttle Creek which may explain the presence of air bubbling in the reservoir at Paso de las Piedras and not during S2-A3B. The biggest difference between these two projects is the contractor proposed a minimum column diameter of 2.6 m (8.5 ft) for Tuttle Creek even though it had only constructed 1.6 to 2.4 m (5.2 to 7.9 ft) diameter primary columns for the cutoff wall at Paso de las Piedras Dam. This increase in column diameter requires a larger volume of material to be evacuated through the borehole annulus which may be problematic for achieving good spoil return. In addition, higher energies would be required for the secondary columns because higher energies were required for the secondary columns at Paso de las Piedras Dam due to densification that occurred during primary column construction (38 to 100 MJ/m instead of 32 to 71 MJ/m) (8543 to 22481 ft-kips per ft instead of 7194 to 15962 ft-kips per ft). It was encouraging that no air bubbling occurred during the first three jet grout columns even though some of the grouting parameters exceeded those at Paso de las Piedras Dam, which lead to some optimism that subsequent jet grouting could continue safely at Tuttle Creek.

Jet Grouting of Upstream Double Fluid Column S2-A7B

Unfortunately during double fluid column S2-A7B on 22 September 2006, air bubbles again occurred in Tuttle Creek Reservoir. This air release occurred in close proximity to the air release that occurred during double fluid S2-A7 and occurred within about 10 seconds

[TABLE 3] Triple fluid jet grouting parameters for upstream parametric column S2-A3B and Paso de las Piedras Dam

Jet Grout Parameters	Tuttle Creek Dam S2-A3B	Paso de las Piedras Dam Primary Columns in Silt and Sands
Air Pressure (bars)	12	12 – 20
Air Pressure (MPa)	1.2	1.2 – 2.0
Air Pressure (psi)	174.0	174.0 – 290.1
Air Flow Rate (liter/min)	8,000	12,000 –18,000
Air Flow Rate (m^3/hour)	480.0	12,000 –18,000
Water Pressure (bars)	440	450
Water Pressure (MPa)	44.0	45,000
Water Pressure (psi)	6,381.7	6,526.7
Water Flow Rate (liter/min)	412	170
Water Flow Rate (m^3/hour)	24.7	170
Grout Pressure (bars)	290	200 – 220
Grout Pressure (MPa)	29.0	20,000 – 22,000
Grout Pressure (psi)	4,206.1	2,900.8 – 3,815.8
Grout Flow Rate (liter/min)	415	180
Grout Flow Rate (m^3/hour)	24.9	180
Withdrawal Rate (m/hour)	6.8	
Specific Energy (MJ/m)	245	32 – 71
Nominal Column Diameter (ft)	Minimum 8.5	5.2 – 7.9
Nominal Column Diameter (m)	Minimum 2.6	1.6 – 2.4

of spoil return loss. Table 4 presents the jet grouting parameters for upstream parametric column S2-A7B.

Significant pressure, probably air and grout, developed in an adjacent vibrating wire piezometer (P-65-3) during drilling for column S2-A7B. Piezometer P-65-3 also exhibited high pressures during jet grouting of column S2-A7B and culminated in the appearance of air bubbling in the reservoir shortly thereafter. Piezometer P-65-3 was tipped in the shale and limestone embankment material at elevation 312.6 m (1025 ft). Data from piezometer P-65-9 in boring 65-3 which is tipped in the foundation sand just below the natural fine-grained blanket at elevation 303.5 m (995 ft)) also showed a rapid increase and decrease in piezometric head from the equilibrium value of about Elevation 313.7 m (1028.5 ft) with initiation and termination of jet grouting.

Upstream Ground Fracture Mechanisms

A number of mechanisms were considered to explain the sustained air bubbling in Tuttle Creek Reservoir. These mechanisms are summarized below and include:

[TABLE 4] Double fluid jet grouting parameters for upstream parametric column S2-A7B

Jet Grout Parameters	Tuttle Creek Dam S2-A7B
Air Pressure (bars)	6
Air Pressure (MPa)	0.6
Air Pressure (psi)	87.0
Air Flow Rate (liter/min)	4,500
Air Flow Rate (m^3/hour)	270.0
Grout Pressure (bars)	438
Grout Pressure (MPa)	43.8
Grout Pressure (psi)	6,352.7
Grout Flow Rate (liter/min)	503
Grout Flow Rate (m^3/hour)	30.2
Specific Energy (MJ/m)	179
Nominal Column Diameter (ft)	Minimum 8.5
Nominal Column Diameter (m)	Minimum 2.6

- Air permeation into permeable foundation sands, storage of large quantities of air, followed by air exiting through pre-existing/natural defects in the natural fine-grained soil blanket, and air entering the reservoir at or near the upstream toe of the dam.
- Air permeation into permeable foundation sands, storage of large quantities of air, followed by fracture of the natural fine-grained soil blanket by elevated air, water, and/or grout pressures, and air entering the reservoir at or near the upstream toe of the dam.
- Ground fracturing of the natural fine-grained soil blanket and air entering the reservoir from the jet grouting.
- The pre-drilling for the jet grout drill rod for column S2-A3 puncturing the PVC casing due to lack of verticality and causing an air release.
- Lack of seal at the bottom of the ungrouted steel casing in the shale and limestone embankment fill in column S2-A7 and S2-A7B, air traveling through pervious zones of the shale and limestone fill, and into the reservoir along the upstream face of the dam.

The continuation of bubbling for at least 48 hours after jet grouting suggests that a large amount of air was stored and it is unlikely that the natural fine-grained blanket or shale and limestone rock fill (predominantly compacted clayey shale) under the reservoir, see Fig. 1, could store that large volume of air. In addition, the close proximity of the shale and limestone suggests that the air would have appeared quicker and more gradually if the leak occurred at the bottom of the steel casing used for column S2-A7 or through a punctured PVC casing for S2-A3. Finally, sonic drilling samples obtained from near the base of the S2-A7B casing suggested the presence of granular material at or near the base of the steel casing in the shale and limestone. This also would facilitate movement of air to the reservoir which should have resulted in air appearing quicker and not continuing for over 48 hours after termination of jet grouting.

Air exiting from the permeable foundation sands through natural defects in the natural fine-grained blanket also is not plausible because the reservoir had been filled to the MPP level since 29 April 1963. Thus, an active seepage condition had been occurring for about 43 years which would have filled any pre-existing/natural defects in the natural fine-grained blanket with sediment. More importantly, pre-existing defects in the fine-grained blanket are not consistent with a 13.7 m (45 ft) hydraulic head loss that is currently occurring across the blanket based on piezometers measurements routinely made over the life of the dam. This mechanism also suggests that loss of spoil return has no consequences, when it is well known in the industry that loss of spoil return will result in a near instantaneous increase in bottom hole pressure and can easily cause hydrofracture.

Therefore, a plausible mechanism for air bubbling in the reservoir during jet grouting for columns S2-A3 and S2-A7 is the accumulation of sufficient pressure with reduced spoil return, storage of large quantities of air in the permeable and high void ratio foundation sands, and subsequent fracture of the natural fine-grained soil blanket that released the stored air for at least 48 hours after cessation of jet grouting. However, the most plausible mechanism for the air bubbling in the reservoir for double fluid column S2-A7B appears to be different and is discussed below. After blanket fracturing occurred, the stored air exited the foundation sands and entered the reservoir. The bubbling was able to continue for over 48 hours because of the large void space of the foundations sands allowed a substantial amount of air to be stored. It is likely that the reason air bubbling was not observed sooner in the reservoir is: (1) the distance air had to travel, from elevation 305.7 (1002.3 ft) or lower to the natural fine-grained blanket, (2) the time required for sufficient pressure to accumulate to fracture the natural fine-grained blanket, and (3) the time required to fracture and/or permeate the reservoir sediment and/or the overlying shale/limestone fill. This assumes that the air traveled upstream of the impervious fill, which is reasonable given the hydraulic conductivity of the impervious fill and the additional vertical stress imposed by the thicker embankment. This scenario also explains air bubbles appearing in the reservoir shortly/immediately after spoil returned slowed in column S2-A7 because the air pressure of 1,200 kPa (174.0 psi) fractured the lower sands and nearby natural fine-grained blanket and shale/limestone fill had been previously fractured by column S2-A3.

Additional evidence of ground fracturing during the upstream parametric columns was obtained via the subsequent subsurface investigation. The evidence consisted of core samples containing a grout fracture from a boring near upstream jet grout column S2-A1. The arrows in Fig. 5 point to grout lenses found in the natural fine-grained blanket at a depth of 23.8 m (78 ft). This core sample was obtained at a ground surface distance of only 1.2 m (4 ft) from the center of jet grout column S2-A3. This ground fracture is significant because it is located downstream of the column. This proved that ground fracturing can occur downstream (towards higher confining stress) as well as upstream of the jet grouting. Appearance of bubbles was not possible downstream of the jet grouting because the surface of the work platform was dry. Hence, air release may have occurred but would have gone undetected by site personnel. Fractures created downstream of the cutoff wall can provide seepage paths through the fine-grained blanket after the wall is complete. These seepage paths could lead to undesirable erosion and piping downstream of the cutoff wall. This is of critical importance because a completed cutoff wall could address the upstream fractures but cannot address downstream fractures. This scenario becomes increasingly dangerous at elevated pool levels that are certain to occur in a flood control reservoir.

Additional evidence of upstream ground fracture included hardened grout being observed in the spoil return for nearby upstream jet grout column S2-A9B and sonic coring for other upstream parametric columns revealing grout and spoil outside of the completed column and downstream of the cutoff wall alignment. Some piezometers upstream of the jet grouting also may have grout in them as was observed in dewatering well No. 2 in the downstream test area described above.

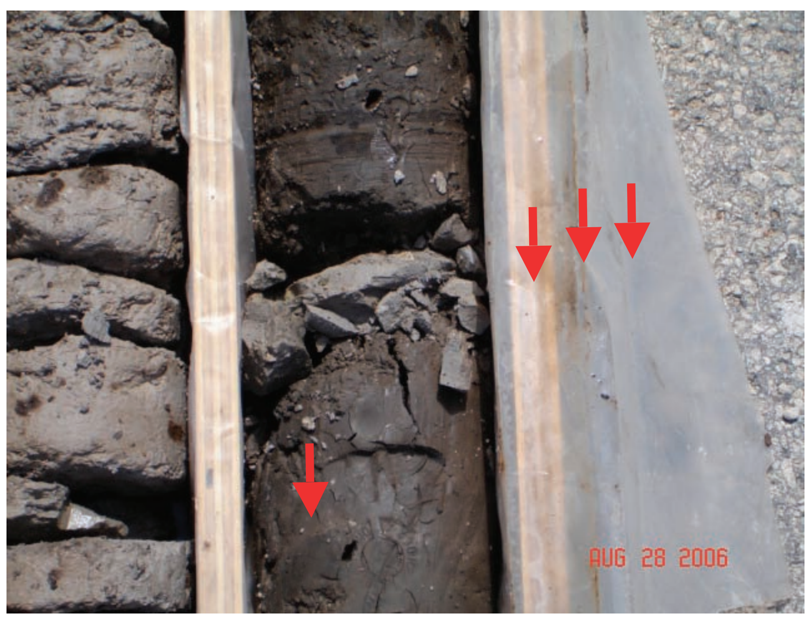
The most plausible mechanism for the air bubbling in the reservoir during jet grouting for column S2-A7B is different than the mechanism presented above for columns S2-A3 and S2-A7. This air release does not appear to be caused by accumulation of sufficient pressure, storage of large quantities of air in the foundation sands, and subsequent fracture of the natural fine-grained soil blanket that released the stored air for at least 48 hours after cessation of jet grouting. This mechanism appears to involve an air release at the bottom of the ungrouted steel casing in the shale and limestone embankment fill, through the shale and limestone fill, and into the reservoir. Alternatively, air could have connected to the reservoir through a previous fracture caused by jet grouting for columns S2-A3 and/or S2-A7. The mechanism is likely different than for columns S2-A3 and/or S2-A7 because the air release occurred shortly after jet grouting started and did not last for days after the jet grouting ceased as discussed below.

Dam Safety Implications and Cancellation of Jet Grouting at Tuttle Creek Dam

If the mechanism or cause of the reservoir air bubbling is fracturing of the fine-grained blanket, there could be detrimental consequences to Tuttle Creek Dam because new flow pathways could be created in an uncontrolled manner through this important seepage control feature. This can lead to increased seepage and hydraulic gradients in the foundation soils which can cause erosion and possibly piping of the foundation soils that would undermine the embankment. More importantly, the discovery of grout fractures downstream of the cutoff alignment indicates that fractures can occur downstream of the jet grouting. This could render the impervious zone embankment material and the natural fine-grained blanket less effective in resisting or reducing seepage over the top of the cutoff wall.

Reducing the effectiveness of the impervious zone would result in seepage directly through the blanket downstream of the cutoff wall and higher gradients acting across the blanket than with the upstream impervious zone being present. This is significant because of the variability of the natural fine-grained blanket across the valley. This variability was mitigated during design by the extended impervious zone so damage to the impervious zone could result in greater seepage and hydraulic gradients in the foundation sands. In addition, the natural fine-grained blanket could be damaged directly by jet grouting because grout was found downstream of the upstream parametric columns in the fine-grained blanket (see Fig. 5). The potential for ground fracture is about the same upstream and downstream of the cutoff wall because the work platform was essentially flat so the difference in vertical effective stress between upstream and downstream is small. The biggest difference between the upstream and downstream sides of the cutoff wall is the downstream is slightly more overconsolidated because of the previous greater height of the embankment in the work platform area, thereby increasing the horizontal effective stress.

Potential fractures or seepage paths could develop along the downstream side of the wall and through fractures in the natural fine-grained blanket and/or the shale and limestone fill downstream of the wall. These scenarios would result in higher gradients occurring in the foundation sands which also could result in unacceptable hydraulic gradients and flow at the downstream relief well system. The potential for seepage along the downstream side of the wall appeared extremely likely because previous jet grout experience suggests that the completed columns will serve as pressure relief points by allowing pressure to escape along the sides of the completed column. If completed columns were to serve as pressure relief points, a flow path could be created along the downstream side of a completed column which would allow the reservoir head to enter the foundation sands along the downstream side of the completed columns. Piezometer measurements made prior to jet grouting show a head loss in the shale and limestone fill so fracturing of this material would allow a higher gradient



[FIG. 5] Grout fracture in the core for column S2-A1 at a depth of 23.8 m (78 feet) (arrows indicate grout found in the embankment materials)

to act at the top of the fine-grained blanket than was present prior to jet grouting.

Because of these dam safety risks and the inability to control these risks, the USACE decided to abandon all upstream jet grouting which meant terminating the upstream cutoff wall and slope stabilization. This reduced the remedial work to only downstream slope stabilization using a non-jet assisted technique. The option of smaller diameter columns and additional rows (allowed in the contract) were viable options, but not options the contractor adopted due to cost concerns. The smaller diameter columns would have reduced the fracture potential by reducing the pressures required but these pressures still were probably high enough to cause ground fracturing. Downstream stabilization involved construction of transverse shear walls using a self-hardening cement-bentonite slurry. The transverse shear walls were excavated using a clamshell device mounted on a crane (see Walberg et al., 2012).

DAM SAFETY RISKS IN JET GROUTING PROCESS

This section discusses some of the risks of jet grouting in or below an operational dam, such as lack of spoil return and the buildup of downhole pressures. At present, there appears to be no viable means in the industry for measuring downhole pressures, detecting the onset of ground fracture, or detecting grout outside of the completed columns to determine the extent of grout migration. This section briefly discusses each of these risks and deficiencies in current industry practice.

- This case history shows that even if spoil return is maintained, high and detrimental ground pressure can still develop and cause ground fracture.
- At present, the authors are not aware of a usable device, in-rod instrumentation, or technique for accurately measuring downhole pressure which allows selection and/or adjustment of appropriate jet grout parameters so ground fracture does not occur, or at least would be recognized when it was occurring.
- The ability to locate grout migration is difficult using standard drilling techniques because of the small size of the sample and the small amount of grout being sought. In addition, the more problematic fractures are ones that are not filled with grout and would go undetected if the investigative means consist only of borehole drilling.

RECOMMENDATIONS FOR FUTURE JET GROUTING AT OPERATIONAL DAMS

This section discusses some recommendations for constructing a jet grout cutoff wall in dams with an operational reservoir.

Control Spoil Blockage

First, it is important to control/manage spoil blockages and poor spoil return to reduce the potential for ground fracture. Brill et al. (2003) present some methods for controlling spoil return but the keys to continuous spoil return appear to be an adequate annulus to evacuate the required material, effectively disaggregating the native soils, and low viscosity spoils. It may be possible to reduce spoil viscosity by adjusting the jet grouting parameters/ methodology or by a precutting technique. A precutting process was used for the Posey Tube project near Oakland, California (Lee et al. 2005) because of difficulties in obtaining good, uniform columns in the stiff native clay layers. Through many test columns the contractor found that precutting was required to detach and disaggregate the clayey materials to facilitate spoil return and allow thorough mixing with grout. Lee et al. (2005) state that to enhance the consistency and structural integrity of the production columns, the contractor “remixed” the new columns by re-inserting the drill rod to the bottom of the columns and re-grouting the columns upward. The first phase would involve precutting with water or a dilute grout followed by a second phase of grout injection from the bottom of the column upward.

Other jet grouting recommendations for improving spoil return and reducing the potential for spoil blockage and ground fracture include:

- Increase hole diameter/annulus as large as possible.
- Install fragmenting PVC drill casing through lower clay zones as suggested by Sembenelli and Sembenelli (1999) to increase the potential for the hole to remain open and remove constrictions.
- Staring erosion at the top of the column and continuing to the bottom so top-down ero-

sion as well as bottom-up erosion and jet grouting occurs.

- Adjust jet grouting parameters and methods to decrease spoil density and viscosity
- Increase station time to increase erosion and disaggregation of in situ materials
- Initiate air flow and pressure with the monitor still in the casing so elevated air pressure is not required to overcome the column head in the borehole and the tendency for cuttings to settle into the boring while the drill rods are idle, e.g., during verticality measurements.
- Install pressure relief wells, i.e., a passive response system, near jet grouting to relieve some pressure to reduce the potential for wide spread ground fracture assuming a high insitu hydraulic conductivity. However, relief wells most likely will not be able to prevent localized ground fracturing.
- Develop a response plan for blockages and instrumentation prior to jet grouting to prevent or limit the amount of ground fracture. The response plan should recognize that poor spoil return and temporary blockages can lead to elevated bottom hole pressures occurring before the response plan is implemented. As a result, the response plan must have early triggers to change grouting methodology so large pressures and ground fracturing do not occur. This response plan also should include criteria, e.g., an unacceptable increase in pressure in adjacent piezometers, for requiring the contractor to "stroke the hole" with the jet grouting drill rod to re-establish suitable spoil return and reduce bottom hole pressures. If high pressures or poor spoil return continue, the plan should require a change in jet grout methodology to address these conditions.

Measure and control downhole pressures

It is also recommended that a device or technology be developed for measuring downhole pressures so the onset of ground fracturing can be monitored which is important for a dam with an operational reservoir. This device is desired because contract language for the Tuttle Creek project that stated "ground fracture shall not occur" was not sufficient. This is because ground fracture likely occurred even though precautionary measures were undertaken. It is anticipated that some in-rod instrumentation could be developed to measure the pressure outside of the monitor during jet grouting. The instrumentation could send the pressure measurements to the surface via cable or possibly by wireless technology. If measuring downhole pressure is not possible, it is recommended that a correlation between downhole pressure and acceptable spoil return/properties be developed so downhole pressures can be indirectly estimated from observable spoil return. Kauschinger (2006) describes such a device that monitors borehole pressure during jet grouting at one or more locations in the borehole. The device would notify the jet grouting operator when there is a risk of soil fracture because the borehole pressure exceeds a predetermined limit or predetermined rate of increase, e.g., 69 kPa (10 psi) in 10 seconds (Kauschinger, 2006). It is important that the pressure required for ground fracture should be agreed upon before the onset of jet grouting so this point is not a source of confusion/contention if indications of ground fracture develop.

Jet grouting impact on seepage control elements

Given the difficulties measuring downhole pressures, maintaining adequate spoil return, and identifying the onset, location, and extent of ground fracture, some technology should be developed for determining the integrity of existing seepage control elements, e.g., the natural fine-grained soil blanket, upstream impervious extended zone material, and the clay core, before, during, and after jet grouting. This technology should monitor fracturing/damage, if any, to the seepage control elements during grouting and estimate the continued effectiveness of the elements as jet grouting production proceeds. One possible technology for accomplishing this objective is mapping and monitoring of subsurface water flow which may provide an insight into flow patterns and changes in flow patterns after the initiation of jet grouting. This is accomplished by energizing a water-bearing zone, e.g., the natural fine-grained blanket and/or impervious zone material, with an AC current. As the AC current flows through the water a magnetic field is generated and is measured at multiple points on the ground surface using patented technology. These data are used to prepare contour maps of subsurface flow that can be used to determine if changes in flow, i.e., ground fractures, have occurred as a result on the jet grouting.

Finally, a double fluid system may be more desirable for jet grouting with an operational reservoir than a triple fluid system because if ground fracture does occur, it may be sealed with either a cement based spoil or a grout fluid. The triple system uses a pair of opposing water jets shrouded with air for ground erosion and another jet for injecting the necessary cement grout or other binder that provides the long term durability and low hydraulic conductivity in the soil. In a double system the erosion is caused by two opposing jets of grout also shrouded with air. In principle the triple system may offer less risk of ground fracture but fractures may partially or fully close before the grout jet can inject cement or binder to seal the fracture. Conversely, the double fluid system causes erosion by the grout jet so if ground fracture does occur a cement/binder would have a better opportunity to fill the fracture(s) and seal it than with the triple fluid system. Control and sealing of fractures is an important consideration given an operational reservoir, the cutoff wall usually being located on the upstream side of the dam, and the impervious core being in close proximity to the embankment material.

CONCLUSIONS AND RECOMMENDATIONS

Jet grouting has seen increased usage in ground improvement efforts to address strength and compressibility of soils. The practice has become popular because of its relative ease of installation (compared to excavations) and its versatility. Recently its use has extended to seepage control and/or strengthening of dams with an operational reservoir. Results of recent jet grout projects involving dams suggest that the process may create some dam safety risks including ground fracture, which can lead to undesired seepage pathways and hydraulic gradients in erodible materials, inadequate securing of the cutoff wall or seepage control measure to the impervious embankment material, and variability in column diameter and consistency. Of course the largest risk is the potential for ground fracture due to the high air, water, and grout pressures required to create large diameter soilcrete columns.

This paper presents recommendations for reducing dam safety risks including control and maintenance of spoil return, measuring/monitoring of downhole pressures, and estimating the effect(s) of jet grouting on the existing seepage control measures in the dam. Given the large consequence of failure, the impact of jet grouting on the embankment and foundation materials should be understood when an operational reservoir is present.

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