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Seismic Evaluation, Soil-Cement Mix Wall and Outlet Pipe Installation, Ivins Bench Dam, Utah

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SEISMIC EVALUATION, SOIL-CEMENT MIX WALL AND OUTLET PIPE INSTALLATION, IVINS BENCH DAM, UTAH

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ABSTRACT

An inspection of Ivins Bench dam and reservoir was performed following a 5.8 magnitude (M_w) earthquake in southern Utah. Sand was observed flowing from the drainage collection system and the Utah Division of Dam Safety requested that the reservoir be drained and the embankment repaired. Dames & Moore evaluated the liquefaction potential of the embankment and concluded that the embankment was not suitable for maximum storage.

A repair concept using a soil-cement mix wall was developed and a dynamic stability analysis was performed using the computer program FLAC. The dynamic analysis indicated that the soil-cement wall would improve embankment stability and significantly lower the phreatic surface downstream of the wall, thus reducing the liquefaction potential.

One hundred and fourteen (114) soil-cement panels (6.83-feet wide and 34-inches thick) ranging from 18 to 50 feet in depth were installed parallel to the upstream crest of the embankment. The panels penetrated 3-feet into the bedrock underlying the embankment. Subsequent to construction, a 36-inch diameter outlet pipe was installed through both the embankment and soil-cement mix wall by microtunneling methods. A new intake structure was also constructed. Construction began in January, 1996 and the reservoir was filled during spring 1997 runoff.

KEYWORDS

Dynamic stability, liquefaction, remedial design, soil-cement wall, design and construction.

INTRODUCTION

Ivins Bench dam was constructed in the early 1900's. Rock buttresses were hand-placed to form the upstream and downstream toes of the embankment. Silty sand was reportedly sluiced in between the buttresses to form the embankment core. The buttresses and core were constructed on shale, sandstone, and claystone bedrock. No grout curtain or cutoff trench was installed. Filter zones were not constructed between the embankment core and the rock buttresses.

Modifications consisting of placement of a seepage blanket on portions of the upstream slope face were completed in 1943. Approximately 10 feet of silty clay was added to the embankment crest at that time. In 1987 a filter zone with PVC drains was constructed at the toe of the downstream rock buttress. Silty sand was placed downstream of the filter zone to further buttress the embankment and to shape the downstream slope. A typical cross- section of the embankment is shown on Fig. 1.

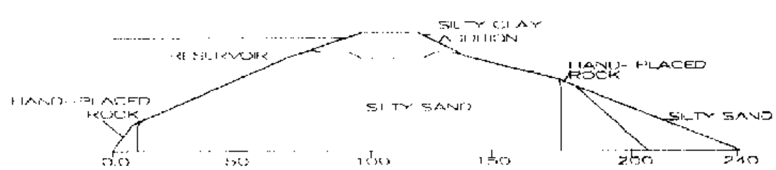

Fig. 1 Typical Embankment Cross-Section

Ivins Bench dam has a maximum height of 40 to 45 feet and has a crest width of 15 to 20 feet. The downstream slope of the embankment ranges from 2.75 to 3.0 horizontal to 1.0 vertical. The upstream embankment slope is 2.5 horizontal to 1.0 vertical. Boulder- sized riprap blankets much of the upstream slope.

Following an M_w 5.8 earthquake on September 2, 1992 in Washington County, Utah, an inspection of Ivins Bench dam was performed by personnel from Utah's Department of Natural Resources, Division of Water Rights (DWR). Sand was observed flowing out of the drainage collection system at the toe of the downstream embankment slope and the DWR requested that the reservoir be drained and repair options evaluated.

Dames & Moore was retained to evaluate the liquefaction potential of the embankment. Stability and liquefaction analyses indicated that the embankment was not suitable for maximum storage, however, a reduced pool could safely be contained on a temporary basis. Several repair options were evaluated including embankment replacement, addition of a downstream buttress, and inclusion of a soil-cement mix wall parallel to the upstream crest of the embankment. Based on cost/benefit analyses, improved stability, and a potentially significant lowering of the phreatic surface in the downstream slope, the soil-cement mix wall option was approved.

SEEPAGE ANALYSIS

A seepage analysis was performed to compare seepage of the embankment with and without a soil-cement mix wall. Permeability values used in the analysis were based on laboratory tests and correlations with grain size and/or material properties. Permeability values used are presented in Table 1.

Table 1 Permeability Values

Material Type	Horizontal Permeability (cm/sec)	Vertical Permeability (cm/sec)
Silty Sand (SM)	1.8×10^{-4}	8.8×10^{-4}
Silty Clay Addition	1.0×10^{-6}	1.0×10^{-6}
Hand Placed Rock	1.0×10^{-2}	1.0×10^{-2}
Soil-Cement Wall	1.0×10^{-6}	1.0×10^{-6}

Figure 2 presents the computed phreatic surface of the original embankment. A potential seepage quantity of 18.6 cubic feet per day (0.096 gpm) per lineal foot of embankment was calculated. Figure 3 presents the computed phreatic surface with a soil-cement mix wall. A potential seepage quantity of 0.55 cubic feet per day (0.003 gpm) per lineal foot of embankment was computed.

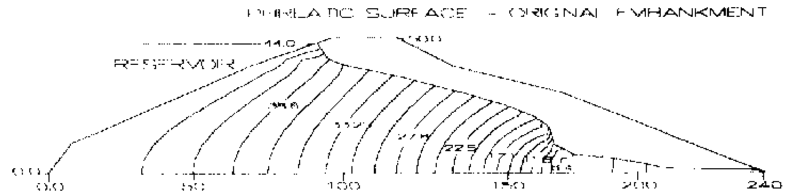

Fig. 2 Phreatic Surface of Original Embankment

Fig. 3 Phreatic Surface with Soil-Cement Mix Wall

DYNAMIC STABILITY ANALYSIS

A dynamic stability analysis was performed which simulated the embankment with the proposed soil-cement mix wall. The analysis was used to assess the susceptibility to liquefaction and to evaluate potential embankment deformation if subjected to strong ground motion. A complete description of the dynamic stability analysis is beyond the scope of this paper, however, the following steps were included in the analysis.

1. Critical seismic sources were identified and response spectra and acceleration time histories were developed. Two sources were identified: (1) a magnitude (M_w) 7.25 Washington Fault centered 12 miles away with a peak horizontal acceleration of 0.33g and (2) a magnitude (M_w) 6.5 random event 5 miles away with a peak horizontal acceleration of 0.44g.

2. Static and dynamic properties of the embankment and foundation were estimated based on laboratory test results and published correlations. Properties included unit weight, shear and bulk modulus, static and cyclic shear strengths, and damping coefficient.

3. Dynamic and liquefaction analyses were performed using the nonlinear, explicit, finite difference program FLAC. The FLAC analysis simultaneously integrates the effects of real-time pore pressure buildup, shaking-induced deformation and post-earthquake stability.

Figure 4 shows the predicted time-dependent horizontal and vertical settlement at the dam crest center from the random earthquake event. Figure 5 shows the absolute displacement vectors and deformed shape of the embankment at 25 seconds after shaking begins. A maximum displacement of 4.29 feet within the lower third of the upstream slope was indicated.

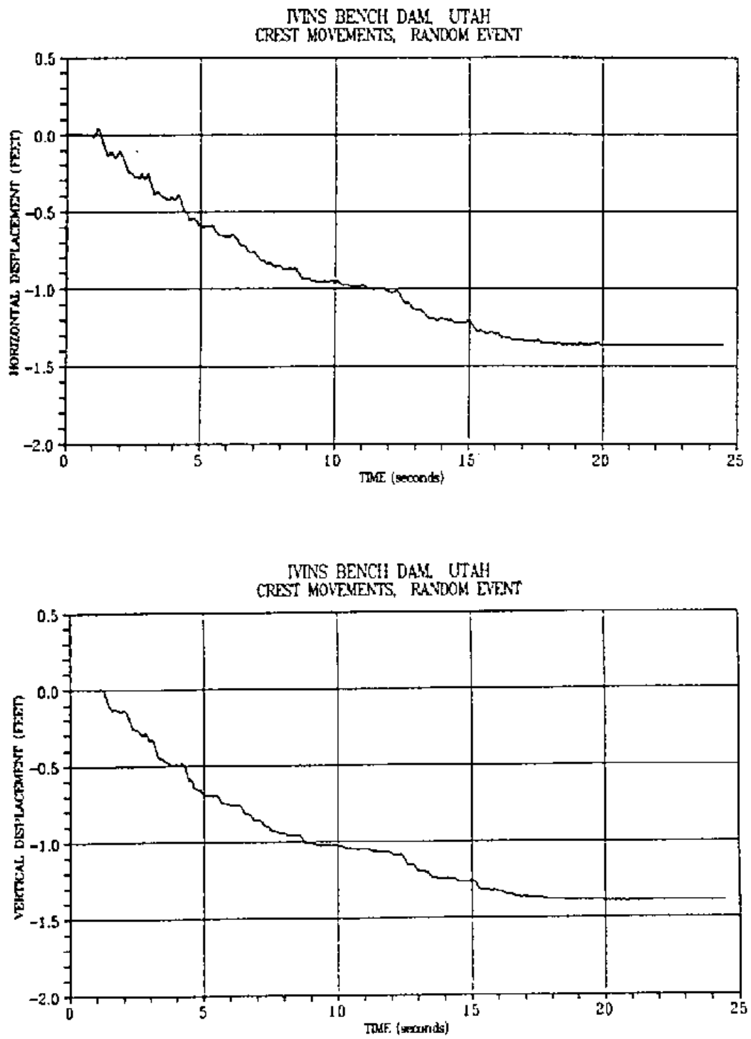

Fig. 4 Crest Movements From Random Event

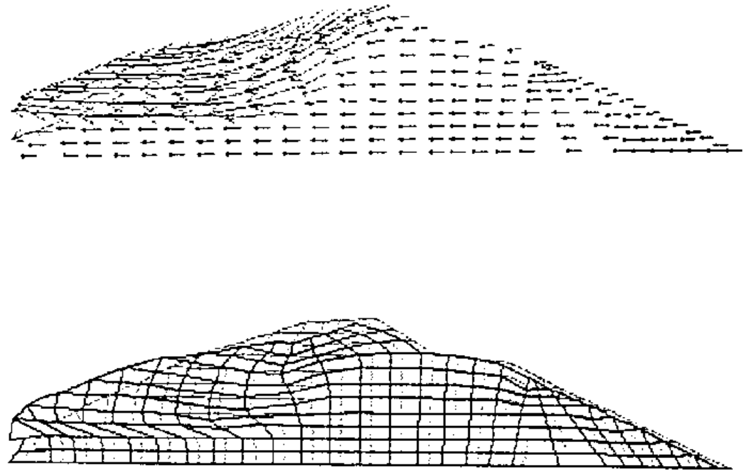

Fig. 5 Displacement Vectors and Deformed Shape, Random Event

Conclusions from the dynamic stability analysis of a soil-cement mix wall included:

1. Earthquake-induced deformations in the downstream portionof the embankment would be small due to reduced seepage through the soil-cement mix wall. The dam would maintain its capacity to safely impound the reservoir.

2. Calculated settlement at the center of the crest would be less than 1.5 feet for the random event and less than 1.0 foot for the Washington Fault event.

3. Significant earthquake-induced excess pore pressures would develop in the saturated sands in the upstream portion of the embankment. In the absence of riprap confinement, the loss of strength resulting from the development of high excess pore pressure could cause flow failure of the upstream section of the embankment into the reservoir.

4. The rockfill toe and riprap layer on the upstream portion of the dam play an essential role in preventing failure of the upstream section of the dam under earthquake loading conditions.

SOIL-CEMENT MIX WALL CONSTRUCTION

Based on the results of stability analyses, an unconfined compressive strength of 350 to 400 pounds per square inch (psi) was targeted for the soil-cement mix wall. Horizontal and vertical permeabilities of approximately 1.0×10^{-6} cm/sec were also required. Because seepage cutoff and stabilization of the silty sand core were crucial to overall stability, mixes for the soil-cement wall were designed to be compatible with the silty sand rather than the upper clay addition.

Laboratory tests on various cement-to-water and cement-to-soil ratios indicated that a cement-to-water ratio of 67 percent cement and a cement-to-soil ratio of 15 to 20 percent cement provided unconfined compressive strengths within the desired range. Based on the results of laboratory testing, a cement-to-soil ratio of 20 percent cement was specified to provide some degree of conservatism for actual field conditions. Two percent bentonite was added to the mix to decrease wall permeability.

Falling head permeability tests were conducted on the specified mix design. Test results indicated that a vertical permeability of 2.43×10^{7} cm/sec could be achieved which would meet the 1.0×10^{-6} cm/sec requirement.

Dames & Moore prepared specifications and a bid package for construction of the soil-cement mix wall. SMW Seiko was awarded the construction contract. Wall construction was initiated on February 17, 1996 and was completed on February 28, 1996.

Wall Construction Equipment

Construction of the soil-cement wall was accomplished using an SMW DH608-120M M90D drill rig. Soil cement columns were formed using three augers guided by a set of leads that were mounted on a crawler. As the augers advanced into the soil, grout slurry was pumped through the hollow stems of the outside auger shafts and was injected into the soil at the auger tips. The auger shafts were equipped with mixing blades and paddles that worked in concert with the auger flights to blend the soil and grout slurry into a relatively homogeneous soil-cement mixture. The mixing blades and paddles were positioned to overlap one another, thus continuously overlapping columns were formed. The three overlapping columns constituted a single soil-cement panel.

When the design depth for each panel was reached, the augers were withdrawn and the mixing process was repeated until the augers reached the ground surface. The center auger rotated opposite of the outer two augers to facilitate mixing. Rotation speed of the augers was frequently adjusted to allow for a constant rate of mixing during penetration based on the degree of drilling difficulty. In order to ensure adequate mixing, the penetration rate of the augers was maintained in the range of 1 to 8 vertical feet per minute during both penetration and withdrawal.

A shallow guide trench was excavated on the centerline of the soil-cement mix wall to define the location of installation and to collect mixing spoils (cuttings). Horizontal alignment of the panels was maintained by marking each panel section at the ground surface. The distance between the centerline of the outside augers was 48-inches. The augers were 34-inches in diameter, thus, an 82-inch (6.83 feet) wide panel was formed during each insertion. After panel completion, the crawler advanced 60-inches resulting in a 22-inch overlap with the previous panel.

Verticality of the augers was maintained by the equipment operator through electronic sensors built into the leads that monitored fore-aft and left-right vertical positions. The verticality of the soil-cement mix wall at Ivins Bench was maintained within 1-foot horizontal to 80-feet vertical (1.25%).

The outside augers penetrated a minimum of 6 feet into bedrock. The middle auger was 3-feet shorter than the outside augers and penetrated 3-feet into bedrock. Thus, the minimum penetration of each panel into bedrock was 3 feet. The soil/bedrock contact was delineated by monitoring auger motor amperage and auger chatter and from correlations with previous investigations. The depth to bedrock was ascertained when a significant increase in amperage was noted.

The mixing plant for wall installation consisted of a grout mixer, grout agitator, grout pump, batching scales, and control unit. The cement was stored in a silo adjacent to the plant. The mixing plant mixed the cement-water in a predefined ratio determined by weight using automatic batching scales in the mixing plant. The bentonite was delivered to the water-cement mix by a calibrated screw feeder. Quality control of the mix design was monitored by on-site specific gravity, unit weight, and marsh funnel tests on selected samples.

The grout injection rate per vertical foot of panel was monitored by both the mixing plant and drill rig operator. Positive displacement pumps were used to transfer the grout from the mixing plant to the augers.

Wall Installation

Prior to construction of the soil-cement mix wall, the depth to bedrock was determined by drilling at approximately 100-foot intervals along the embankment crest with a solid flight auger.

Thus, a target depth of the wall was known prior to construction. The final wall depth was determined when 3- feet of penetration into bedrock was achieved.

To accommodate the width of the drill rig, the wall was constructed through the upstream crest of the embankment, rather than through the embankment centerline. A track-mounted backhoe was used to excavate a guide trench approximately 3-feet wide and 2-feet deep. Slurry overflow from the trench was allowed to permeate into the upstream riprap facing.

One hundred and fourteen (114) soil-cement mix panels were installed along the upstream crest of the embankment. Panel depths ranged from 18 to 50 feet. A total of 15,865 square feet of 34-inch wide soil-cement mix wall was installed.

The original outlet pipe through the embankment was severed during wall installation. When the augers punctured the outlet pipe, the downstream valve on the outlet pipe was only partially closed. Several cubic yards of soil-cement slurry flowed out of the pipe before the valve could be closed completely. Some of the soil-cement slurry was lost from adjacent panels. After the outlet valve was closed, soil-cement slurry was noted at the top of the intake box on the upstream side of the embankment. Upon stabilization of the panel which penetrated the outlet pipe, the adjacent panels were re-drilled to ensure wall continuity. Thus, both the upstream and downstream portions of the original outlet pipe were filled with slurry.

SOIL-CEMENT MIX TESTING

The slurry mix was continuously monitored at the mixing plant by regulating mix weights and specific gravity. Specifications required that one bulk sample of soil-cement from every 5,000 square feet of wall be retrieved for strength testing. Soil cement samples were collected by lowering a 6-inch wide I-beam with an attached sample collection cylinder into the soil-cement slurry. In-situ slurry samples were retrieved from panels 42, 48, 58, 79, 84, 92, and 96. Samples were divided and placed into 2.5-inch by 6-inch plastic canisters and sent to Dames & Moore's Salt Lake City laboratory for testing. Select samples were also sent to SMW Seiko's Hayward, California laboratory for test verification of test results.

The unconfined compressive strength test results from collected samples are graphed on Fig. 6. The linear trend line is included on Fig. 6. Unconfined compressive strengths generally averaged between 350 and 400 psi for samples tested with a maximum value of 637 psi and a minimum value of 290 psi.

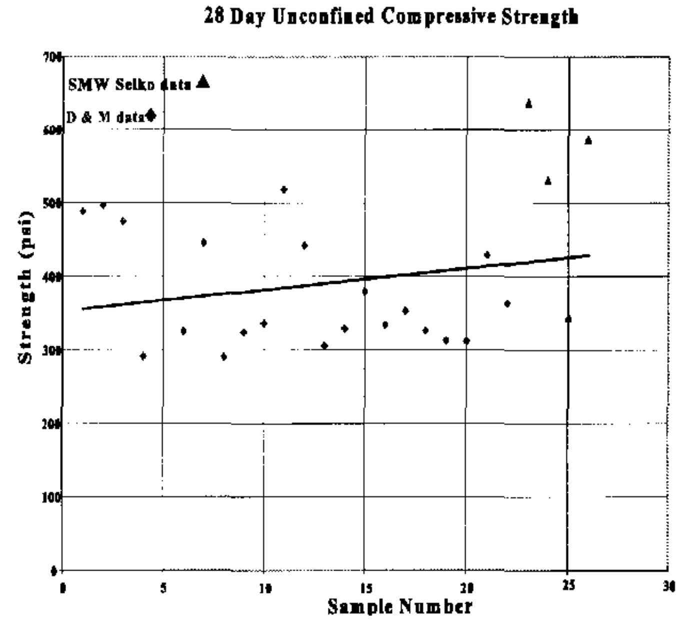

Fig. 6 28 Day Break Unconfined Compressive Strength Test Results

NEW OUTLET PIPE INSTALLATION

During construction of the soil-cement mix wall, the original 12-inch diameter outlet pipe for the dam was severed and filled with soil-cement slurry in anticipation of installation of a new 36-inch diameter pipe. Dames & Moore contracted with Willco Far West for pipe installation and Willco mobilized to the site on March 18, 1996. Pipe installation began on March 27, 1996 and was completed on April 12, 1996. The pipe passed pressure testing on April 17, 1996.

The new steel outlet pipe was 36-inches in diameter with a wall thickness of 0.312-inches. The pipe was coated on the outside with bituminous asphalt prior to installation.

A pit for the boring machine was excavated at the downstream toe of the embankment to allow for pipe installation and removal of excavated soil. To facilitate installation of 40-foot long sections of pipe, the pit was excavated to approximately 55 feet in length. The pit backstop, against which the boring machine pushed, consisted of sandstone bedrock.

The pipe was installed using an Allied 36-inch boring machine. The boring equipment included a boring machine, guide tracks, augers, and cutting head. An articulated section was installed on the initial section of pipe to control pipe orientation. The cutting head extended approximately 6-inches in front of the pipe and was attached to continuous flight augers extending from the cutting head to the boring machine. The auger diameters tapered from 36-inches at the pipe head to 24-inches at the boring machine.

The pipe was advanced by rotating the cutting head to remove embankment material several inches in front of the pipe while simultaneously pushing the pipe forward. Thrust was provided by extensions on the boring machine that were inserted into cut-away sections of the guide track. The guide track was held in place by the weight of the pipe and boring machine and bedrock backstop.

Vertical positioning of the pipe was accomplished by means of an articulating head that could be directed up or down to control the grade of the pipe. The grade of the pipe was determined by means of a water filled tube on top on the pipe. By measuring the head in the tube and knowing the pipe length, the average grade of the pipe could be determined. A 2-percent grade was maintained for the pipe.

The horizontal alignment of the pipe could not be adjusted automatically during drilling. However, since the augers rotate to the right, past experience indicated that pipes also tend to drift to the right. The pipe drift was compensated for by applying a shim on the right side of the pipe tapering towards the left. The shim tended to push the pipe towards the left and compensate for the rightward drift. After installation, horizontal drift of the pipe was measured at less than 2 inches.

The pipe was installed in 40-foot sections. Upon pushing a section to the end of the boring pit, the boring machine was backed to the opposite end of the pit and a subsequent section of pipe was added. All joint welds were performed by certified welders. Concurrent with pipe installation, 2-inch diameter holes were cut into the pipe (top and on each side) at approximately 20-foot centers and 2-inch caps were threaded into the holes.

After approximately 160 feet of pipe were installed, side friction between the pipe and the silty sand embankment increased beyond the thrust capabilities of the boring machine and the pipe could not be advanced. A second boring machine was brought to the site and installed in tandem with the first machine. The tandem boring machines pushed the pipe to 190 feet where the upstream receiving pit excavation was encountered.

After breaking through on the upstream side, operations were halted for approximately three days with plans to push an additional 20-feet of pipe through the embankment to achieve the desired length. However, after the three day hiatus, the pipe was so tightly bound that attempts to push with the tandem boring machines caused the backstop to fail. To rectify the problem, the upstream receiving pit was enlarged, the articulating head was removed from the pipe, and 20-feet of pipe were welded onto the upstream end.

Outlet Pipe Grouting

Following pipe installation, grout ports were installed in the 2-inch diameter holes and grout was pumped with a concrete pump in order to fill the annular space between the pipe and excavation wall and to create water stops along the pipe. Due to the large voids encountered in the hand-placed rock and the compressible nature of the embankment material, it was decided that an excessive volume of grout could be pumped into the embankment without significant pressure buildup. Therefore, a specified volume of grout was pumped into the ports to fill the calculated annular space. Pumping into several ports was stopped early due to excessive pressure buildup.

After completion of grouting, the grout ports were removed and threaded caps were inserted into the threaded ports. The threaded caps were then welded and sealed.

Outlet Pipe Backfilling

A downstream filter was constructed around the pipe by encasing 10-feet of the pipe in 2-feet of gravel and encasing the gravel with 2-feet of cement sand. Silty sand fill was then placed over the filter to allow for a minimum cover of 3-feet over the pipe.

SUMMARY

One hundred and fourteen (114) soil-cement panels were installed along the upstream crest of the embankment. Panel depths ranged from 18 to 50 feet. Unconfined compressive strengths of samples tested from the wall ranged between 290 and 637 psi.

Upon completion of the soil-cement mix wall, 210-feet of 36-inch diameter steel pipe was installed through both the embankment and the wall. The project was completed on time and within budget and the reservoir was filled during spring 1997 runoff.