Case Study

Seismic Retrofit of Tuttle Creek Dam

Francke C. Walberg, M.ASCE[1]; Timothy D. Stark, F.ASCE[2]; Peter J. Nicholson[3]; Gonzalo Castro, M.ASCE[4]; Peter M. Byrne[5]; Paul J. Axtell, M.ASCE[6]; John C. Dillon[7]; William B. Empson[8]; Joseph E. Topi[9]; David L. Mathews[10]; and Glen M. Bellew[11]

Abstract: This paper discusses the seismic retrofit of Tuttle Creek Dam near Manhattan, Kansas, including investigations, seismic analyses, design, construction, and stabilization techniques used. Original plans called for stabilization of the upstream and downstream slopes and installation of an upstream cutoff wall to reduce underseepage. However, constructability and dam safety issues, along with the results of refined seismic deformation analyses, led to cancellation of the jet grouted upstream slope stabilization and cutoff wall. Downstream slope stabilization was to be accomplished by jet grouting or soil mixing, but ultimately was accomplished using a self-hardening cement-bentonite (C-B) slurry to construct transverse shear walls. A total of 351 transverse shear walls were constructed along the downstream toe by primarily clamshell equipment. Typical shear walls are 13.7 m long, 1.2 m wide, and extend 18.9 m deep or about 6.1 m into the coarse foundation sands. The walls are spaced at 4.3 m on center along the downstream toe for a replacement ratio of about 29%. In addition to the transverse shear walls, the relief well collection ditch along the downstream toe was replaced with a buried collector system to further improve downstream stability and underseepage control. **DOI: 10.1061/(ASCE)GT.1943-5606.0000818.** *© 2013 American Society of Civil Engineers.*

CE Database subject headings: Jet grouting; Soil mixing; Core walls; Shear walls; Soil mechanics; Soil permeability; Soil liquefaction; Earthquakes; Displacement; Slope stability; Kansas; Dams; Case studies; Seismic effects.

Author keywords: Jet grouting; soil mixing; cutoff walls; shear walls; soil mechanics; permeability; liquefaction; earthquake induced displacement; slope stability.

Introduction

Tuttle Creek Dam is a U.S. Army Corps of Engineers (USACE) project, located on the Big Blue River in the Kansas River Basin (see Fig. 1). It is part of a system that provides a comprehensive plan for flood control, water supply, and supplemental release for navigation of the Missouri River. Construction started in 1952 and was completed in 1961. Reservoir filling began in 1962 and the multipurpose pool (MPP) elevation was first reached in 1963. Tuttle Creek Dam is a zoned earthfill embankment (Lane and Fehrman 1960) and is located about 10 km (6 mi) north of Manhattan in east central Kansas. The embankment is 2,300 m (7,500 ft) long and about 43 m (137 ft) high, with a crest width of 15.2 m (50 ft). There is a gated, concrete chute emergency spillway on the left abutment. The 290 m (952 ft) wide spillway structure consists of a concrete gravity section with 6.4 m (21 ft) by 12 m (40 ft) tainter gates, a 183 m (600 ft) long concrete apron and an unlined discharge chute to the valley floor. The regulating outlet works structure consists of a tower with gated twin horseshoe 6 m (20 ft) diameter conduits and a stilling basin near the right abutment. The top of the dam is at elevation 353.3 m (1,159 ft m.s.l.) and the original ground surface is about 312.4 m (1,025 ft m.s.l.) across the valley. The MPP level is elevation 327.9 m (1,075 ft m.s.l.). The full pool, or top of the flood control pool, is elevation 346.5 m (1,136 ft m.s.l.). The reservoir capacity is over 2.5 million megaliters (2 million acre-ft) at flood control pool.

A typical cross section of the dam is shown in Fig. 2. Most of the construction materials for the embankment shells (shale and limestone fills upstream and downstream, berm fill downstream) came from the required excavation for the outlet works and the spillway. The majority of the downstream shell consists of hydraulically placed sand or sand and gravel obtained from a downstream borrow pit. The lower portion was constructed of cleaner and coarser sands and gravels to serve as a horizontal drainage blanket under the embankment. Most of the sand fill was hydraulically deposited but

[1]Senior Consultant, URS Corporation, 8300 College Blvd., Overland Park, KS 66210. E-mail: Francke_Walberg@URSCorp.com

[2]Professor, Univ. of Illinois at Urbana-Champaign, 205 N. Mathews Ave., IL 61801, (corresponding author). E-mail: tstark@illinois.edu

[3]President, Nicholson Consulting, LLC, 22029 State Road 7, Boca Raton, FL 33428.

[4]Senior Principal, Geotechnical Engineers, Inc. (GEI) Consultants, Inc., GEI Consultants, Inc., 400 Unicorn Park Drive, Woburn, MA 01801.

[5]Professor, University of British Columbia, 6250 Applied Science Lane, Vancouver, British Columbia, Canada.

[6]Principal Engineer, Dan Brown and Associates, 10134 Glenwood, Overland Park, KS 66212.

[7]Chief, Cost Estimating Section, U.S. Army Corps of Engineers, Kansas City District, 601 E 12th St., Kansas City, MO 64106.

[8]Senior Levee Safety Program Risk Manager, Risk Management Center-West-Denver, Institute for Water Resources, U.S. Army Corps of Engineers, 12596 W. Bayaud Ave., Ste. 400, Lakewood, CO 80288.

[9]Senior Geologist, Geotechnical Design and Dam Safety Section, U.S. Army Corps of Engineers, Kansas City District, 301 E. 12th St., Kansas City, MO 64106.

[10]Chief, Engineering Division, U.S. Army Corps of Engineers, Kansas City District, 601 E 12th St., Kansas City, MO 64106.

[11]Chief, Geotechnical Design and Dam Safety Section, U.S. Army Corps of Engineers, Kansas City District, 601 E 12th St., Kansas City, MO 64106.

Note. This manuscript was submitted on June 6, 2011; approved on August 8, 2012; published online on August 20, 2012. Discussion period open until November 1, 2013; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 139, No. 6, June 1, 2013. ©ASCE, ISSN 1090-0241/2013/6-975–986/$25.00.

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

Fig. 1. Aerial view of Tuttle Creek Dam (photo used with permission of U.S. Army Corps of Engineers, Kansas City District)

compacted by large dozers tracking and spreading the material while it was still essentially in a saturated condition. The hydraulically deposited sand is dense and not liquefiable based on the relatively high SPT blow counts obtained in subsequent investigations.

The central impervious core is composed of select (i.e., higher shear strength and lower hydraulic conductivity) silts and clays that were obtained from borrow areas in the natural floodplain blanket located upstream and downstream. In most areas the central impervious core zone was extended about 70 m (230 ft) upstream to lengthen the underseepage path (see Fig. 2). Soils in the native alluvial foundation of the dam consist of 2.4 to 8.2 m (8 to 27 ft) of lean clay to silts (fine-grained soil blanket) underlain by deposits of sand and gravelly sand of variable thickness and density up to a depth of 12.2 to 24.4 m (40 to 80 ft). Bedrock consists of alternate layers of shale and limestone (Permian age), with the shale beds varying between 0.6 and 11 m (2 and 36 ft) in thickness. A deep buried river channel is present in the bedrock surface near Station 48+00 (see Fig. 3). A geologic profile of the river valley is available in Lane and Fehrman (1960) as well as construction details. Because depth to bedrock is 12.2 m (40 ft) or more, the dam does not have a positive cutoff to bedrock across the valley alluvium. Thus, the upstream natural fine-grained soil blanket, extended impervious core zone,

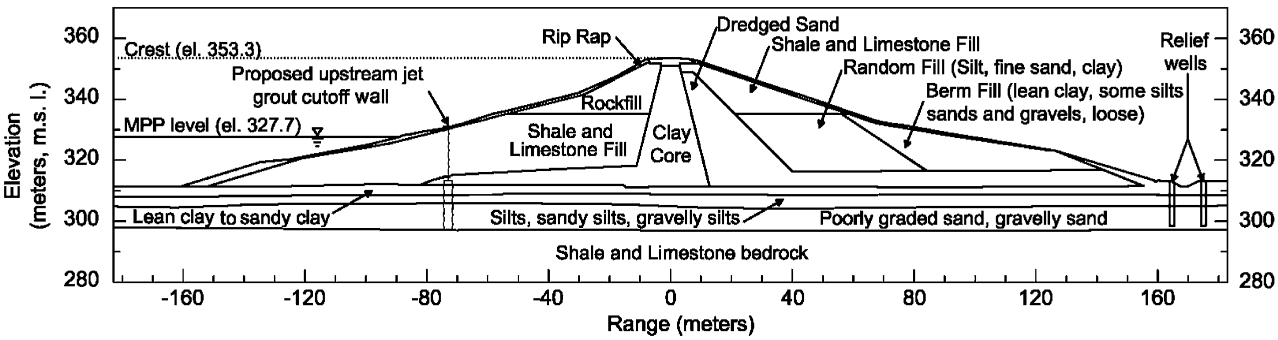


Fig. 2. Typical dam cross section around Station 50+00

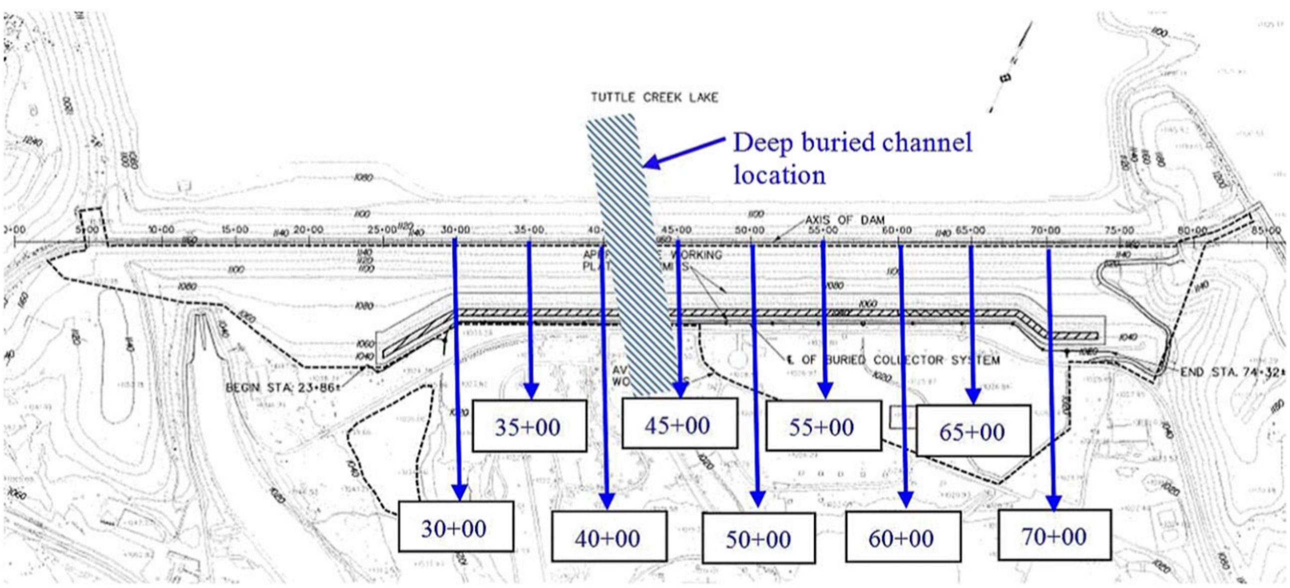


Fig. 3. Plan view showing location of Stations 30+00 to 70+00

and relief wells with a collection trench along the downstream toe control significant foundation seepage across the valley. This configuration of seepage control is typical within the USACE and particularly the USACE, Kansas City District (KCD), e.g., Milford Dam (see Stark et al. 2011). Near each abutment, stabilizing berms were constructed upstream and downstream because of soft soils encountered in a terrace deposit on the right side and in an old oxbow deposit near the left side of the valley.

Dam Safety Assurance Program

The project was executed under the USACE Dam Safety Assurance Program (DSAP), (USACE 1999), which provides the USACE with the means to address both the seismic and hydrologic inadequacies of its national inventory of dams. Only the seismic aspects of the DSAP for Tuttle Creek Dam are discussed in this paper, although hydraulic aspects were addressed during this construction as well.

For seismic adequacy, USACE criteria require high hazard dams, such as Tuttle Creek Dam, to withstand a maximum credible earthquake (MCE) without catastrophic failure, although economic damage may be tolerated. A dam is classified as high hazard when failure or mis-operation of the dam results in loss of life. DSAP regulation requires USACE Districts to perform a three-phase investigation to support a finding of whether the dam is considered safe. If the project proceeds past the initial data-gathering phase (Phase I) and the detailed seismic analysis (Phase II) and is found to be unsafe, an evaluation report is prepared to justify the necessary remediation. The evaluation report and an Environmental Impact Statement (EIS) were jointly prepared for Tuttle Creek Dam to serve as a decision document to secure funding for the remedial design and construction (Phase III). During Phase III, the remediation design was finalized and plans and specifications were prepared for bidding purposes.

Phase II Investigations

Tuttle Creek Dam is located in an area of moderate seismicity associated with an old continental rift zone, the Mid-Continent Geophysical Anomaly, MGA. The main seismic source zones are the Nemaha Ridge uplift zone and the Humboldt Fault zone, both located just to the east of the dam. The MCE, i.e., design earthquake, is a magnitude 6.6 event at a distance of 20 km. The peak ground acceleration (PGA) has a return period of about 3000 years. The MCE has a peak horizontal ground acceleration (PHGA) of 0.28 g mean and 0.56 g mean plus one standard deviation (Somerville et al. 2003).

DSAP Phase II investigations include detailed subsurface explorations, laboratory studies, and liquefaction assessments of the alluvial foundation soils. KCD conducted subsurface explorations including SPT, CPT, vane shear, shear wave velocity testing, and high quality undisturbed sampling of the alluvial fine-grained soil blanket. Cyclic triaxial testing was conducted on the fine-grained blanket silts and clays by Castro et al. (2003). Fig. 4 presents the typical subsurface conditions near the downstream toe of the dam. The groundwater surface at the downstream toe is typically located at a depth of 2.7 m (9 ft) or elevation 310.2 m (1017 ft) but it is dependent on reservoir elevation and tailwater conditions.

The initial liquefaction assessment consists of analyses at critical locations of the dam, determination of pre-earthquake static stresses, and a simplified liquefaction assessment (Seed and Idriss 1971) using one-dimensional SHAKE site response analyses. The Phase II post earthquake stability was assessed using limit equilibrium slope stability methods and assigning residual/steady state/liquefied strengths (Seed and Harder 1990) to the foundation zones that were predicted to liquefy. Because the post-earthquake limit

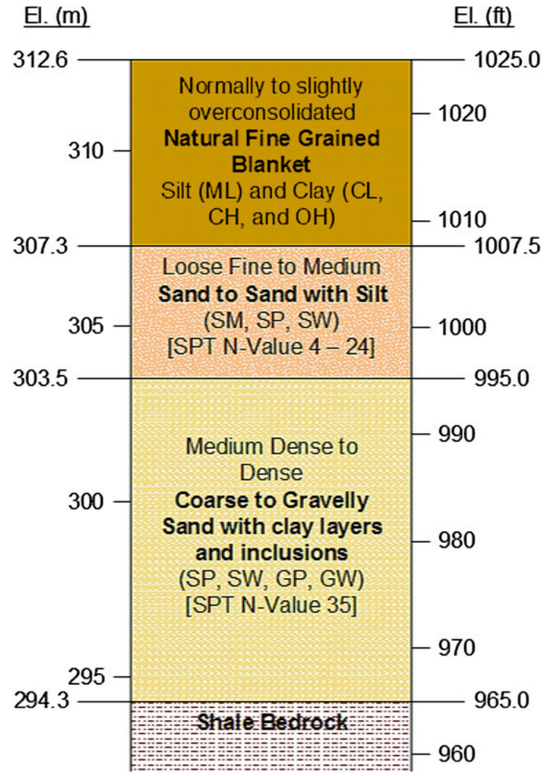


Fig. 4. Generalized subsurface profile of alluvial foundation at downstream toe [average ground surface elevation downstream of dam, 312.6 m (1025 ft)]

equilibrium stability analyses indicated a likelihood of a flow slide, post-earthquake deformations were estimated using finite element analyses using the software *Dynaflow 1* (Popescu 1998; Prevost 2002) and *TARA3-FL* (*TARA3-FL*; Finn 2004). Soil shear strength parameters and unit weights used in these analyses for both the embankment and foundation materials are shown in Table 1.

Site investigations and geotechnical analyses identified the upper alluvial foundation sands as being susceptible to liquefaction under the MCE. The clays and silts of the natural fine-grained blanket also appeared susceptible to pore-water pressure generation and strength loss during earthquake loading (Castro et al. 2003). Both limit equilibrium and finite element post-earthquake deformation analyses showed significant deformation was expected, although it was recognized that the feasibility investigation phase numerical modeling did not accurately replicate the behavior of the fine-grained blanket under seismic loading. However, overtopping of the embankment following the predicted earthquake deformations was unlikely because the dam has about 25.6 m (84 ft) of freeboard at MPP. However, pressure relief wells at the downstream toe are critical to protect the dam against foundation internal erosion and the predicted permanent deformations would disable these relief wells. With damage to or loss of the relief wells, the likely failure mechanism is internal erosion and piping of the foundation soils.

Phase II Evaluation Report and EIS

The allowable post-earthquake deformation criteria established for the Tuttle Creek Dam are: 1.5 m (5 ft) vertically at the crest, 3 m (10 ft) laterally at the upstream toe, and 0.3 m (1 ft) laterally at the downstream toe to prevent damage to the relief wells. Finite element deformation analyses performed in 2002 using *Dynaflow* (Popescu 1998) and *TARA3-FL* (Finn 2004) predicted unacceptably large

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

Table 1. Embankment and Foundation Properties: Strengths, Unit Weights

Material	Unit weight: Dry (kN/m^3)	Unit weight: Moist (kN/m^3)	Unit weight: Saturated (kN/m^3)	Drained strength: c' (kPa)	Drained strength: φ' (degrees)	Undrained strength: c (kPa)	Undrained strength: φ (degrees)
Impervious clay core	15.7	18.9	19.6	0	30.0	Peak: 38.3	11.3
						Residual: 14.4	5.7
Shale and limestone fill	17.3	20.4	21.2	0	28.0	9.6	19.8
Berm	14.5	17.3	—	0	28.0	—	—
Random fill	17.0	19.6	20.4	0	38.0	0	38.0
Dredged sand	17.0	19.6	20.4	0	38.0	0	38.0
Rockfill	18.9	18.9	—	0	40.0	—	—
Foundation clay blanket							
Free field and toes	87	105	117	0	30.0	$\mathrm{Su}/\sigma'_{\mathrm{vo\ Peak}}$: 0.35	
Under mid-slope	92	—	—			$\mathrm{Su}/\sigma'_{\mathrm{vo\ Residual}}$: 0.12	
Under crest	97	—	—				
Foundation sands							
Upper	94	—	125	0	33.0	—	
Lower	105	—	130	0	33.0		
Foundation rock	—	—	150	—	—		

deformations both upstream and downstream largely as a result of liquefaction and strength loss in the fine-grained blanket. As a result, KCD concluded that rehabilitation of the dam was required to prevent an uncontrolled release from the reservoir during or after the design earthquake. Release of the reservoir would result in devastating flood effects to portions of the City of Manhattan and beyond. To prevent an uncontrolled release of the reservoir, stabilizing the foundation under both upstream and downstream slopes with the reservoir in operation was the recommended alternative. Additionally, the stabilization was to include an upstream seepage cutoff wall to reduce the importance of the seismically vulnerable downstream pressure relief wells. To ensure public safety prior to implementation of the selected alternative, KCD installed an interim Dam Failure Warning System (DFWS) (Empson and Hummert 2004). The DFWS was an interim measure because of community requests and its potential to reduce loss of life prior to completion of the stabilization.

The Environmental Impact Statement (EIS) recommended stabilization of the foundation soil without reservoir drawdown. The stabilization methods identified in the evaluation report and EIS are as follows for the upstream slope and the downstream slope.

Upstream Slope

For the upstream slope, the stabilization method for liquefiable foundation silty clays and sands and construction of a cutoff wall is accomplished by jet grouting through predrilled holes through the embankment. The cutoff wall [depth of approximately 36.6 m (120 ft) on average] would contribute to stabilization of the upstream slope but its primary function was to reduce seepage and piezometric levels to acceptable levels at the downstream toe so the relief wells would not be critical.

Downstream Slope

For the downstream slope, the stabilization of liquefiable foundation silty clays and sands is accomplished using jet grouting or soil mixing along the downstream toe.

The EIS allowed some adjustments to the exact nature of the soil stabilization equipment, techniques, and admixtures. However, a significantly different technology not specifically included in the EIS, e.g., stone columns, could not be used without resubmitting and reapproving the EIS. This limited the technologies that could be considered in the design phase and still have the project completed in the necessary funding timeframe. However, if a significantly different technology was shown to have substantial advantages over already approved methods, then it would have been judicious to resubmit a revised EIS for approval. The evaluation report recommended that final design and development of plans and specifications would include additional subsurface exploratory work, soil testing, test drilling through the embankment (completed Fall 2003), and a test program to demonstrate jet grouting and soil mixing technologies at the site. The test program also allowed the development of design parameters for use in numerical analysis of the final design.

Phase III Investigations

Foundation Modification Construction Contract

In 2005, KCD awarded a construction-manager-at-risk contract that included a base contract. The objective of this contracting mechanism was to obtain the expertise and innovations of a specialty foundation contractor during stabilization design. A base contract in the amount of $49 million was awarded for the construction of an upstream cutoff wall and preconstruction services. Contract options available for later construction award included: (1) a test program for site-specific evaluation of jet grouting and soil mixing, (2) main upstream foundation stabilization, and (3) downstream foundation stabilization.

Final Design

Shortly after awarding the construction-manager-at-risk contract, KCD selected an advisory panel (AP) to assist with analysis, design, review of submittals, construction specifications, and conducting site visits. Task orders consisting of overall project review, final cutoff wall design, downstream test program option planning, and interpretation of test program results were initially issued. Subsequent task orders include conducting a seismic deformation analysis and remediation design using the FLAC software package (*FLAC 5.0*) and assembling a geotechnical baseline report for the main construction option. The construction contractor also elected to form its own panel of experts early in the project.

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

Jet Grout Cutoff Wall

Design

The jet grouted upstream cutoff wall was to be 0.3 m (10 ft) wide and constructed 73 m (240 ft) upstream of the dam centerline. At this location, it would create a barrier from near the upstream tip of the extended impervious core zone to foundation bedrock. The required depth and height of the wall would also be less than if it were constructed at the centerline. Additionally, this location would avoid rerouting traffic from the highway located at the dam crest. Predrilling through the shale and limestone embankment fill would allow construction to be accomplished with overlapping jet grout columns.

Specifications for the cutoff wall required a minimum 1,170 kPa (170 psi) strength and a differential hydraulic head across the wall of 11.9 m (39 ft) at MPP. The objective was to reduce downstream piezometric levels so the seismically sensitive pressure relief system along the downstream toe would no longer be necessary. The cutoff wall was to be constructed using multiple (at least two) rows of jet grout columns. The wall was to penetrate a minimum of 0.3 m (1 ft) into the bedrock except between stations 47+75 and 48+50 where a 3 m (10 ft) bedrock socket was required because of the presence of the deep buried channel and to ensure bedrock was encountered, as opposed to boulders suspended in the soil matrix. During Phase III exploratory drilling, the buried channel was found to extend deeper than originally thought, from elevation 283.5 m (930 ft) to about elevation 278 m (913 ft). The channel has an irregular bedrock surface and large slump blocks were found to be present above bedrock.

At the proposed location, the cutoff wall would have to tie not only into bedrock, but also the extended portion of the impervious core. Constructing a cutoff wall with overlapping jet grout columns would also require precise borehole alignment to ensure overlap of adjacent columns to form a continuous barrier. Additionally, the column diameter would have to be constant through the variable soft to stiff clays and silts to loose to dense sands and gravels of the foundation. Thus, site-specific correlations between jet grout energy and column diameter had to be developed and the energy varied to account for the considerable variation in foundation strata. An additional challenge of the jet grouted cutoff wall was that, in accord with the EIS, the reservoir had to be maintained, which meant that as the cutoff wall progressed, piezometric levels upstream of the wall would increase toward the reservoir head and thus high gradients would be encountered as wall closure approached. Finally, contract specifications were prepared, which emphasized the criticality of performing the jet grouting so the embankment and natural fine-grained blanket were not damaged by ground fracturing. Prevention of ground fracturing was a major concern for the Tuttle Creek Dam because of the important and fragile underseepage control system.

The contractor proposed utilizing high capacity pumps to produce column diameters of 2.6 m (8.5 ft) and two rows of columns to create the 3 m (10 ft) thick wall. The proposed column diameters were larger than any that the contractor had constructed previously. Field trials to establish jet grout parameters, column diameter measurements and uniformity, energy correlations, and assessment of quality of treated soil, were undertaken as described in the following section.

Field Trials: Downstream Parametric Columns

In the spring of 2006, field trials for the cutoff wall and slope stabilization began about 152 m (500 ft) downstream of the dam under the base contract and the Test Program Option (TPO) awarded to the contractor by the KCD. The main objective of the TPO was to investigate the viability of jet grouting and soil mix technologies before beginning production of the upstream and downstream foundation ground improvement. At this location, trials with various parameters could be conducted without risk to dam safety (note that the in situ stress conditions were significantly different from the expected values below the upstream slope of the dam). The contractor's goal of the parametric columns was to achieve a column diameter of 2.6 m (8.5 ft) and a minimum unconfined compressive strength of 1,170 kPa (170 psi). The contractor used a specific energy approach with the objective of correlating specific energy to column diameter. For example, the specific energy, E, in MJ/m, for triple-fluid jet grouting the specific energy is calculated by adding the energy imparted by the water and air eroding jets [see Eq. (1)]:

$$E_{JG\text{-}Triple} = \frac{(P_W * Q_W + P_A * Q_A)}{V_S} \tag{1}$$

where P_W and P_A are the water and air pressures in MPa, respectively, Q_W and Q_A are the water and airflow rates in m^3/hour, respectively, and V_s is the jet withdrawal speed in m/hour. Use of these units will yield values of $E_{JG\text{-}Triple}$ in MJ/m.

Under the base contract a total of 18 parametric jet grout columns were constructed primarily using triple fluid (air, water, and grout) jet grouting, although nine were constructed using double fluid technique (only air and grout). Column properties and diameters were checked by making cores of completed columns with sonic drilling and conventional core drilling with a double tube core barrel. The contractor experienced considerable difficulty in constructing the downstream parametric columns including maintaining spoil return, instability of the excavated column roof, and settlement of the soilcrete after column completion. The contractor reported that many of the triple fluid columns achieved a 2.6 m (8.5 ft) diameter; however, several columns had diameters up to 3.7 m (12 ft) while the double fluid column diameters were from 2.1 to 2.4 m (7 to 8 ft). These reported diameters were all in the portion of the column above 12 m (40 ft) depth. Below 12 m (40 ft) in the denser sands and gravels, and in the deeper stiff clay layer, column diameters were significantly smaller than 2.6 m (8.5 ft). These results suggest that upstream jet grouting, where stresses and material strength and densities are increased, would require greater energy by increasing water, air, and/or grout pressures or by decreasing jet withdrawal speed or station time of the jet grout monitor at each elevation step.

Under the TPO, 27 soil mix columns were also constructed in the downstream test area adjacent to 27 jet grout columns. The downstream test area was approximately 56.4 m (185 ft) wide and 103.7 m (340 ft) long (see Stark et al. 2009 for details). The top of the columns were about elevation 310 m (1016 ft), and unlike the case of the upstream field trial, parametric columns for the cutoff wall (described in the next section) did not extend to bedrock, but rather were carried only to elevation 299 m (980 ft). After the jet grout columns were complete and subsequent coring of the columns had been conducted, the groundwater within the cement-bentonite (C-B) slurry wall surrounding the test area was lowered to 11.3 m (37 ft) below ground surface (b.g.s.), i.e., elevation 301.4 m (989 ft), to allow excavation of the soil surrounding the columns. Stark et al. (2009) describe the excavation and sectioning of some of the jet grout columns to determine column integrity and homogeneity. The sectioned jet grout test columns contained more than 40 to 50% native soil that was not broken up and evacuated during the jet grout process. Most of the inclusions were greater than 75 to 100 mm (3 to 4 in.) in size and some were much larger. The observed inclusions in the completed jet grout columns included significant

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

amounts and large pieces of both fine-grained (silts and clays) and coarse-grained (fine sands and sands) soils.

In summary, the significant amount and large size of the soil inclusions found in the completed jet grout columns suggested that blockage of the annulus between the boring casing and jet grouting drill rod should be expected when constructing large-diameter columns. Blockage or insufficient spoil return will also lead to accumulation of high pressures in the subsurface and possible ground fracturing. Because high water, grout, and air pressures are used to construct these columns, some air bubbles were observed at the ground surface during downstream jet grouting, although not to the extent observed in the reservoir during subsequent upstream parametric columns.

In addition to sectioned jet grout columns, three jet-assisted soil-mixed columns were also sectioned. The sectioned jet-assisted soil-mixed columns did not have significant or measurable inclusions, and were thoroughly mixed. This indicated a high degree of mixing was achieved during column construction (Stark et al. 2009). However, the soil mix technology available to the contractor was also jet-assisted, with a single axis in which grout was introduced into the soil mix column under high pressure. As a result, this soil mixing technology posed risks similar to jet grouting and was subsequently viewed as a dam safety risk.

Field Trials: Upstream Parametric Columns

The results of the downstream parametric columns were not completely representative of the results that would be obtained on the upstream slope for a variety of reasons, including the fact that the in situ earth and water pressures are greater upstream than downstream. As a result, in early July 2006, the contractor was permitted to move to the work platform on the upstream slope to construct additional trial columns to develop correlations of column diameter, energy, and material type. These parametric columns were constructed from the cutoff wall work platform constructed at elevation 333.8 m (1095 ft), immediately downstream from the proposed cutoff wall alignment on the left side of the embankment. Predrilling with a sonic drill rig was conducted through the upstream embankment and a 254 mm (10 in.) diameter PVC casing was installed in unhardened grout backfilled boreholes from the work platform to the proposed top of the jet grout column. Prior to commencement of jet grouting, the hardened grout was removed from the casing interior by a 203 mm (8 in.) drill bit, leaving a 25 mm (1 in.) wide annulus of hardened grout in the casing prior to drilling the hole to bedrock for the jet grouting of the column. The column construction began 0.3 m (1 ft) below the top of the bedrock and extended into the extended impervious core zone at elevation 314.0 m (1030 ft) as would be required for cutoff wall construction. Seventeen parametric columns were installed between Station 65+50 and Station 67+00 on the upstream side of the dam. The triple and double fluid jet grout parameters for the upstream field trials are shown in Table 2.

The first six columns (fivetriple fluid, one double fluid) were constructed without significant incident at diameters of 2.6 m (8.5 ft). However, during construction of the seventh column (a triple fluid column), loss of spoil return occurred repeatedly during jet grouting and the contractor was unable to restore continuous spoil return. For this column, the water pressure and grout pressure were both about 440 bars (44 MPa, 6380 psi). When spoil return was lost, water and grout were being injected at the rate of about 800 L/min (200 gal./min) while the air pressure was being maintained at 12 bars (1.2 MPa, 175 psi). Spoil return was lost even though steps, e.g., repeated stroking of the hole, spraying water into top of the hole, and temporarily stopping construction, were performed multiple times to restore spoil return. After about 100 min from when continuous spoil return was first lost, air bubbling was observed in the reservoir just upstream of the work area. After this observation, the contractor abandoned the column and grouted the drill hole. The air bubbling in the reservoir appeared from approximately Station 65+00 to 67+00 (see Fig. 3) and was suggestive of the possibility that ground fracture may have occurred toward the reservoir. It is possible that ground fracturing may have propagated downstream and/or laterally as well; or not at all. The bubbling subsided after about 48 h, suggesting a large volume of air was stored under high pressure prior to the air bubbles appearing in the reservoir. The air was probably stored in the pervious sand foundation materials, and then released along fractures through the embankment and/or natural fine-grained blanket into the reservoir.

Table 2. Jet Grout Parameters for Upstream Parametric Columns (1 bar = 100 kPa)

	Triple fluid	Double fluid
	Three nozzles	Two nozzles
Nozzles (number and diameter cm/in.)	1: Water (0.45–0.55/0.18–0.22) 2: Grout (0.55–0.70/0.22–0.28) 3: Air (0.21/0.08)	1: Grout (0.5–0.65/0.20–0.26) 2: Air (0.21/0.08)
Cement/water ratio	0.9–1.0	0.6−0.9
Water pressure (bars)	440–480	—
Water flow rate (L/min)	340–345	—
Grout pressure (bars)	250–440	430–445
Grout flow rate (L/min)	448	300–500
Air pressure (bars)	7–15	6–12
Airflow rate (L/min)	7,700	4,000–7,000
Withdrawal rate (m/hr)	8.4	6.8
Specific energy (MJ/m)	82–175	179–183

The loss of spoil return and the injection of large volumes of air and fluids under high pressure can fracture in situ materials because pressure builds up almost instantaneously after a loss of spoil return. Fluid injection pressures, which normally are dissipated with head losses in the hydraulic system and in the erosion process, can result in bottom hole pressures far exceeding those required to initiate ground fracturing when spoil return is interrupted even intermittently.

While air escaping into the reservoir may have indicated damage to the embankment and/or the natural fine-grained blanket, the manifestation of damage, e.g., increased seepage or gradient, can occur long after the fracturing occurred, particularly at higher reservoir levels. As a result, KCD carefully monitored the downstream relief wells and toe area for signs of internal erosion, sand boils, and increased seepage for three weeks, including continuous surveillance during the first 24 h. Piezometers were also carefully monitored to identify any changes in foundation piezometric levels. No abnormal reading or observations were made, so the heightened surveillance was reduced after about three weeks. No change in the reservoir level occurred during this period.

Suspension of jet grouting was directed by the KCD to assess possible ground fracturing and to develop techniques for preventing similar incidents. A revised plan of action for spoil blockage allowed a loss of spoil return for 30 s before action to remove the blockage, e.g., stroking the hole with the jet grout drill rod, was required. With implementation of the revised response plan, conditional upstream jet grouting resumed four days later with a double fluid column. The

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

grout injection pressure was about 450 bar (45 MPa, 6525 psi), and the air pressure was about 12 bars (1.2 MPa, 175 psi). During jet grouting, air bubbling was again observed in the reservoir just upstream of the column. After observation of the air bubbling, jet grouting was terminated and the hole was grouted. Spoil return had been quite viscous and somewhat sporadic, but with no more than 10 to 15 s lapses. The revised plan of action was not implemented before air bubbling was observed in the reservoir because the maximum blockage time was less than the 30 s threshold. It became apparent that when spoil return was lost, ground fracturing was occurring because of continued injection of incompressible fluids. This required development of a new action plan that would require additional steps to prevent ground fracturing.

The new action plan involved installation of real-time instrumentation to quickly detect pore pressure changes in the vicinity of jet grouting and two major adjustments in the jet grouting procedure. The first adjustment involved a larger borehole annulus by using a 254 mm (10 in.) casing and 228.6 mm (9 in.) drill bit to drill out the hardened grout in the casing prior to jet grouting instead of a 203.2 mm (8 in.) drill bit. A larger drill bit resulted in the grout annulus being reduced from 25 mm to 12.5 mm (1 in. to 0.5 in.), which created additional space for evacuation of the jet grouting spoil. The second adjustment involved the jet grouting procedure bringing the monitor up into the casing before starting the air and fluid circulation and then moving it to the bottom of the hole instead of starting at the bottom of the hole. This initiated spoil return at a much lower pressure rather than initiating return at the bottom of the drill hole under a full column of native soil. After installation of an array of vibrating wire piezometers with real-time readouts along with open tube devices, jet grouting was resumed in late September 2006. The first three jet grout columns were completed with sporadic or periodic short losses of spoil return, accompanied by brief, immediate increases in piezometric pressure. There were no visible air releases into the reservoir. However, during construction of the fourth column, spoil return was lost shortly after beginning jet grouting and air bubbling again occurred in the reservoir. This air release occurred in close proximity to the prior air release at Station 65+00 to 67+00 and occurred within about 10 s of the loss of spoil return. Instrumentation installed to monitor the jet grouting process showed an immediate pore pressure increase following any interruption in spoil return. Indications of ground fracturing included hardened grout observed in the spoil return of a subsequently constructed column and a subsequent sonic core encountered grout in the natural fine-grained blanket downstream of an adjacent column.

Afterward, the changes required to achieve acceptable ensurance of spoil return and no ground fracturing could not be agreed upon within the terms of the contract, and KCD had to decide whether to terminate the cutoff wall or accept dam safety risks from potential damage to the upstream fine-grained blanket and the extended impervious core zone of the dam. Considerations included not only the results of the jet grouting field trial, but also results of the downstream TPO for ground improvement (both jet grout and deep soil mixing), dam safety risk analysis, and recently completed seismic deformation analyses that showed upstream deformations were within acceptable limits.

Dam Safety Concerns and Jet Grouting Considerations

Based on the results of the TPO and downstream and upstream parametric columns, it was apparent that jet grouting had to be more carefully controlled or serious damage could occur to the embankment and/or fine-grained blanket and pose an unacceptable dam safety risk. Fracturing of the fine-grained blanket and/or upstream impervious core downstream of the cutoff wall could result in shortened flow paths and higher piezometric levels at the downstream toe. Piezometric data suggested the upstream fine-grained blanket caused a significant vertical head loss, so any increased flow would render the cutoff wall ineffective. Fig. 5 shows the reduction in seepage path, and thus vertical head loss, through the fine-grained blanket would greatly increase the hydraulic pressure in the pressure relief wells at the toe of the dam. Additionally, 75 to 90% of the completed field trial jet grout columns had a void at the top because the grout level dropped as a result of bleed and/or unhardened grout permeating into the coarse sands and gravels near the base of the columns. The grout level was subsequently brought to the top of the column; however, intimate contact with the overlying embankment might be difficult to achieve in light of column roof instability. These factors led to concerns about erosion of the foundation and embankment soils and unacceptable increased seepage, hydraulic gradients, and formation of sink holes in the upstream embankment which could lead to dam failure.

In addition to dam safety issues, construction of the cutoff wall posed many challenges. To achieve a 3 m (10 ft) thick wall, the contractor proposed a large column diameter (2.4 to 2.75 m) to minimize the amount and cost of drilling through the upstream embankment materials (rock fill). Not only did the wall have to tie into the foundation bedrock, but the top of the cutoff wall had to be intimately secured to the extended upstream impervious core zone. The columns had to be constructed within strict alignment tolerances and at constant diameter to ensure overlap of adjacent columns. The variable stratigraphy also had to be known in great detail to adjust the specific energy at precise locations along the

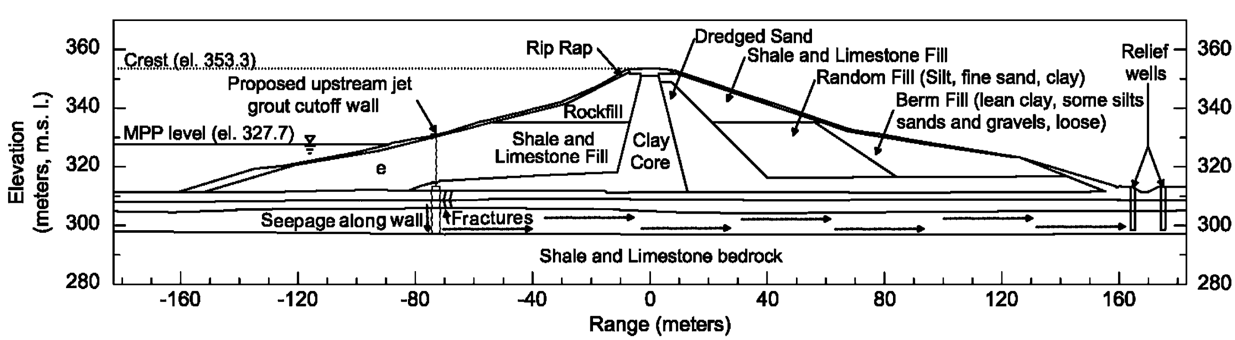


Fig. 5. Risk of decrease in vertical head loss through fine-grained blanket as a result of jet grouting

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

column. Field trials and the TPO showed that stability of the column roof had to be maintained and complete mixing of materials attained to avoid windows in the wall. Based on these concerns, it was not clear the that the proposed cutoff wall would not have gaps and would achieve the 11.9 m (39 ft) head drop required in the specifications (see Stark et al. 2009).

Because of these jet grouting challenges, dam safety risks, and the inability to control these risks, long-term cutoff wall performance (Rice and Duncan 2009 a,b), and results of the seismic deformation analysis described in the next section, the KCD decided to abandon upstream jet grouting, which meant forgoing the upstream cutoff wall and upstream slope stabilization.

Seismic Deformation Analysis

Initial seismic deformation analyses estimated using *Dynaflow* (Popescu 1998) and *TARA3-FL* (*TARA3-FL*; Finn 2004) predicted large displacements of the existing unremediated dam, which justified the retrofit of both upstream and downstream slopes.

The design ground motion for the deformation analyses is the Castaic accelerogram from the 1971 San Fernando, California earthquake, N69W component, scaled to a peak acceleration of $0.3g$. The depth and magnitude of the earthquake are 14 km (9 mi) and 6.5, respectively. The original accelerogram has a peak acceleration and duration of $0.27g$ and 40 s, respectively. The resulting time history matches the 84th percentile acceleration spectra for the MCE in a range near the fundamental period of the dam (0.3 to 0.6 s). The scaling and development of the design ground motion is discussed in Somerville et al. (2003). The design ground motion is considered to be conservative because the response spectra of the design event plots mostly above the evaluation mean plus sigma design response spectrum for the range of natural period of the dam, and yielded the most damaging response.

Later, the KCD appointed an advisory panel to assist with a seismic deformation analysis of the unremediated dam using the software *FLAC* (*FLAC 5.0*) and the calibrated UBCSAND and UBCTOT constitutive models (Perlea 2006). The application of the *FLAC-UBCSAND* analysis led to much smaller permanent displacements than those previously predicted for the existing dam, such that upstream remediation was not required and a better understanding of the remediation required for the downstream slope was obtained. The permanent deformations predicted using *FLAC* are at least an order of magnitude lower than the values estimated using the software *Dynaflow* (Popescu 1998) and *TARA3-FL* (*TARA3-FL*). The reason for this difference is the constitutive soil models used for the foundation materials, and in particular the fine-grained blanket, were more representative of the conditions at Tuttle Creek. For example, the main difference between the *TARA3* and *FLAC* analyses is that *TARA3* (Finn 2004) utilizes a hyperbolic stress-strain model for the fine-grained blanket that could not effectively model the undrained shear behavior, which involves dilation at low effective stresses and high applied-stress ratios. The high applied-stress ratios resulted in unreasonably large strains and low blanket strength, which caused the predicted deformations to be extremely large because the hyperbolic model cannot replicate the concave stress-strain behavior that results from dilation. Another difference is that the *TARA3* model uses an accumulated shear strain, and not maximum shear strain, to estimate shear strength, which results in an undrained residual strength being applied to a substantial portion of the fine-grained blanket and runaway permanent displacements in the blanket. In short, *TARA3* did not properly accommodate the strain softening behavior of the fine-grained blanket.

Some of the limitations with the *Dynaflow* analysis (Popescu 1998) include prediction of liquefaction triggering, no calibration with laboratory results, and no apparent mechanism for incorporating a liquefied strength back-calculated from flow failure case histories (Stark and Mesri 1992). As a result, once liquefaction triggered in both the foundation sands and fine-grained blanket, the shear strength dropped to an extremely low value, e.g., zero, which resulted in the development of large permanent displacements. The *FLAC-UBCSAND* analysis and results are described in Stark et al. (2012) but the estimated permanent deformations are summarized as:

- Crest settlement of about 0.6 m (2 ft);
- Permanent deformations at the upstream toe of less than about 0.6 m (2 ft); and
- Permanent deformations at the downstream toe of about 1.5 m (5 ft), although significantly larger displacement may occur.

The predicted evolution of permanent vertical and horizontal displacements during the earthquake is shown in Fig. 6 at several locations on the surface of the dam. The horizontal velocity and acceleration design ground motion is initially large and dissipates, while most of the displacement accumulates in a steady and gradual manner over the first 20 s of the design ground motion. The estimated permanent deformations result from limited liquefaction at the upstream toe, extensive liquefaction of foundation sand at the downstream toe, and liquefaction of the fine-grained blanket under the dam (see Fig. 7).

A comparison of the permanent deformations discussed previously and the allowable post-earthquake deformations of 1.5 m (5 ft) vertically at the crest, 3 m (10 ft) laterally at the upstream toe, and 0.3 m (1 ft) laterally at the downstream toe shows that potential movements of the downstream slope were still problematic for the design ground motion. Furthermore, the estimated downstream deformations are sensitive to the extent to which liquefaction occurs beneath the downstream section of the dam and could be larger than predicted. As a result, stabilization of the downstream slope and toe was recommended and implemented. Because jet grouting was deemed a high-risk solution for a low-risk problem, the original plans for upstream slope stabilization and an upstream cutoff wall were eliminated. Eliminating the upstream slope stabilization and cutoff wall eventually resulted in a project savings of about \$65 million dollars. However, stabilization of the downstream slope and toe was still needed to protect the downstream seepage control system and provide assurance that a flow slide would not occur.

Downstream Slope Remediation

Downstream slope stabilization was planned to be performed using jet grouting or soil mixing along the downstream toe. However, concerns about jet grouting and the jet assisted soil mixing proposed by the contractor resulted in these stabilization techniques being reconsidered for the downstream slope. The concerns about downstream-jet-assisted soil mixing involved the close proximity of the horizontal sand drain in the downstream shell of the dam and the relief well system just downstream of the slope toe which could be clogged by fugitive grout. Soil mixing was also abandoned because the contractor's proposal was limited to jet assisted soil mixing equipment. Additionally, the high cost of jet grouting and soil mixing was a concern. Given the concerns with jet grouting and jet assisted soil mixing and economics, another technology was sought that could be used without having to revise the EIS and construction contract.

Given the better-than-expected performance of the C-B cutoff wall surrounding the downstream test section area, the ease of

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

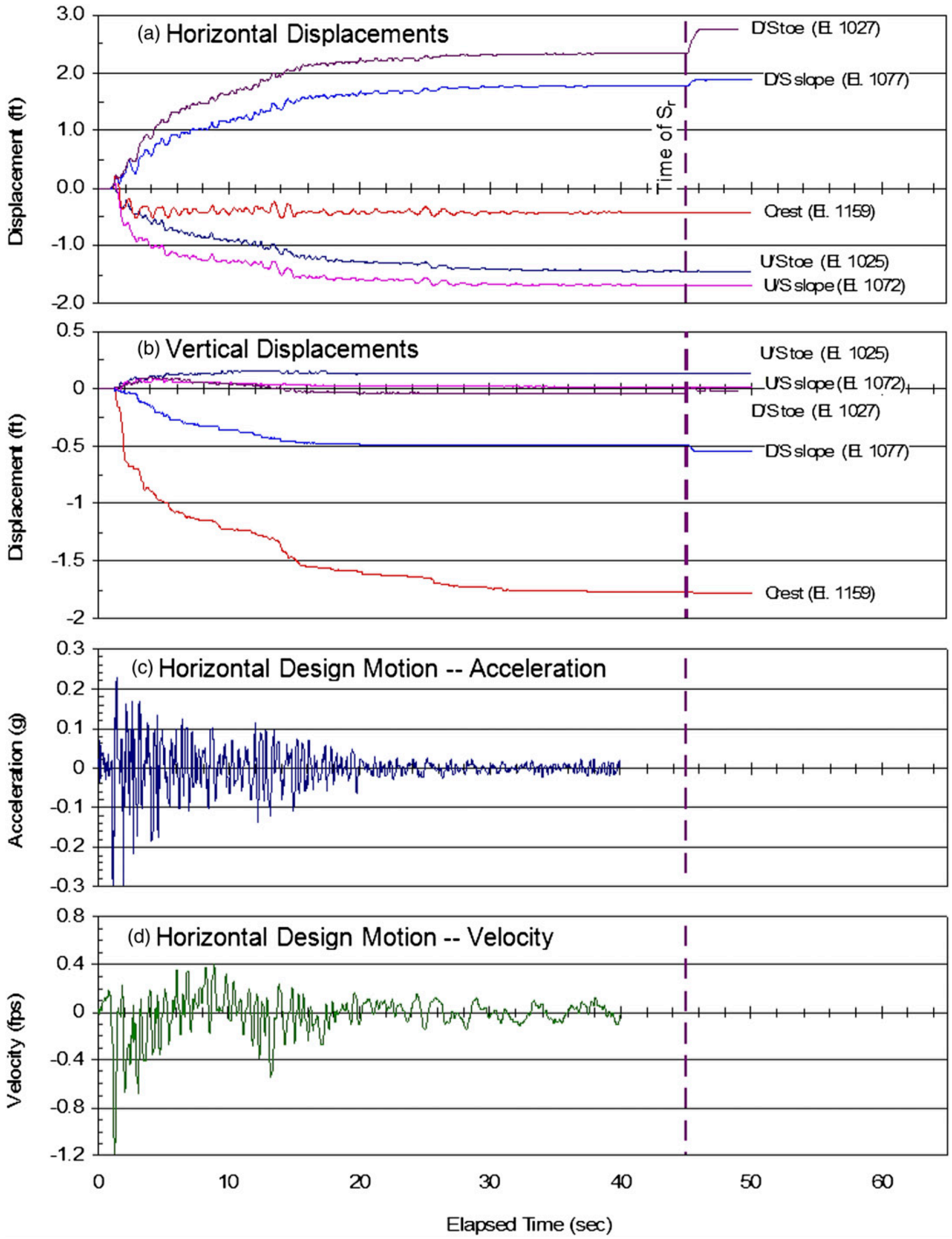


Fig. 6. Time histories of (a) horizontal, (b) vertical, (c) horizontal acceleration design motion, and (d) horizontal velocity design motion

construction, ease in verifying stabilization limits, and possible economic benefits, it was decided to use transverse C-B walls for stabilization of the downstream slope. Historically, most C-B walls have been used for permeability reduction and seepage control rather than as a structural element, so some design and testing was required to obtain a C-B mix that could meet the wall strength requirements. The technical issues to be addressed were to determine the configuration of the slurry walls to stabilize the downstream slope without impacting foundation underseepage, develop a suitable construction technique, and find a C-B slurry mix that would yield the desired static and seismic performance of the wall. Some of the concerns about self-hardening slurry were: typical unconfined compressive strength (UCS) is less than about 700 kPa (100 psi) at an age of 28 days, brittle stress-strain behavior, unknown large displacement strength, and maintaining slurry workability during construction.

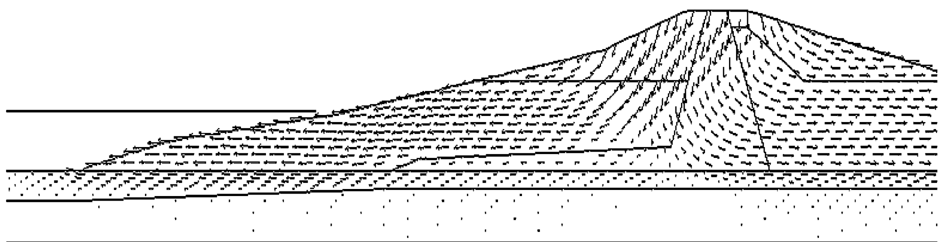
To achieve the desired structural capacity of the walls, a UCS of 680 kPa (98 psi) in 28 days was required. A seismic deformation analysis was performed using the calibrated *FLAC* model described previously to evaluate the impact of the transverse shear walls on downstream slope deformation. This analysis shows that the unreinforced and relatively brittle shear walls would be exposed to large shear strains during or immediately after the design seismic event. Such loading may crack the shear walls, after which the frictional resistance of the cracked section would govern the ability of the shear walls to resist gravitational forces induced by the slope.

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

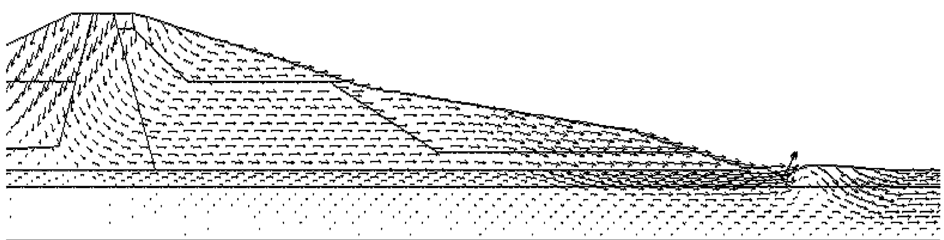
Therefore, the frictional resistance had to be sufficient to limit deformations at the downstream toe after cracking because of the presence of the fragile pressure relief well system. Fig. 8 shows the critical limit equilibrium failure surface passing through the C-B shear wall and exiting at the toe of the dam. The close proximity of the shear walls to the relief wells necessitated the high strength requirement for the walls. As a result, KCD specified that the completed walls exhibit a peak UCS of at least 2,060 kPa (300 psi) and a large displacement friction angle of 40°. The higher UCS and large displacement friction angle necessitated the use of blast furnace slag as a C-B additive. In general, blast furnace slag increases UCS but results in brittle stress-strain behavior. To facilitate mixing, pumping, and stress-strain behavior, bentonite was added to the mix but other clays, including attapulgite and sepiolite, were considered. The addition of 50 to 75% ground, granulated, blast furnace slag cement was studied to develop the optimal mix (Axtell and Stark 2008). A soil-cement-bentonite slurry was also considered, but space limitations on the downstream slope, higher cost, and other concerns led to the use of self-hardening C-B slurry.

After considerable testing and analysis, the selected slurry mix consists of a 50/50 blend of portland cement and ground, granulated, blast furnace slag cement with 4.5% bentonite. The mix had a 0.5 cement water ratio and used approximately 1% Lamsperse, a retarder admixture, to slow hardening while the wall is being excavated.

The majority of the transverse shear walls were constructed using this self-hardening C-B slurry and a clam-shell device. A long-reach excavator used during production tests resulted in 20% higher strength walls than those constructed with a clam-shell excavator (Axtell et al. 2009), but both methods resulted in walls that met the performance specifications. The majority of the walls were constructed with a clam-shell device because there were fewer



(a) **Upstream displacement vectors**



(b) **Downstream displacement vectors**

Fig. 7. (a) Upstream and (b) downstream displacement vectors at end of post dynamic analysis with application of liquefied strengths for initial unremediated case

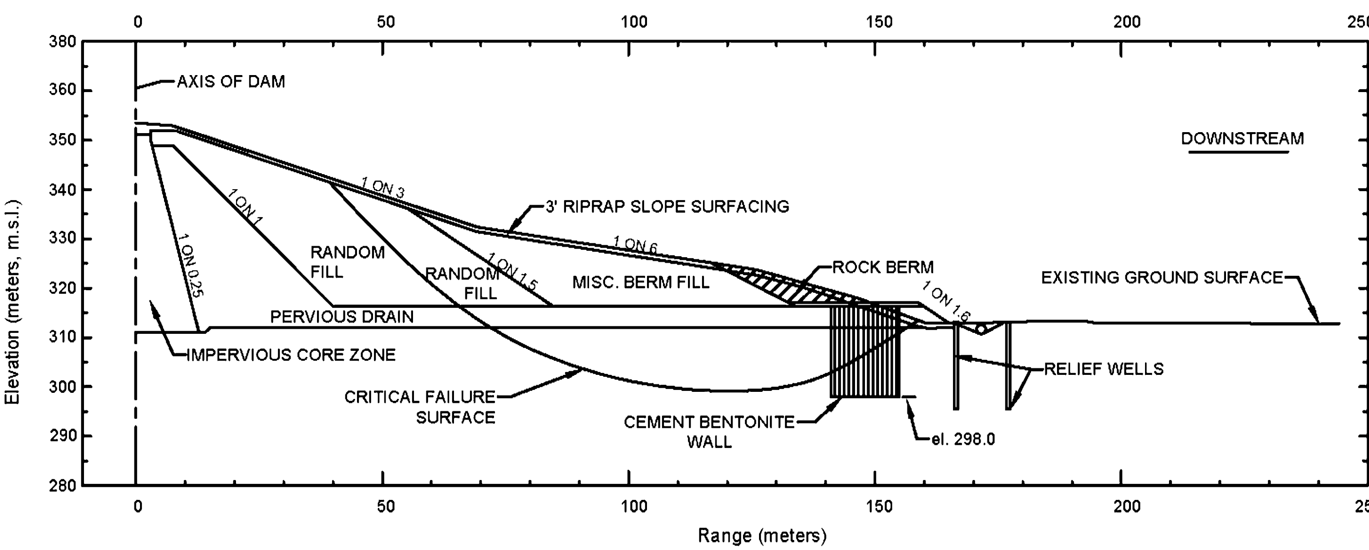


Fig. 8. Cross section showing cement-bentonite (C-B) transverse shear wall and critical limit equilibrium failure surface

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

maintenance issues. The shear walls are typically 13.7 m (45 ft) long, 1.2 m (4 ft) wide, and 18.9 m (62 ft) deep. The 351 shear walls have a 0.31 m (10 ft) gap between adjacent walls, which corresponds to a replacement ratio of about 29% (Axtell et al. 2010). The walls were installed transverse to the dam axis between stations 24+92 and 73+60 (see Fig. 3). Stations 10+00 to 25+00 and Stations 70+00 to 75+00 were not included because of the presence of nonliquefiable materials and the large upstream and downstream stabilizing berms that were installed during the original construction.

Limit equilibrium and *FLAC* analyses were used to design and verify the performance of the transverse shear walls. The limit equilibrium analyses estimated an unremediated factor of safety (FS) of about 0.9 and a FS of 1.25 with the proposed transverse shear walls. A FS of 1.25 was deemed acceptable for the level of estimated permanent deformation obtained using the calibrated *FLAC* model and the design ground motion based on the Castaic accelerogram described previously (Marcuson et al. 1990). The *FLAC* analyses estimated the following seismically-induced permanent deformations of the remediated dam:

- Crest settlement of about 0.5 m (1.5 ft);
- Permanent deformations at the upstream toe of less than 0.5 m (1.5 ft); and
- Permanent deformations at the downstream toe of less than 0.6 to 0.9 m (2 to 3 ft).

The displacements could be larger or smaller than estimated if field shaking and engineering properties differ from those used in the seismic analysis. However, it was concluded that the transverse shear walls provide a worthwhile benefit to the seismic response of the dam and reduce the likelihood of a large liquefaction-induced flow slide as indicated by limit equilibrium slope stability analyses. In addition to the 351 transverse shear walls installed along the downstream toe of the dam, the existing relief well ditch was replaced with a buried collector system to further improve downstream slope stability (USACE 2007) (see Fig. 8).

Recommendations

This paper provides a summary of the recently completed seismic retrofit of Tuttle Creek Dam near Manhattan, Kansas. This case history provides the following recommendations for future dam seismic retrofit projects:

- Developments during construction can lead to changes in retrofit technology; as a result, the decision documents, e.g., EIS, construction contract, and various design documents, should provide flexibility to designers and owners to accommodate possible changes in retrofit technology;
- The use of jet grouting on dams with active reservoirs can pose dam safety concerns that cannot be easily controlled or assessed and should be carefully evaluated; some of the dam safety risks involved with high pressure and flow rate required for jet grouting large diameter soilcrete columns include: (1) ground fracturing which can lead to undesirable seepage pathways and higher hydraulic gradients in erodible materials; and (2) added uncertainty in securing the top of the jet grout columns to impervious embankment and/or foundation materials;
- Small diameter jet grout columns may be feasible for active dams because lower jet grouting pressures, e.g., air, water, and grout pressures, are required to erode and evacuate the native materials; however, high pressures can still develop and cause ground fracturing if spoil return is not maintained;
- If jet grouting is used for an active dam, a rapid monitoring system should be installed, such as piezometers, which can quickly reflect the build-up of pressure in the foundation or embankment materials; in addition, a system to monitor the reservoir for any indication of bubbling and downstream flow rate and composition should be developed;
- An action plan for jet grout spoil blockages should be developed and approved prior to jet grouting to prevent or limit the amount of ground fracture; the action plan should recognize that poor spoil return and temporary blockages can lead to elevated bottom hole pressures occurring before the plan is required to be implemented; as a result, the action plan must have early triggers to change grouting methodology so large pressures and ground fracturing do not occur; this action 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; and
- Transverse shear walls constructed using slurry trench techniques and self-hardening C-B slurry appears to be a viable slope stabilization technique for liquefiable foundation materials; however, the geometry of the shear walls must be designed to preserve underseepage flows.

References

Axtell, P., and Stark, T. D. (2008). “Increase in shear modulus by soil mix and jet grout methods.” *Deep Found. Inst. J.*, 2(1), 11–22.

Axtell, P., Stark, T. D., and Dillon, J. C. (2009). “Strength difference between clam-shell and long-reach excavator constructed cement-bentonite self-hardening slurry walls.” *Contemporary Topics in Ground Modification, Problem Soils, and Geo-Support, ASCE Geotechnical Special Publication No. 187*, M. Iskander, D. F. Laefer, and M. H. Hussein, eds., ASCE, Washington, DC, 297–304.

Axtell, P., Stark, T. D., and Dillon, J. C. (2010). “Peak and post-peak shear strength of cement-bentonite.” *Deep Found. Inst. J.*, 4(1), 59–65.

Castro, G., Perlea, V., and Walberg, F. C. (2003). “Dynamic properties of cohesive soil in the foundation of an embankment dam.” *Proc., 21st Int. Conf. on Large Dams*, International Commission on Large Dams (ICOLD), Paris, Q.83–R.3.

Empson, W. B., and Hummert, J. B. (2004). “Warning the downstream community after an earthquake.” *Proc., Dam Safety Conf.*, Association of State Dam Safety Officials, Lexington, KY.

Fast Lagrangian Analysis of Continua (FLAC) 5.0 [Computer software]. Minneapolis, MN, Itasca Consulting Group, Inc.

Finn, W. D. L. (2004). “Report on Seismic Deformation Analyses of Tuttle Creek Dam.” U.S. Army Corps of Engineers, Kansas City District, Kansas City, MO.

Marcuson, W. F., Hynes, M. E., and Franklin, A. G. (1990). “Evaluation and use of residual strength in seismic safety analysis of embankments.” *Earthq. Spectra*, 6(3), 529–572.

Lane, K. S., and Fehrman, R. G. (1960). “Tuttle Creek Dam of rolled shale and dredged sand.” *J. Soil Mech. Found. Div.*, 86(SM6), 11–34.

Perlea, V. G. (2006). “Draft memorandum: Tuttle Creek Dam, summary of soil properties for use in FLAC analysis.” U.S. Army Corps of Engineers, Kansas City District. USACE, Washington, DC.

Prevost, J. H. (2002). Dynaflow - A nonlinear transient finite element analysis program, Version 02, Technical Report, Department of Civil and Environmental Engineering, Princeton University, Princeton, NY. http://www.princeton.edu/~dynaflow/.

Popescu, R. (1998). “Evaluation report, Appendix V, Phase II, special investigations, Part 2: Detailed field investigation and evaluation of repair alternatives: Seepage analysis, Appendix F, *Dynaflow* analysis,” U.S. Army Corps of Engineers, Kansas City District, Kansas City, MO.

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986

Rice, J. D., and Duncan, J. M. (2009a). "Findings of case histories on the long-term performance of seepage barriers in dams." *J. Geotech. Eng.*, 136(1), 2–16.

Rice, J. D., and Duncan, J. M. (2009b). "Deformation and cracking of seepage barriers in dams due to changes in the pore pressure regime." *J. Geotech. Eng.*, 136(1), 16–25.

Seed, H. B., and Harder, L. F. (1990). "SPT based analysis of cyclic pore pressure generation and undrained residual strength," *Proc., H. Bolton Seed Mem. Symp.*, BiTech Publishers, Ltd., Vancouver, BC, Canada, Vol. 2, 351−376.

Seed, H. B. and Idriss, I. M. (1971). "Simplified procedure for evaluating soil liquefaction potential." *J. Soil. Mech. Foun. Div.*, 97(9), 1249−1273.

Somerville, P., Walberg, F. C., and Perlea, V. G. (2003). "Seismic hazard analysis and selection of design earthquake for a dam in Kansas. *Proc. 21st Int. Conf. on Large Dams,* International Commission on Large Dams (ICOLD), Paris, Vol. 3, 473–495.

Stark, T. D., Axtell, P. J., Lewis, J. R., Dillon, J. C., Empson, W. B., Topi, J. E., Walberg, F. C. (2009). "Soil inclusions in jet grout columns." *Deep Found. Inst. J.*, 3(1), 44–55.

Stark, T. D., Lewis, J. R., Castro, G., Walberg, F. C., and Mathews, D. L. (2011). "Liquefaction subsurface investigation for Milford Dam." *Can. Geotech. J.*, 48(10), 1504–1519.

Stark, T. D., Beaty, M. H., Byrne, P. M., Castro, G., Walberg, F. C., Perlea, V. G., Axtell, P. J., Dillon, J. C., Empson, W. B., Mathews, D. L. (2012). "Seismic deformation analysis of Tuttle Creek Dam." *Can. Geotech. J.*, 49(3), 323–343.

Stark, T. D., and Mesri, G. (1992). "Undrained shear strength of liquefied sands for stability analyses." *J. Geotech. Eng.*, 118(11), 1727–1747.

TARA3-FL [Computer software]. Dept. of Civil Engineering, Univ. of British Columbia, Canada.

U.S. Army Corps of Engineers (USACE). (1999). "Engineering and design for civil works projects." *Engineer Manual ER 1110-2-1150*, U.S. Army Corps of Engineers, Washington, DC.

U.S. Army Corps of Engineers (USACE). (2007). "Tuttle Creek Dam: Foundation modification project: Downstream stabilization and buried collector system, plans and specifications." U.S. Army Corps of Engineers, Kansas City District, Kansas City, MO.

J. Geotech. Geoenviron. Eng., 2013, 139(6): 975-986