Soil Improvement for Seismic Retrofit of Tuttle Creek Dam

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ABSTRACT: This paper discusses the seismic retrofit of Tuttle Creek Dam near Manhattan, Kansas. Seismic analyses, construction, and stabilization techniques are presented. Constructability and dam safety issues, along with 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 jet-assisted soil mixing, but ultimately was accomplished using self-hardening cement-bentonite slurry to construct transverse shear walls to reinforce the liquefiable foundation sands. A total of 351 transverse shear walls were constructed along the downstream toe by primarily clam shell 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 foundation sands. The walls are spaced at 4.3m on center along the downstream toe for a replacement ratio of about 29%.

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. Tuttle Creek Dam is a zoned earthfill embankment (Lane and Fehrman, 1960) and is located about 10 km (6 miles) 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). A typical cross-section of the dam is shown in Figure 1. The reservoir capacity is over 2.5 million megaliters (two million acre-ft) at flood control pool.

Soils in the native alluvial foundation of the dam consist of 2.4 to 8.2 m (8 to 27 ft) of silt and clay (fine-grained soil blanket) underlain by deposits of sand, silty sand, and gravelly sand of variable thickness 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. 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 natural fine-grained soil blanket underlying the embankment, extended impervious fill zone, and relief wells with a collection trench along the downstream toe (see Figure 1) control significant foundation seepage across the valley.

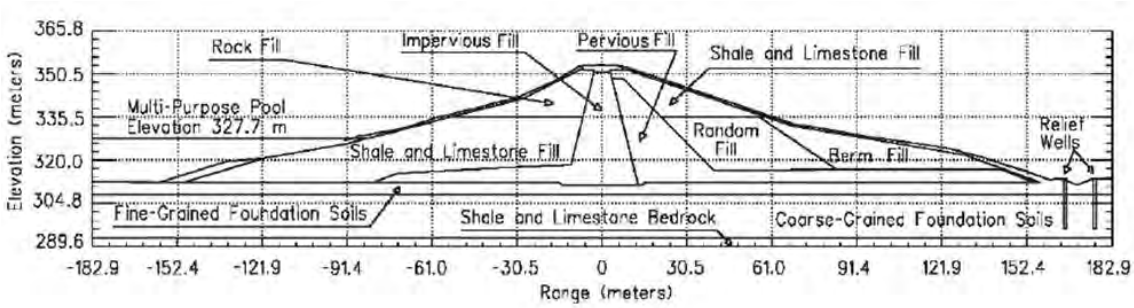

Figure 1. Typical dam cross section around Station 50+00

Tuttle Creek Dam is located in an area of moderate seismicity associated with an old continental rift zone. 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 Maximum Credible Earthquake (MCE), i.e., design ground motion, is a magnitude 6.6 event a distance of 20 km from the site. The MCE has a peak horizontal ground acceleration (PHGA) of 0.28g mean, and 0.56g mean plus one standard deviation (Somerville et al., 2003). The peak ground acceleration (PGA) has a return period of about 3000 years.

Site investigations and geotechnical analyses identified the upper alluvial foundation sands as being susceptible to liquefaction when subjected to the MCE. The clays and silts of the natural fine-grained blanket also appeared susceptible to pore pressure generation and strength loss during earthquake loading (Castro et al., 2003). Both of these responses indicated unacceptable post-earthquake permanent deformations.

SEISMIC ANALYSES AND LIQUEFACTION ASSESSMENT

Allowable post earthquake deformation criteria established by USACE for Tuttle Creek Dam were: 1.5 m (5 ft) vertically at the crest, 3 m (10 ft) laterally at the upstream toe, and 0.3 m (1 foot) laterally at the downstream toe to prevent damage to the relief wells. Finite element deformation analyses performed in 2002 using DYNAFLOW and TARA-3FL predicted unacceptably large deformations both upstream and downstream, largely as a result of liquefaction of foundation sands and an assumed loss of strength in the low plasticity natural fine-grained blanket. Pressure relief wells at the downstream toe are critical to protect the dam against foundation internal erosion and the predicted earthquake induced deformations would likely disable these relief wells. With damage to or loss of the relief wells, the likely failure mechanism is internal erosion/piping of the foundation soils. As a result, USACE 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 flooding to portions of the City of Manhattan and beyond. To prevent an uncontrolled reservoir release, stabilizing the foundation under both upstream and downstream slopes was recommended. Additionally, upstream stabilization was to include a seepage cutoff wall to reduce the importance of the existing seismically vulnerable pressure relief well system.

UPSTREAM CUTOFF WALL AND SLOPE REMEDIATION

In 2005, USACE awarded a construction-manager-at-risk type contract. USACE also selected an Advisory Panel (AP) to assist with analysis, design, submittal reviews, construction, quality assurance, and to conduct site visits.

The original remediation design called for upstream slope stabilization of liquefiable foundation silty clays and sands and construction of an upstream cutoff wall by jet grouting through pre-drilled 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. The downstream slope stabilization was initially planned to consist of stabilization of liquefiable foundation silty clays and sands using jet grouting or jet-assisted soil mixing along the downstream toe.

Downstream Test Program and Parametric Columns

In the spring of 2006, a jet grout and soil mixing test program began downstream of the dam and is described by Stark et al. (2009). At this location, jet grout and soil mixing trials with various parameters could be conducted without risk to dam safety, although under in-situ stress conditions that are significantly different than expected upstream of the dam. USACE's goal of the test program was to gather information that could be used in both upstream and downstream stabilization design which were scheduled to follow cutoff wall construction. A total of twenty-seven (27) parametric jet grout and twenty-seven (27) soil mixed columns were constructed as part of this program.

The Contractor also constructed downstream non-contractual parameteric columns in the test program area to assist in developing the most efficient parameters for cutoff wall construction. The performance requirements of the cutoff wall included a minimum ten foot cutoff wall thickness, with a minimum of two rows of columns to obtain a piezometeric head drop of 39 feet across the wall. The minimum required unconfined compressive strength of the wall was to be 1,170 kPa (170 psi) or higher.

A total of eighteen (18) jet grout columns were constructed using triple fluid (air, water, and grout) jet grouting techniques and nine (9) were constructed using double fluid technique (only air and grout). A cement-bentonite slurry wall was constructed around the test program column area to facilitate dewatering and excavation of the area after column construction.

After the columns were complete and subsequent coring of the columns had been conducted to measure soilcrete properties, the groundwater within the cement-bentonite (C-B) slurry wall that surrounded the trial 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 jet grout and soil mix columns and inspection of the columns. The excavation allowed for measurement of column diameter at various depths and through multiple soil strata. In addition, cutting or sectioning of some of the jet grout columns was performed to determine column integrity and homogeneity. Stark et al. (2009) describes the downstream test area and the excavation and cutting results that showed the jet grout columns contained more than 40 to 50% native soil. This native soil was not evacuated during the jet grout process and the soil inclusions were still intact, indicating the erosion process did not break up the native soil. Most of the soil inclusions were greater than 75 to 100 mm (3 to 4 inches). The observed inclusions in completed jet grout columns included significant and large pieces of both fine-grained (silts and clays) and coarse grained (fine sands and sands) soils. Stark et al. (2009) also describes the sectioning of the soil mix columns. In contrast to the completed jet grout columns, the soil mix columns were essentially homogeneous soilcrete with only a few small soil inclusions. The soil mix columns also had a uniform diameter whereas the diameter of the jet grout columns varied considerably.

Upstream Jet Grout 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 upstream earth and water pressures are higher than downstream. In early July 2006 the contractor also constructed upstream parametric columns to develop correlations between column diameter, jet grout energy, and material type in the presence of the embankment and reservoir. These parametric columns were constructed from a work platform created to construct the cutoff wall at elevation 333.8m (1095 ft), immediately downstream of the proposed cutoff wall alignment on the left side of the embankment. Pre-drilling with a sonic drill rig was conducted through the upstream embankment and a 25.4cm (8 inch) PVC casing with no end cap to prevent floating was installed in grout backfilled boreholes from the work platform to the proposed top of column. The PVC casing was used to prevent borehole squeezing while the jet grout monitor was in the foundation soils and jet grouting was occurring. Prior to commencement of jet grouting, the hardened grout was removed from the interior of the PVC casing by drilling. The column construction began 0.3 m (1 ft) below top of bedrock and extended into the base of the extended impervious fill zone at elevation 314.0 m (1030 ft) as would be required for cutoff wall construction. In total seventeen upstream parametric columns were installed.

The first six columns (five triple fluid and one double fluid) were constructed without significant incident. However during construction of the seventh column (a triple fluid column), loss of spoil return occurred continually during jet grouting and the contractor was unable to restore a 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 liters (200 gallons) per minute 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 minutes from when continuous spoil return was first lost, air bubbling was noticed in the reservoir just upstream of the work area. After this observation, the contractor abandoned the column and grouted the drill hole. Air bubbling in the reservoir suggested ground fracturing may have occurred, at least towards the reservoir. The bubbling subsided after about 48 hours suggesting a large volume of air was stored under high pressure prior to air bubbles appearing in the reservoir. The air was likely stored in the pervious sand foundation materials and then released along fractures through the embankment and/or natural fine-grained blanket into the reservoir.

While air escaping into the reservoir may have indicated damage to the embankment and/or the natural fine-grained blanket, the manifestation of damage – increased seepage gradient – can occur long after the hydraulic fracturing occurred, particularly when the reservoir level increases. As a result, USACE increased monitoring of the downstream relief wells and toe area for signs of internal erosion, sand boils, and increased seepage for several months including continuous surveillance immediately following. Piezometers were also monitored to identify any changes in foundation piezometric levels. No abnormal reading or observations were detected including following high pool events of 7.0, 8.2, and 9.8 m (23, 27, and 32 ft) above multipurpose pool level which have occurred since the incident.

Suspension of jet grouting was directed by the USACE to assess possible ground fracturing and assist the Contractor in developing techniques to prevent similar incidents of ground fracturing, which was prohibited in the contract documents. 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, was required. In addition, USACE installed a significant grid of vibrating wire piezometers at various elevations and a real-time monitoring system to help determine if and when ground fracturing was commencing. With implementation of the revised response plan and real-time monitoring system, conditional upstream jet grouting resumed with a double fluid column. The grout injection pressure was about 450 bar (45MPa, 6525 psi) and the air pressure was about 12 bars, (1.2 MPa, 175 psi). During jet grouting, spoil blockage again occurred and the real time monitoring system showed responses within a few seconds. Air bubbling was again observed in the reservoir just upstream of the column. After observation of the increase in foundation pressures via the vibrating wire piezometers and the related air bubbling, jet grouting was terminated and the hole grouted. Spoil return had been quite viscous and somewhat sporadic, but with no more than 10 to 15 second lapses. The revised plan of action was not implemented before air bubbles were observed in the reservoir because the maximum blockage time was less than 30 seconds which was the "trigger" time. It became apparent when spoil return is lost, the continued injection of incompressible fluids can quickly cause ground fracturing. This required development of a new plan that would take additional steps to prevent ground fracturing. It was known that high pressures were required to achieve large diameter jet grout columns but the contract required that ground fracturing was not to be induced. The Contractor had to develop a system to achieve their desired column diameter without inducing ground fracturing. This proved problematic because of blockages and an unwillingness to increase casing diameter which would have created a larger annulus for eroded soil to be evacuated through to the ground surface.

Ultimately the changes required to achieve an acceptable level of assurance of spoil return and no ground fracturing could not be agreed upon within the terms of the contract and USACE had to decide whether or not to terminate the cutoff wall or accept dam safety risks from potential damage to the upstream fine-grained blanket and the extended impervious zone of the dam. Considerations included the results of the upstream jet grouting field trial, the downstream test program for ground improvement (both jet grout and deep soil mixing), and recently completed seismic deformation analyses, described below, that showed upstream deformations were within acceptable limits.

SEISMIC DEFORMATION ANALYSIS USING FLAC

Due to new modelling systems and piezometer information from newly installed piezometers, USACE selected the AP to assist with a permanent seismic deformation analysis of the unremediated dam using the software FLAC (Itasca 2000) and the calibrated UBCSAND and UBCTOT constitutive soil models using field and laboratory data. UBCTOT was used to model softening and strength loss of the fine grained materials and was based on testing of high quality undisturbed samples and in situ vane shear tests. The application of the FLAC analysis led to prediction of much lower permanent displacements than those previously predicted using DYNAFLOW and TARA-3FL for the existing dam, such that upstream remediation was not required and led to a better understanding of the remediation required for the downstream slope. The main differences between the DYNAFLOW and FLAC analyses are that DYNAFLOW does not model post-liquefaction stress-strain behaviour or shear strength which results in an extremely low value, e.g., zero, being assigned to the fine-grained blanket which results in large displacements at the base of the embankment. The main differences between the TARA-3FL and FLAC analyses are TARA-3FL uses a hyperbolic stress-strain model that could not model the post-peak shear behaviour of the fine-grained blanket and uses accumulated shear strain, not maximum shear strain, which results in an undrained residual strength being applied to a substantial portion of the blanket and "runaway" displacements in the blanket.

The FLAC seismic deformation analysis (see Stark et al., 2010) yielded the following estimates of permanent deformation: crest settlement of about 0.6 m (2 ft), permanent deformations at upstream toe of less than about 0.6 m (2 ft), and permanent deformations at downstream toe of about 1.5 m (5 ft). These estimated permanent deformations resulted from limited liquefaction (high seismically induced pore water pressures) of the fine-grained blanket and upper portion of the foundation sand at the upstream toe, extensive liquefaction of foundation sand at the downstream toe, and liquefaction of the fine-grained blanket under the main portion of the dam.

A comparison of the permanent deformations above 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 revealed that potential movements of the downstream slope were still problematic for the design ground motion. As a result, stabilization of the downstream slope and toe was recommended and implemented. USACE and AP did did not feel the benefits of a successfully constructed cutoff wall and upstream stabilization was worth the high probability of damage to the foundation blanket and/or embankment. 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 liquefaction induced 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 contractor’s jet-assisted soil mixing generated by the upstream jet grouting problems resulted in these stabilization techniques being reconsidered for the downstream slope. The concerns about jet grouting were deemed valid for the downstream slope because of the close proximity of the horizontal sand drain in the downstream shell of the dam (see Figure 1) 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 only proposed using jet-assisted soil mixing equipment instead of paddle soil mixing equipment. The same risks of jet grouting were inferred for the jet-assisted soil mixing. 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. More recently, the USACE New Orleans District has used this jet assisted soil mixing for levee improvement with satisfactory results (Schmutzler et al., 2012).

Given the better than anticipated performance of the C-B cutoff wall surrounding the downstream test section area, the ease of construction, ease in verifying stabilization limits, uniformity, and possible economic benefits, it was decided to attempt to use C-B walls for stabilization of the downstream slope. Historically most C-B walls have been used for hydraulic conductivity reduction and seepage control rather than as a structural element, so some design and testing was required to obtain a C-B mix and design that could meet wall strength requirements for downstream slope stabilization. The technical issues to be addressed were to determine the configuration of the slurry walls to stabilize the downstream slope without impacting continuous foundation underseepage, develop a suitable construction technique, and find a C-B slurry mix that would yield the desired wall performance. Some of the initial concerns about self-hardening slurry included: the relatively low typical unconfined compressive strength (UCS), a brittle stress-strain behaviour, unknown large strain or displacement strength, and 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 for the wall geometry and spacing. A seismic deformation analysis was performed using the calibrated FLAC model described above to evaluate the impact of the transverse shear walls on downstream slope deformation. This analysis showed 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 and shear strains 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. Large deformations at the downstream toe are not acceptable because of the presence of the fragile pressure relief well system. As a result, USACE specified that the completed walls had to exhibit a peak UCS of at least 2,060 kPa (300 psi). Walls with this undrained shear strength were shown through drained Consolidated-Undrained (CU) triaxial testing to have a large strain (> 10%) friction angle of at least 40 degrees. The higher UCS and large strain friction angle necessitated the use of an additive such as slag. In general, blast furnace slag increases UCS but results in more brittle stress-strain behaviour. The addition of 50 to 75% ground, granulated, blast furnace slag cement was studied. Greater strength was achieved with higher ratios of slag, however available slag resources resulted in a 50% blend of Portland cement and slag. To facilitate mixing, pumping, and stress-strain behaviour, clays other than bentonite, including attapulgite and sepiolite, were considered. Cement/water ratios between 0.3 and 0.5 were trialled to determine an appropriate mix.

A self-hardening soil-cement-bentonite slurry was also considered but mixing space limitations on the downstream slope, higher cost, and other concerns lead to use of self-hardening C-B slurry. Plastic cement walls also were attempted, however the contractor experienced trench stability failures in the test walls, probably due to a low slurry density, and they were not considered further.

After considerable testing and analysis, the selected slurry mix of a 50/50 blend of Portland cement and ground, granulated, blast furnace slag with 4.5% bentonite was selected for the transverse shear walls. This mix has a 0.5 cement/water ratio and used approximately 1% Lamsperse, a retarder admixture, to slow hardening while the wall was being excavated. Slurry permeation borings were performed just outside the wall limits to determine whether or not permeation would impact underseepage flows. In general, these borings showed global permeation of only a couple of inches or less throughout the depth of the walls, with farther permeation along isolated coarse grained lenses.

The majority of the transverse shear walls were constructed using the selected self-hardening cement-bentonite 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 clamshell excavator (Axtell et al., 2009) but both methods resulted in walls that met the performance specifications. However, the majority of the walls were constructed with a clamshell bucket as the contractor preferred this equipment because there were fewer maintenance and repair issues, i.e., the long reach excavator was too fragile for the deep excavations. 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 3.1 m (10 ft) gap between adjacent walls which corresponds to a replacement ratio of about 29%. The walls were installed transverse to the dam axis between stations 24+92 and 73+60 (see Figure 2). Stations 10+00 to 24+92 and Stations greater than 73+60 were not deemed problematic because of the presence of non-liquefiable materials, limited relief well flows in these areas, and the large upstream and downstream stabilizing berms that were installed during original construction. In addition to the 351 transverse shear walls installed along the downstream toe of the dam, the existing relief well ditch was filled and replaced with a buried collector system to provide additional support to the downstream slope toe.

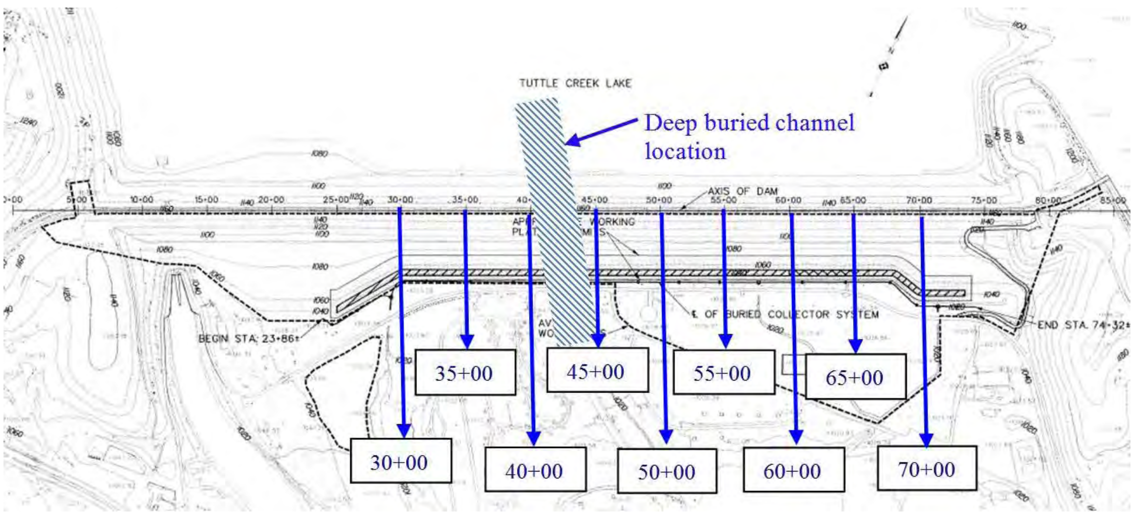

Figure 2. Plan view showing location of transverse shear walls from Stations 24+92 to 73+60

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 (see Figure 3 for a typical cross-section and critical failure surface). A FS of 1.25 was deemed acceptable for the level of estimated permanent deformation obtained using the calibrated FLAC model. The FLAC analyses estimated the following seismically-induced permanent deformations of the remediated dam using the design ground motion of the scaled Castaic accelerogram from the 1971 San Fernando, California earthquake, i.e., N69W component: crest settlement of about 0.5 m (1.5 feet), permanent deformations at upstream and downstream toes of less than 0.5 m (1.5 feet) and 0.6 to 0.9 m (2 to 3 ft), respectively. Because of model uncertainties, these estimated deformations are general magnitudes that were used to evaluate the pattern of post remediated behavior of the embankment. The exact magnitude of deformations are not as critical as understanding the patterns and sensitivity of those deformations within the variability of the modeled parameters and within the physical variability of the in-situ materials. The FLAC analyses indicated the shear walls could experience damage when subjected to the design or similar large ground motion. The high large strain friction angle indicated by drained CU triaxial tests, will likely result in the shear walls providing adequate shear resistance during the design ground motion and subsequent aftershocks. If observed displacements were extremely large, the shear walls may not provide sufficient resistance in subsequent large ground motions so their resistance would have to be re-evaluated after the design ground motion occurs.

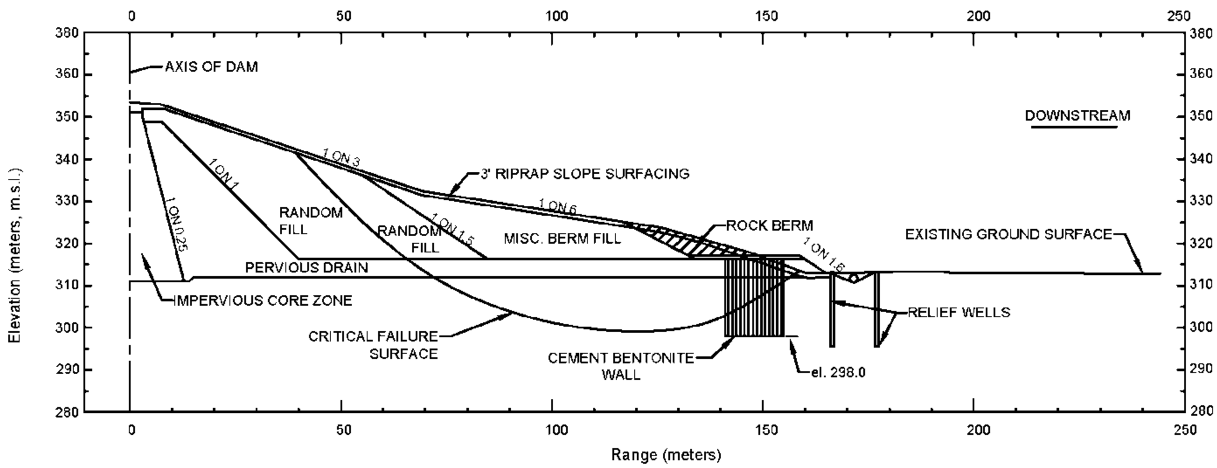

Figure 8. Cross section showing cement-bentonite transverse shear wall and critical limit equilibrium failure surface

SUMMARY

This paper provides an overview of the recently completed seismic retrofit of Tuttle Creek Dam near Manhattan, Kansas. This case history provides the following conclusions and 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., Environmental Impact Statement (EIS), should provide flexibility to designers, contractors, and owners to change retrofit technology without having to revise the EIS.
- The use of compressed air assisted jet grouting on operating dams can pose dam safety issues that cannot be easily monitored or controlled and should be carefully evaluated before implementing.
- FLAC permanent deformation analyses using site specific soil constitutive models for the foundation sands and fine-grained blanket led to considerable reduction in the amount of remediation and a large cost savings.

- Transverse shear walls constructed using slurry trench techniques and self-hardening C-B slurry appear to be a viable slope stabilization technique for liquefiable foundation material if underseepage flows are not inhibited.

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