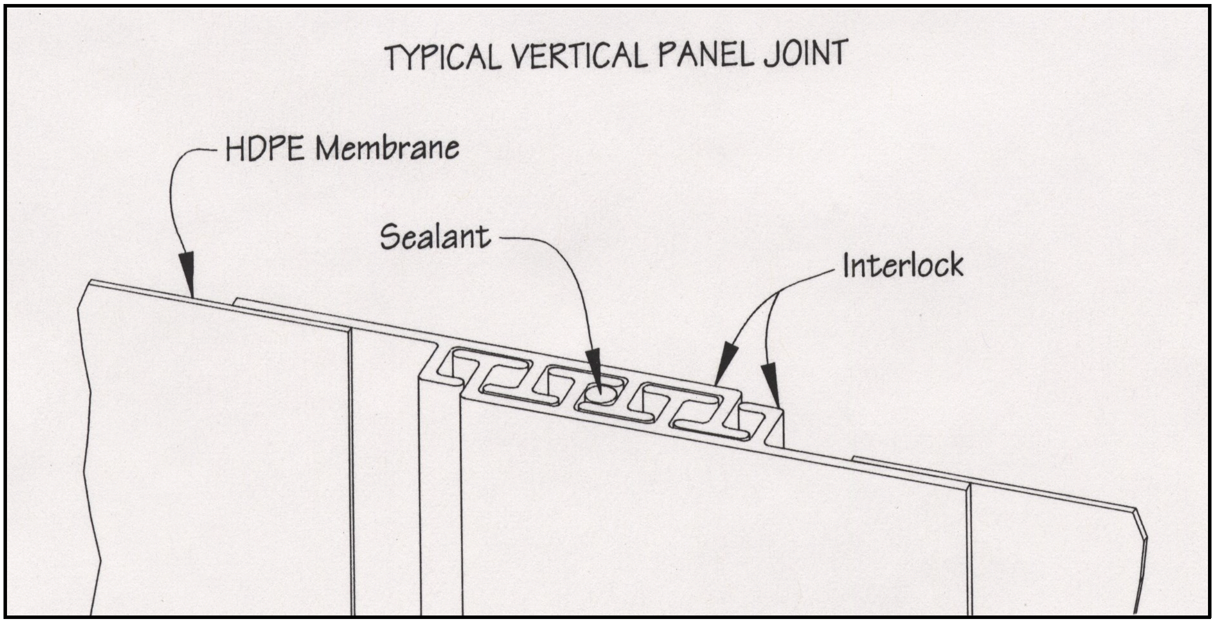
WATER OPERATION AND MAINTENANCE BULLETIN

No. 186 December 1998



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UNITED STATES DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

This *Water Operation and Maintenance Bulletin* is published quarterly for the benefit of water supply system operators. Its principal purpose is to serve as a medium to exchange information for use by Reclamation personnel and water user groups in operating and maintaining project facilities.

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Cover photograph: *Placement of sealant in interlocked joint.*

UNITED STATES DEPARTMENT OF THE INTERIOR **Bureau of Reclamation**

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LIQUEFACTION MITIGATION OF A SILTY DAM FOUNDATION USING VIBRO-STONE COLUMNS AND DRAINAGE WICKS—A TEST SECTION CASE HISTORY AT SALMON LAKE DAM

by Ron Luehring[1], Bob Dewey[1], Lelio Mejia[2], Mike Stevens[3], and Juan Baez[4]

Abstract

The use of stone columns in combination with drainage wicks is shown to effectively mitigate the potential for liquefaction of non-plastic silty soils. To achieve acceptable foundation treatment, proper implementation of stone column construction methods, equipment, and sequencing are essential. This paper presents the results of a test section using dry, bottom-feed vibro-stone column construction in up to 70 feet of silt-interbedded fluvial-lacustrine sandy foundation materials beneath Salmon Lake Dam. Standard Penetration Tests (SPTs) and Cone Penetrometer Tests (CPTs) were used for site characterization before and after stone column construction. Liquefaction potential was determined by comparing measured values of penetration resistance to values required to resist liquefaction under the maximum credible earthquake. State-of-the-practice data conversions were used to perform the liquefaction analysis on the basis of clean sand equivalent blowcounts. The test section results show that: (1) drainage (air and pore pressure relief) is provided by stone columns and drainage wicks during construction, (2) foundation treatment meeting Bureau of Reclamation (Reclamation) design objectives is achieved by soil densification between the columns, and (3) liquefaction can be mitigated by stone column treatment by measurable density increases even in fine-grained silty soils. Key discussion is provided based on observations related to column spacings, diameters, and sequencing of construction. Test section results presented in this paper form the design basis for liquefaction mitigation of the entire downstream foundation.

Background

Salmon Lake Dam is situated on a tributary of Salmon Creek about 15 miles northwest of Okanogan in north-central Washington. Completed in 1921, it consists of a 30-foot-high zoned earthfill embankment with a crest length of 1,260 feet and a combined spillway/outlet works structure. A cross section of the existing embankment and planned dam safety modifications is provided on Figure 1.

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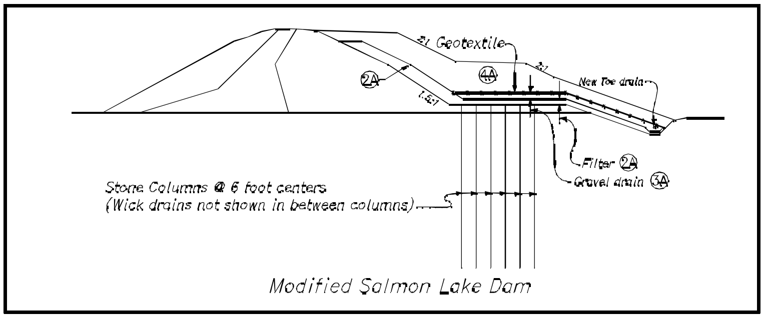

Figure 1

The dam foundation consists of Quaternary fluvio-lacustrine sediments under most of the embankment. These sediments are generally cohesionless, interbedded to laminated silty sand, with interbeds and lenses of silt with sand, sandy silt, poorly graded sand, and silty sand with gravel. Minor deposits of silt, and silty gravel, organics, and volcanic ash were encountered during geotechnical explorations.

Analysis of the earthquake catalog led to the determination of a maximum credible earth-quake (MCE) of M_L 6.5 for a random event at a distance of 29 kilometers [1]. The maximum peak horizontal acceleration for this source was estimated to be 0.26 g [2]. This MCE has sufficient energy to produce significant shear strength loss in foundation layers susceptible to liquefaction.

Vibro-stone column technology had proven successful for Reclamation and the U.S. Army Corps of Engineers dam safety modifications to Mormon Island Auxiliary Dam [3, 4]. Although estimates indicated this technique to be the most cost-effective structural mitigation at Salmon Lake, it was unknown if the presence of a high percentage of low-plasticity fines and/or apparently dense gravels would hamper the construction and/or the effectiveness of the stone columns. Reclamation experience for dam safety modifications to Jackson Lake and Steinaker Dams [5], where wick drains were used in combination with dynamic compaction, indicated ground improvements in fine-grained silty soils could be enhanced by installation of drainage wicks prior to performing dynamic compaction. However, it was unknown whether the vibro-stone column process could also be enhanced by installation of drainage wicks. A test section was therefore constructed to investigate whether stone columns and drainage wicks could effectively mitigate liquefaction potential. Data were collected as necessary to compare foundation strengths before and after ground improvement to optimize final dam safety designs.

Test Section Design

Testing Locations

Test site locations were determined based on existing site explorations (SPT, Becker Hammer Penetration Tests, and cross-hole geophysics). Two test sites were investigated at the down-stream toe representing a range of foundation conditions. Site C, located at about dam

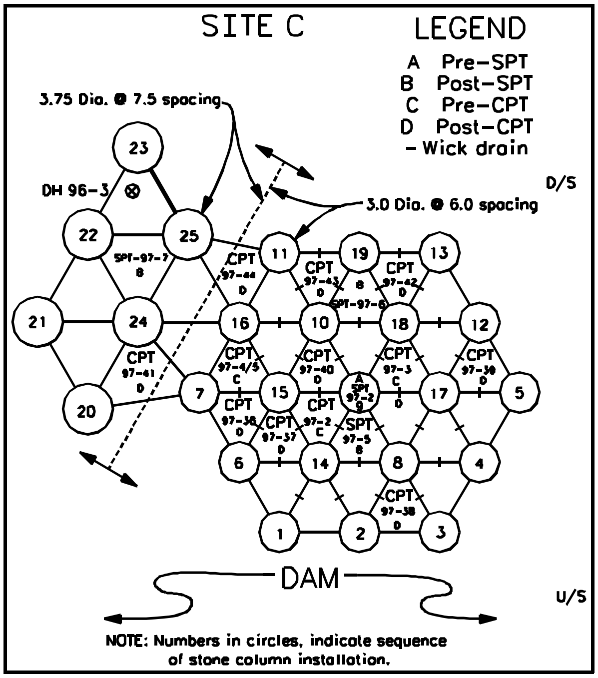

Figure 2

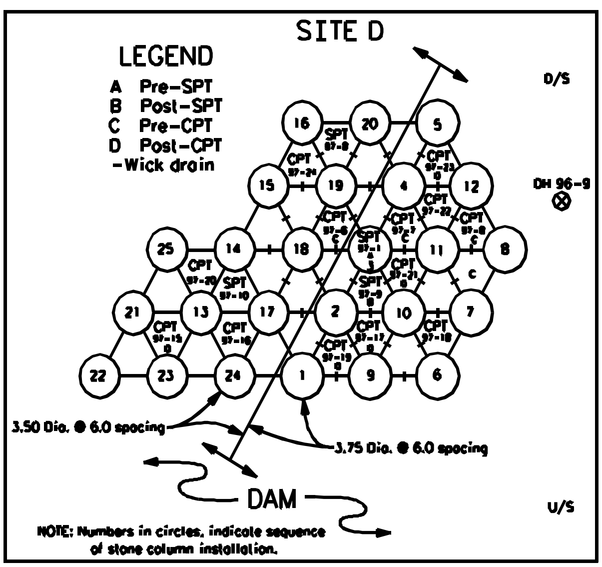

Figure 3

station 10+50, was selected due to the generally higher fines content of the foundation materials. Site D, located at about dam station 4+95, was selected because it appeared to contain the greatest concentration of materials considered most likely to liquefy.

Drainage Wicks

Thirty-two wick drains were installed at each test site prior to the construction of any stone columns to allow dissipation of air and pore pressures that are inherent with the dry, bottom feed method of installing stone columns. The wick drains were located equidistant between the planned locations of the stone columns extending to the full design column depths (Figures 2 and 3). The number of wick drains surrounding any stone column varied as shown.

Stone Column Layout and Depths

Combinations of column diameters (3.0, 3.5, 3.75 feet) and spacings (6.0 and 7.5 feet) were constructed and investigated to optimize final designs. Figures 2 and 3 illustrate various combinations of spacing, diameter, and sequencing. Note that Site C used predominantly 3.0-foot-diameter columns, while Site D used larger diameter columns. The upstream three rows were advanced to depths of 55 feet, and the downstream two rows were advanced to depths of 68 to 70 feet.

Methodology for Evaluation of Liquefaction Potential

Over the past five years, the foundation/embankment explorations of Salmon Lake Dam has progressed from a general geologic and materials investigation to a specific site characterization geared towards qualitatively evaluating liquefaction triggering.

According to Seed [6], "there appears to be a strong consensus that *in situ* testing methods currently considered sufficiently 'mature' (well-documented, well-calibrated, and verified) to serve as a useful engineering basis for evaluation of resistance to "triggering" of liquefaction are: (1) the Standard Penetration Test (SPT), (2) the Cone Penetrometer Test (CPT), (3) the Becker Penetration Test (BPT), and (4) shear wave velocity measurements (v_s)." Since the SPT and CPT are considered reliable for most sandy and low-plasticity silty soils, both were selected to be used for site characterization prior to and after drainage wick installation and stone column construction.

Liquefaction Evaluation Based on SPT Data

The liquefaction potential evaluation data were analyzed to compare the foundation material's measured resistance to liquefaction (represented by $N_{1(60)m,(cs)}$) to values required (represented by $N_{1(60)r,(cs)}$) to resist liquefaction under the MCE. The comparison was made on a clean sand basis. Since the foundation has a fair percentage of materials with fines (minus No. 200), a fines correction, $\Delta N_{1(60)}$ is required and applied to the measured values. Previously developed state-of-the-practice methodologies were employed which relate cyclic shear stress to required corrected blowcounts for clean sands [6].

CPT Methodology for Liquefaction Analysis

In order to evaluate liquefaction triggering, one must compare a measured penetration resistance (normalized and corrected to a clean sand equivalent, $q_{c1N,m(CS)}$) to a required value ($q_{c1N,r(cs)}$).

If: $q_{c1N,m(cs)} \succ q_{c1N,r(cs)}$ Then, no liquefaction

$q_{c1N,m(cs)} \prec q_{c1N,r(cs)}$ Then, potential for liquefaction

According to Robertson and Wride [7], it is possible to correct the measured CPT penetration resistance to an equivalent clean sand value by estimating grain characteristics (apparent fines content) directly from the CPT. However, it is noted that estimates of the fines content from the CPT can be unreliable in some cases. As discussed below, the apparent fines content obtained from the CPT generally underestimates the laboratory measured fines content at this site.

The CPT fines correction was computed using two different methods to derive fines contents (i.e., fines contents from laboratory tests on adjacent SPT samples and "apparent fines" contents from theoretical equations using the CPT friction ratio). When using the correlations proposed by Robertson and Wride, a decision was made to defer to the "apparent fines" derivation using the CPT friction ratio so that consistency with the liquefaction analysis method proposed would be maintained throughout the entire application of the method. The measured fines content is preferred when using CPT correlations by others.

As discussed below, the conclusions for liquefaction potential for this site are the same regardless of which of the above two approaches is used. This indicates that the Robertson and Wride fines content correction has compensated for the lower apparent fines content. The results obtained using the apparent fines content are used below to illustrate the foundation improvement. It was recognized that using measured fines contents together with Robertson and Wride's Δq_{c1} resulted in a correction which was too large.

Stone Column Construction

Equipment

Stone columns were constructed using the dry Vibro-Replacement Bottom-Feed Method. The main component of the system was a 165-horsepower electrical vibrator. A crawler crane was used to suspend the vibrator and a fixed lead system. The guide system of fixed leads, which aids in ensuring verticality, supports a skip bucket for the delivery of stone to the supply tube. Heavy extension tubes attached to the vibrator allow treatment to reach the specified depth. A rubber tired loader was used to feed gravel to the skip bucket, which in turn fed the gravel to a pressure chamber mounted on top of the vibrator. The loader was used to maintain the working surface reasonably free of mud and turbid water generated by the process and to fill any craters with suitable backfill materials to maintain the elevation of the working surface.

Procedure

At each stone column location, the vibrator penetrated the foundation soils to the specified design depths by a combination of weight, vibration, and air. No significant difficulties were encountered (i.e., pre-augering and/or jetting water were not required to aid in penetration). After driving to full depth, the vibrator was retrieved in 2- to 4-foot lifts to allow stone backfill placement. The vibrator was re-driven through each backfill increment until stone column diameters reached the dimension shown on Figures 2 and 3. This process was repeated until the working surface was reached. The completed stone columns and the *in situ* soil formed an integrated system having low compressibility and high shear strength.

Construction Monitoring and Quality Control

A load cell mounted on the loader bucket measured the amount of stone, in pounds, being placed within each skip. A depth encoder mounted on the crane established depth at any given moment. Stone consumption, in tons per foot, is thus obtained by dividing the input of the load cell by the difference between the initial depth and final depth for that lift. Depth, vibrator power consumption, weight of backfill per skip, and air pressure were continuously monitored and recorded for later analysis.

A pressure gage visible to the crane operator helped signal any plugging of the stone feed pipe system to prevent possible blowouts and hydraulic fracturing of the embankment foundation.

General Observations

During construction at Site C, between one and three cubic yards of waste was observed at the surface. This waste began to exit and accumulate adjacent to the probe as it was being withdrawn at depths less than about 22 feet. The waste was a mixture of *in situ* materials (sands and silts) and minus 1-inch crushed gravel used to construct the columns. This is the result of vibro-replacement of the materials that may have contained higher fines contents (estimated greater than 60 percent passing the No. 200 sieve). During the densification effort, the materials were squeezed up the path of least resistance which was along the probe and follower tubes. Measured increases in density (penetration resistance) were less in these materials and at these depths.

It is estimated that as much as five to six cubic yards of waste materials were displaced onto the working surface at Site D. Craters and caving developed at the ground surface while constructing columns D1 through D5. In several instances, wick drains were exposed within the stone columns, indicating the radius of the craters were at least 3 feet. Caving conditions became less prominent for construction of subsequent columns.

Air bubbles were observed on the water surface of a drainage ditch located downstream of the test sites. As a result, riser pipes were installed to piezometers located downstream of the test sites to measure effects of increased pressures at these locations. The piezometer riser at Site D did not raise above the ground surface during construction of columns D1 through D23 (Figure 3). However, during construction of D24 and D25, the water level in this riser rose to as high as 8 feet above the ground surface. Both of these stone columns were constructed to 68 feet in depth and did not have wick drains installed in their immediate vicinity prior to their installation.

Groundwater levels rose higher for construction performed at Site D as compared to Site C.

Behavior of Drainage Wicks

The drainage wicks began to emit water and air when penetration of the probe for stone column C1 had been advanced to a depth of 29 feet, and, for the most part, continued to vent water and air to the surface during construction of each of the subsequent stone columns. Drainage wicks at Site D produced considerably more water than was produced during construction at Site C (this phenomenon may be a function of a larger column diameter since the column spacing was the same). Several drainage wicks at Site D were noted to be "ejecting air/water up to 4 feet above ground surface" during construction of column D1. Since practically all drainage wicks were "blowing" air and water during the construction of the stone columns, some connectivity between the drainage wicks must have existed. Perhaps the most benefit provided by the drainage wicks was that they provided avenues for air pressures to be released during the stone column construction process and, in doing so, protected other areas of the foundation and embankment from disturbance or hydraulic fracturing. Installation of the drainage wicks is expected to provide short-term benefits of enhancing the vibro-stone column process (based on after treatment investigation results) and potential long-term benefits of improving foundation soils resistance to liquefaction.

Construction Testing Results

Test section site characterization prior to treatment for Site C included two (2) SPTs and four (4) CPTs and at Site D included three (3) SPTs and three (3) CPTs, as depicted on Figures 2 and 3. After-test-section explorations at Site C include three (3) SPTs and nine (9) CPTs. After-test-section explorations at Site D include three (3) SPTs and ten (10) CPTs. Site characterization after construction was conducted a minimum of two weeks after the construction of the last stone columns at each site in an effort to allow pore pressures to dissipate prior to testing.

Figures 4 and 5 compare the before- and after-Test-Section (TS) corrected[5] SPT blowcount data measured $N_{1(60)m,(cs)}$ against the required $N_{(60)r,(cs)}$ at Sites C and D, respectively. All measured $N_{1(60)m,(cs)}$ values that fall below the required $N_{1(60)r,(cs)}$ line indicate potential for triggering of liquefaction. The majority of before results (hollow shapes) lie near or below the required line for liquefaction for both sites, while after results, represented by solid shapes, show significantly higher blowcounts and indicate foundation improvement by measured densification increase. A closer examination of the data at Site C indicates that blowcounts from SPT DH97-5 (which lies within the interior or inner ring of test section Site C) remains entirely above the required line. This improvement may be attributed to: (1) maximum confinement from adjacent columns (inner ring), (2) optimal location in construction sequence, and (3) a maximum number of adjacent wick drains. Significantly better results are observed when comparing Site D inner ring data to Site C's inner ring data. This is attributed primarily to the much larger area replacement ratio (Ar) provided by installing 3.75-foot-diameter columns (an Ar of about 31 percent) to the 3.0-foot-diameter columns (an Ar of about 20 percent).

[5] All data were evaluated on the basis of clean sands (CS) [8]. N represents SPT blowcount in blows per foot, normalized to 1 ton per square foot (1) of overburden pressure and 60 percent (60) of the maximum theoretical energy. Measured (m) and required (R) values of the corrected blowcount are represented.

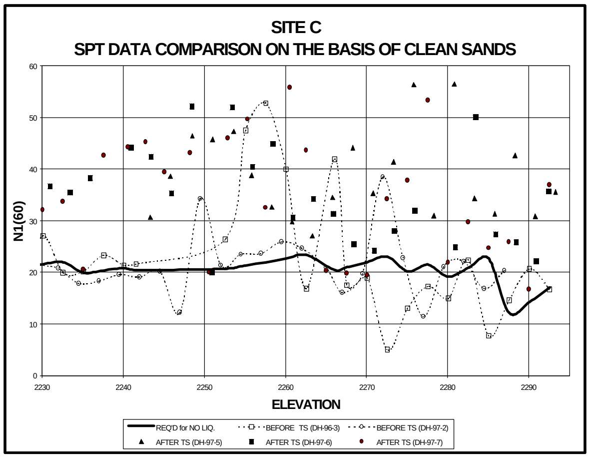

Figure 4

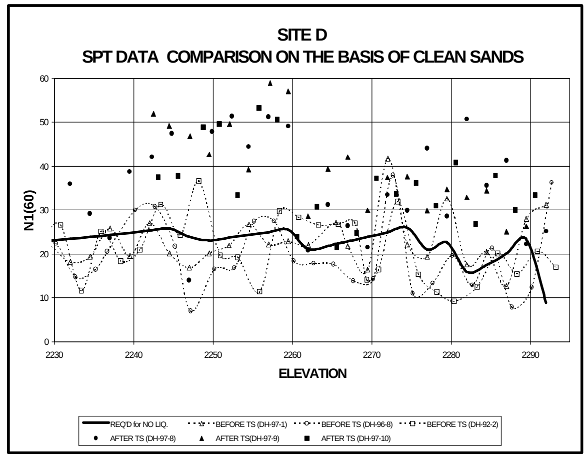

Figure 5

Comparing CPT data on Figures 6 and 7 indicates significant foundation improvement occurring (especially within the inner ring) at Site C. The inner ring for Site C consists of columns C8, C14, C15, C10, C18, C17. Fewer data points fall below the required $q_{c1Nr(cs)}$ line, which would suggest a lesser amount of liquefaction triggering. Although the stone columns penetrated a maximum of 55 feet compared to the 68 to 70 feet in the rows furthest downstream, the results indicate significant foundation treatment at Site C. At Site D, a comparison of Figures 8 and 9 indicates greater soil improvements for all stone columns (especially those with 3.75-foot diameters).

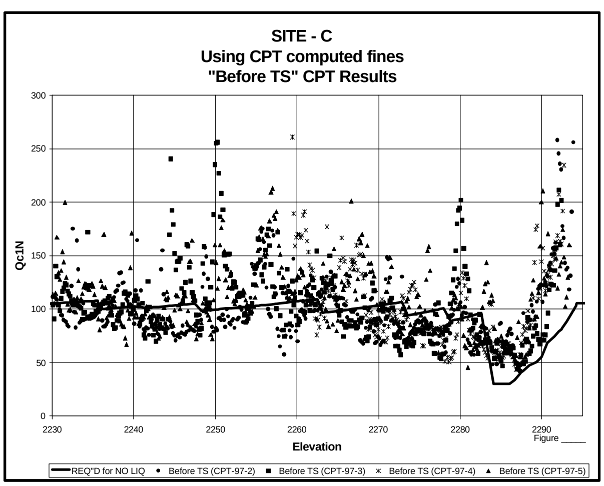

Figure 6

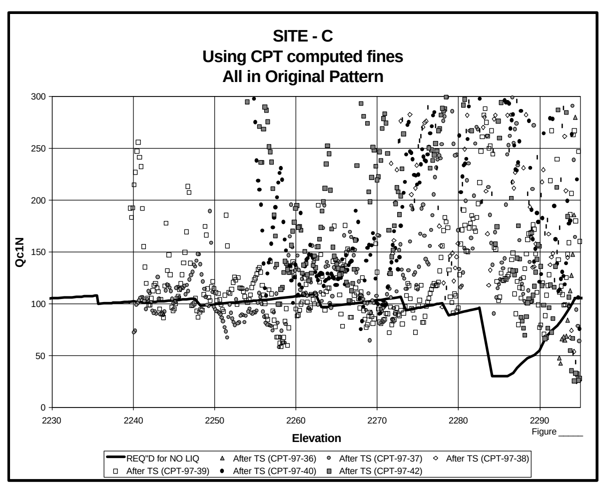

Figure 7

The 3.75-foot-diameter stone columns were constructed on 7.5-foot spacing at Site C (Figure 2) without the benefit of surrounding wick drains. Figure 4 compares the SPT DH97-7 measured $N_{1(60)m,(cs)}$ results to the required $N_{1(60)r,(cs)}$ line at Site C. In general, the 7.5-foot stone column spacing at Site C did not provide the level of foundation improvement as compared to the stone columns constructed on 6.0-foot centers. It is recognized that a small (insufficient) sample size was used to support this conclusion.

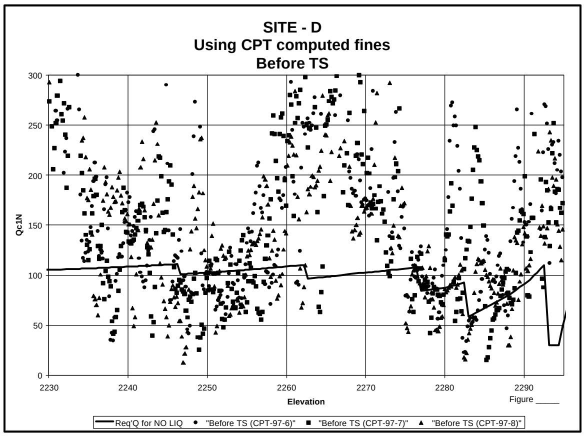

Figure 8

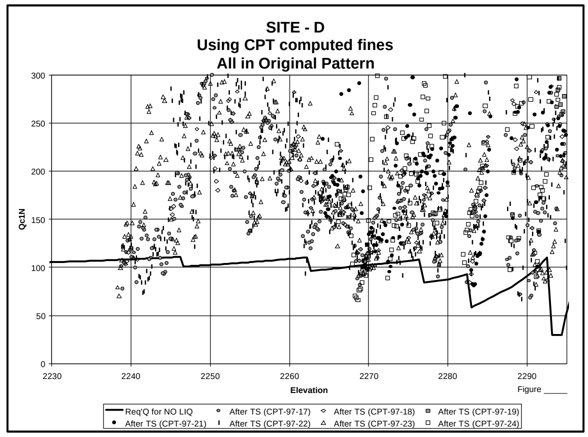

Figure 9

Sequencing Issues

The sequencing of stone column construction proposed for Site C considered maximizing the effectiveness of foundation treatment with depth and constructability. The numbers in the stone columns shown on Figures 2 and 3 are the order in which they were constructed. The stone column construction sequence used for Site C was modified for Site D by maximizing the effect of "closure" (i.e., outer stone columns completed before inner stone columns). Due to the somewhat random construction sequencing at Site C, amperage generation of the equipment (which may correlate to foundation improvement) was not significant until after the ninth column had been constructed. This lack of amperage generation is likely due to the minimal confinement or closure from adjacent stone columns. The sequence used at Site D (Figure 3) appeared to be more effective in treating the foundation than the sequence used at Site C. The work plan for this site minimized the movement/setup of the equipment and maximized stone column construction production and was considered nearly optimal. Generated amperage was generally higher in Site D than C and more consistent with depth. However, it should be noted that material types present at Site D were generally coarser in nature than at Site C, and this may be another possible explanation, other than constructed area replacement values, that may impact analyses pertaining to amperage levels.

Analysis of SPT Measured Fines vs. CPT Apparent Fines Contents

One important question that has been advanced during this investigation is the validity of material recognition (i.e., fines) when using the CPT. It is acknowledged that these materials may vary from point to point with depth, but in the gross sense, materials should coincide with actual adjacent SPT sample gradations. Comparing the percentage of fines (laboratory results) determined from SPT explorations and those theoretically derived from the CPT friction ratio indicates the CPT results generally underestimate the laboratory results. The measured CPT results (e.g., CPT97-37 through -43 for Site D), normalized and corrected for fines, were analyzed to show how they compared to the values required for triggering (using SPT fines or the CPT apparent fines). It was observed that CPT results are similar regardless of whether the fines content was obtained from laboratory tests or theoretical equations from the CPT friction ratio (i.e., CPT apparent fines).

Conclusions

To achieve acceptable foundation treatment, proper implementation of stone column construction equipment, methods, and sequencing is essential.

Proper sequencing and the effect of closure was highly beneficial in optimizing foundation improvement. At least two passes of stone column construction, each pass having a specific sequence of stone column completion (outermost to innermost), will be utilized for the full foundation treatment contract.

It could be concluded that, for the identified loading conditions, construction of appropriately sequenced and spaced 3-foot-diameter stone columns on 6-foot centers may provide adequate foundation treatment by averaging of existing data. As evidenced in Site C, significant densification did occur; however, numerous data points still indicated liquefaction triggering. Furthermore, evaluation of information presented indicates that the amount of pore pressure increase to be expected under dynamic loading and, thus, the overall loss of shear strength to be realized within pockets or lenses of looser soils present within the foundation, could be reduced dramatically using the larger area replacement ratio (Ar) option. This is evidenced by the fact that essentially no values fall below the liquefaction triggering level for the higher Ar option (Site D) in comparison to localized areas or pockets of liquefaction which may trigger using the lower Ar option (Site C). This concern resulted in selection of the higher Ar option for the majority of the final design, which essentially uses the lower quartile of the penetration resistance improvement after treatment instead of the average. This has been determined to be appropriately conservative by Reclamation for this particular application (i.e., liquefaction mitigation at a high hazard, high risk [loss-of-life] facility).

The currently proposed configuration of stone columns for treating the entire dam foundation will consist of six rows of stone columns: the upstream two rows will be 3.75 feet in diameter, followed by two rows that are 3.0 feet in diameter and, finally, two rows at 3.75 feet in diameter. Center to center spacing will be 6 feet, and all columns will extend to elevation 2237 (about 60 feet in length). The shear strength of the treated area will be based on the assumptions that liquefaction will not be triggered to a steady state condition but will be reduced to account for some pore pressure increase.

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ENVIRONMENTALLY SAFE "GREEN" LUBRICANTS FOR WICKET GATES

by Leslie J. Hanna and Clifford A. Pugh[1]

Introduction

Greases are commonly used in hydroelectric facilities to lubricate wicket gate bushings. However, the greases presently used in many facilities could contain lead, phosphorous, lithium, and benzene compounds which ultimately might be introduced into waterways and affect water quality and the biological food chains. In an effort to address this issue, the Bureau of Reclamation (Reclamation) has conducted lubrication tests on candidate "environmentally acceptable" greases as possible replacements for greases currently used. Replacement of these lithium-based greases requires that water quality standards are met, as well as providing lubrication and protection of surfaces to maximize the service life of the wicket gate bushings. Current Reclamation standards specify a 40-year service life for bushings. Also, it should be noted that greases that are approved as "food grade" do not necessarily meet water quality standards. In order to assure that lubrication standards are met, laboratory tests were conducted. Some limited field tests have also been conducted at Reclamation's Hungry Horse Dam in Montana.

Reclamation's Water Resources Research Laboratory (WRRL) in Denver constructed a test facility and conducted tests to determine the relative lubricating performance of several candidate "green" lubricants. This paper compares these data with lubricating performance of a baseline lithium grease currently used in wicket gates. Additional chemical and physical property tests are also recommended, including toxicity and biodegradability (see Appendix A at the end of this article.) Often, many of these tests are supplied by the manufacturer.

Scope of the Study

- The study described in this paper concentrated on comparing relative lubricating performance of various greases. The work did not include analysis of other chemical and physical properties of the candidate greases. These additional tests would facilitate evaluation of the environmental effects of the various greases.

- The tests to date were all conducted at a constant water temperature (about 68 degrees Fahrenheit). A proposal has been prepared to evaluate lubrication performance at lower temperatures (about 34 degrees Fahrenheit.) This work has not yet been funded.

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- The tests were performed on five candidate “green” greases, one lithium-based grease, and one self-lubricating bushing. The lubricating properties are intended as a relative comparison.

Mechanical Test Setup and Procedure

A test apparatus was developed by the WRRL to establish a standard test to compare the mechanical performance of various greases for the bushing applications. The test apparatus was based on a 1:4 scale model of a prototype wicket gate at the Mt. Elbert Powerplant near Leadville, Colorado. The model gate is enclosed in a rectangular conduit with flow and pressure through the model roughly scaled to represent flow through one wicket gate passage at Mt. Elbert. The test head on the gate ranged from 21 feet to 54 feet. A motor-driven operator is attached to the shear lever arm. The model gate is controlled to simulate gate movements under automated generator control, the most severe duty cycle experienced by a wicket gate. The operator cycles the gate continuously on a 20-second, 4-degree stroke, with a 7-second pause between each cycle. In addition, a full 22-degree closing and opening stroke is executed three times per equivalent prototype day. Total model test time for each test conducted was 20 hours. This involved 1,330 opening and closing cycles at 4 degrees, and 40 opening and closing strokes at 22 degrees. Gate torque measurements were used to predict relative performance. Torque was measured with strain gages mounted on the wicket gate shaft in the test rig as shown on figure 1.

Test Results

Grease was injected into the bushings at 4-hour intervals, which simulates 60-hour prototype intervals. A lithium-based grease (Lubricant A) was used as the baseline for performance comparisons because lithium-based greases have typically been used for wicket gate lubrication. In addition, a test case using no grease (water lubricated) was used for comparison and to confirm the sensitivity of the test apparatus. Five "green" lubricants and one set of self-lubricated bushings were tested. The test apparatus and bushings were completely cleaned after each test case to prevent cross contamination between greases. The bushings were also inspected at this time for damage or scoring but, in each case, showed none. Maximum gate torque was recorded twice per hour during the full gate stroke. Figure 3 is a typical strip chart recording of the stresses in the two strain gages on the gate shaft during a full (22-degree) closing and opening stroke.

The top curve on each graph in figures 4 and 5 displays the maximum test apparatus torque values (in 1,000 lb-in) recorded during gate opening. The bottom curve on each graph displays the maximum torque values recorded during gate closing. To interpret the meaning of these graphs, a free body diagram of the test apparatus was used to analyze the forces acting on the gate. The difference between the opening and closing curves represents the

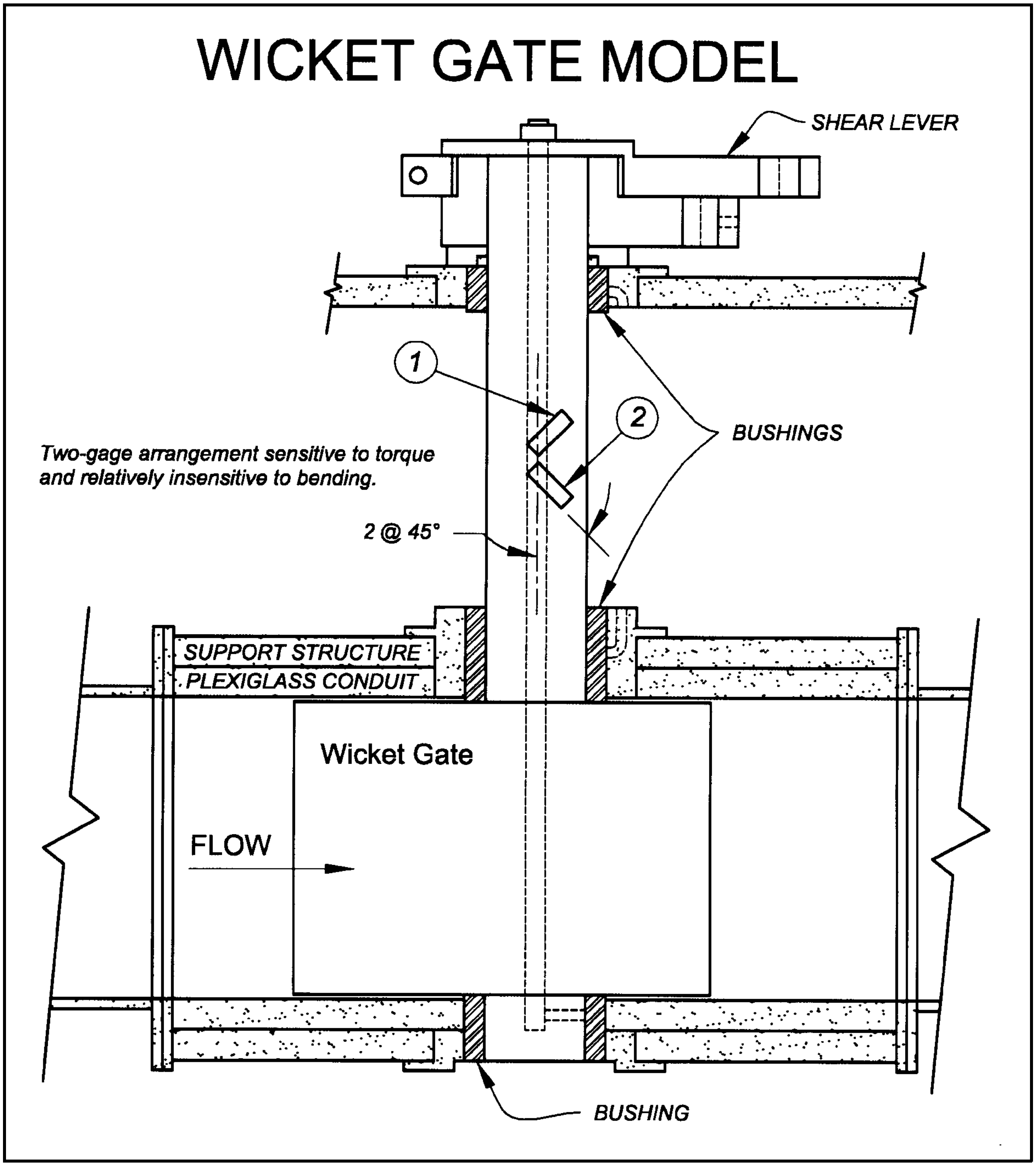

Figure 1.—Section through wicket gate model.

torque due to twice the friction torque inherent in the system. Since the torque force is a function of the lubrication properties of the grease, this value provides a quantitative tool to compare the performance of the greases in a standardized test. The maximum torque values during a full cycle were recorded and plotted over time for each test case. Using this analysis, torque due to friction (near the end of the test when the friction had stabilized) for each test

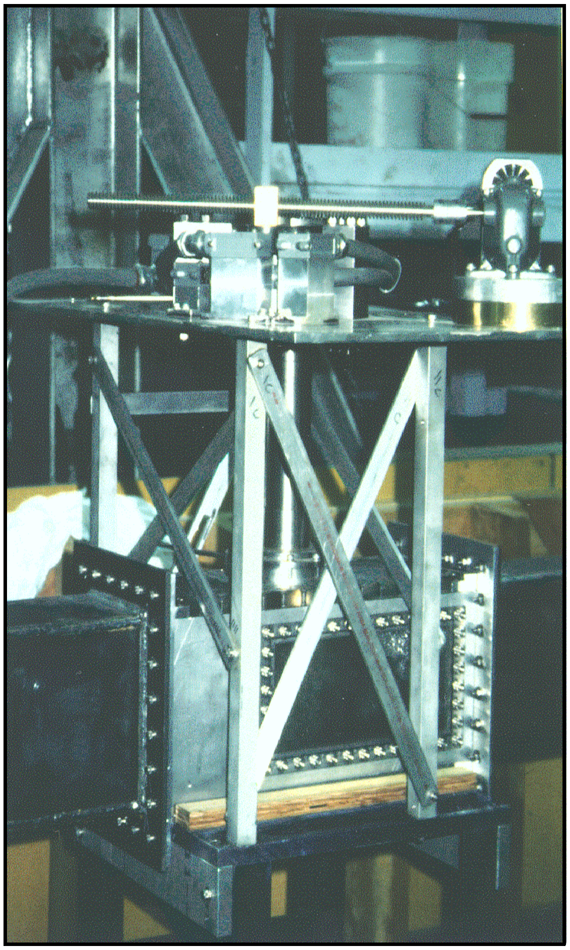

Figure 2.—Photograph of the wicket gate test apparatus.

case is given in table 1. Note from figure 4 that the friction torque for the "no lubricant" (water lubricated) case is still rising after 60 strokes.

The values given in table 1 show a relative comparison of how these "green" lubricants will perform compared to the traditional lithium-based grease. The results of these tests may be used as a baseline, in conjunction with field tests, to determine which lubricants will perform well in the field. Other mechanical properties of the grease, such as workability, will be important to field personnel.

Self-Lubricated Bushing Tests

Results of tests conducted on a self-lubricated bushing indicate that the self-lubricated bushing provides 86 percent of the lubrication difference between the water-lubricated and the lithium-base lubricant. However, more extensive tests are needed to determine the long-term viability of self-lubricated bushings. Wear characteristics of these bushings over an extended period of time are important because no lubricant is being added and the greasing process also acts to purge sand and silt from the bushing.

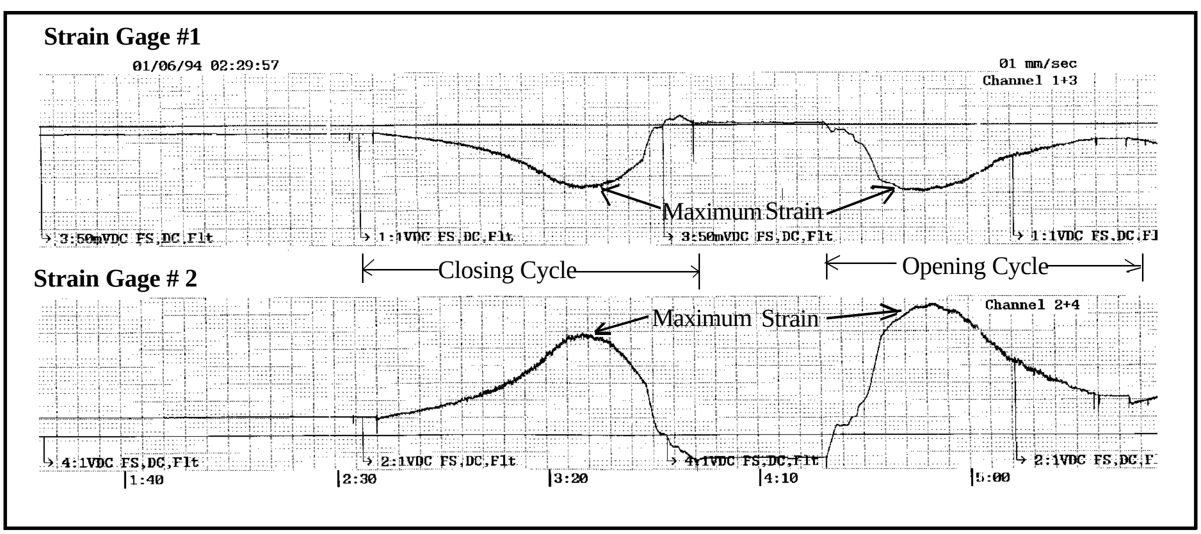

Figure 3.—Typical recording of wicket gate closing and opening cycle.

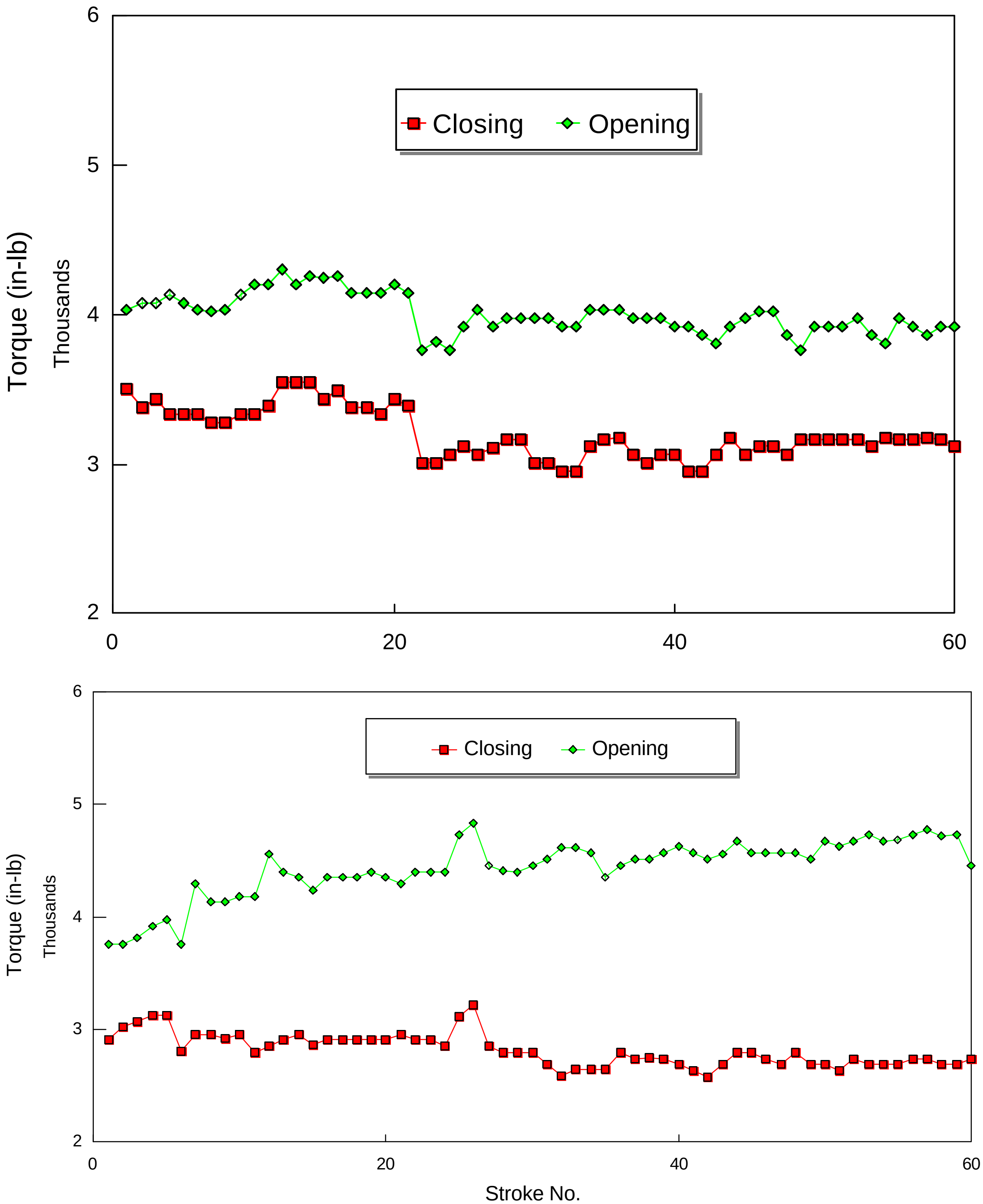

Figure 4.—Maximum torque versus stroke for baseline grease (above) and no grease—water only (below).

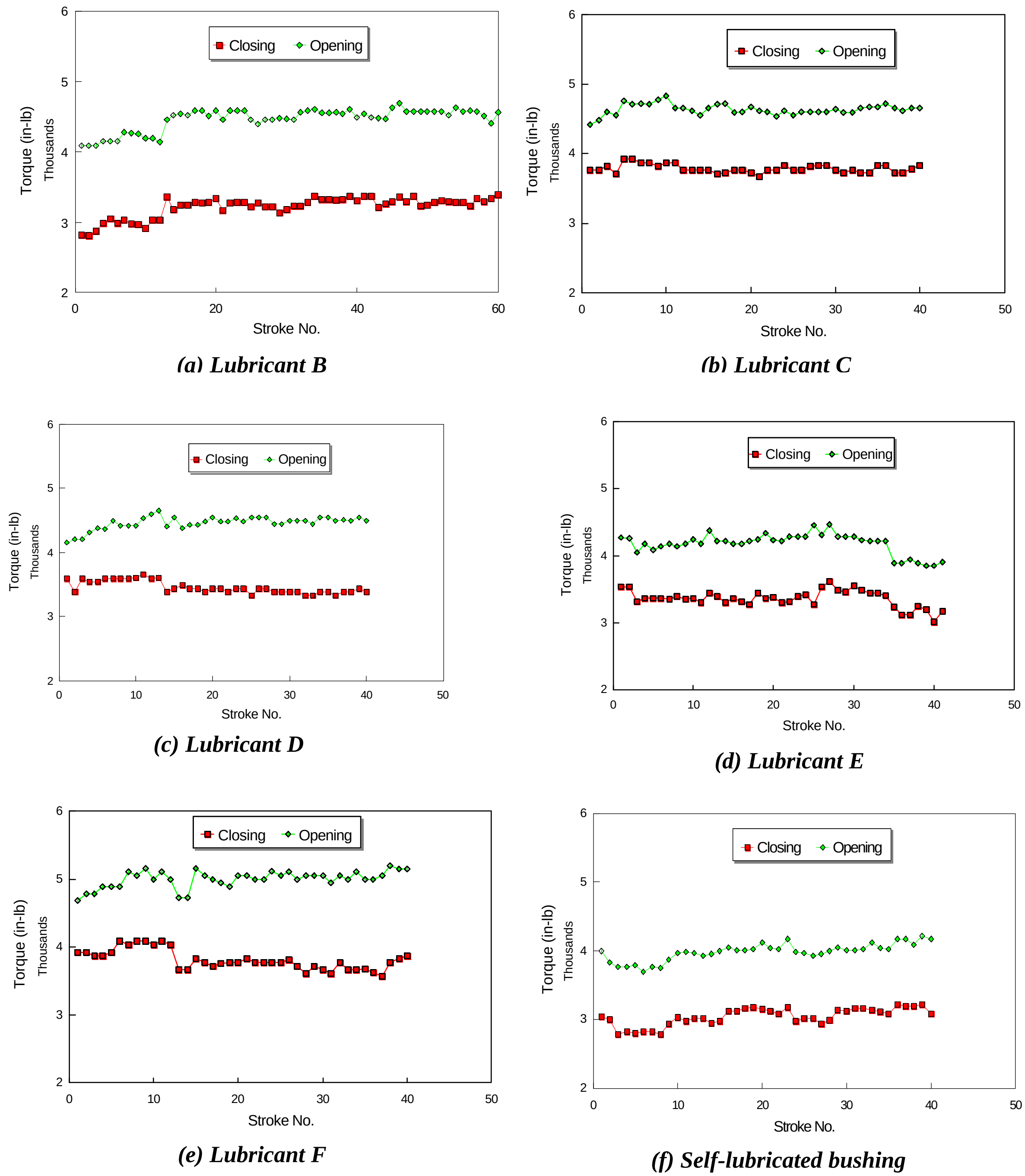

Figure 5.—Maximum torque versus stroke for five "green" lubricants and one self-lubricated bushing.

Table 1.—Friction torque indicating lubricating performance

Test case	Type of lubricant[1]	Friction torque (in-lb)	Percent lubrication[2]
Lubricant A (lithium-based grease)	L	401	100
Lubricant B	FG	629	55
Lubricant C	SE	437	93
Lubricant D	FG	590	63
Lubricant E	SE	377	105
Lubricant F	FG	675	46
Self-lubricating bushing	—	470	86
No lubricant (water only)	—	905	0

[1] L - Lithium, FG - food grade, SE - synthetic ester
[2] Percent of the difference between no lubricant (water only) and the standard lithium-based grease.

Conclusions

- The lubrication tests performed by Reclamation, as well as the property tests listed in Table A-1, can be used as a basis for the selection of environmentally safe "green" lubricants. Selection of an environmentally safe lubricant should be based both on environmental standards and mechanical performance.

- The lubrication tests indicated that ester-based lubricants performed significantly better than the food-grade greases that we tested. The average “percent lubrication” (PL) for the two synthetic ester-based greases was 99 percent. For the three food-grade greases tested, the average PL was 55 percent.

- More testing is recommended to ensure that mechanical performance, as well as environmental standards, are met. Many manufacturers have recently produced new products in an effort to meet environmental standards. However, until complete property tests are conducted, it will be difficult to determine the applicability of the products based solely on manufacturers' data and claims. In addition, more extensive tests will be required to determine the long-term viability of self-lubricated bushings.

Disclaimer

The test apparatus was designed to simulate conditions encountered in Reclamation's applications. The results are intended to allow relative comparisons of the candidate grease's lubricating properties. These tests do not imply an endorsement by Reclamation for any commercial product. Actual lubricant performance will also depend on field conditions. The lubricants and self-lubricated bushing tested in these investigations were contributed by the manufacturers.

Appendix A

Lubricant Property Tests

A list of additional tests which may be important to consider in conjunction with the lubrication tests performed by WRRL are provided in table A-1 at the end of this appendix. These tests were scheduled to be conducted on the same greases tested in the mechanical test rig; however, funding limitations prevented completion of the tests. Some of the tests may be available through the manufacturer or provided on the material safety data sheets. A discussion of these recommended tests as they relate to wicket gate grease applications follows:

(1) *LC_{50} for toxicity*—This test has been the standard required in Canada. (The "microtox" test can be used initially as a screening device since it shows high correlation with the LC_{50} and is much less expensive). In the United States, the 1986 Environmental Protection Agency standard, "Quality Criteria for Water," includes the LC_{50} test as part of the criteria for oil and grease. Individual States determine their own regulations, but most States have adopted these criteria. Several of the lubricants tested have received a food-grade designation. However, this designation alone does not guarantee that the grease is nontoxic and environmentally acceptable.

(2) *Biodegradability (CEC L-33-T-82)*—The Acronym CEC stands for Coordinating European Council. The test was developed to determine the biodegradability of lubricants in water. Vegetable oils and a number of synthetic esters easily meet biodegradability criteria. However, there are serious performance concerns for vegetable oils, especially at low temperatures. Ester-based lubricants can be designed to be readily biodegradable and nontoxic and possess lubricant performance advantages over vegetable oils; however, they are higher in cost.
Two of the lubricants tested were ester-based lubricants.

(3) *Copper strip corrosion test (ASTM D4048)*—This test identifies undesirable reactions of the lubricant with the bronze bushing that could lead to excessive and unnecessary wear. The copper corrosion test became of particular interest after field testing one of the "green" lubricants at Hungry Horse Dam in Montana. On inspection of the power unit, which had used this product for about 6 months, there appeared to be a copper coating on the wicket gate shaft. This was not seen on the units that had used the lithium-based lubricant. A chemical analysis of a sample scraped from the shaft indicated that the sample contained a significant amount of copper. Additionally, a sample of the lubricant used in the model tests showed significantly more copper than an unused sample of the same grease (3,640 milligrams per kilogram as opposed to 3 milligrams per kilogram in the unused sample). Galvanic and resistivity tests of the lubricant conducted by

Reclamation's Materials Engineering Branch showed that the grease had high resistivity to current flow, thus eliminating this as the cause of the copper transfer. These results may indicate that the grease is chemically reacting with the bronze.

(4) *Element scan (ASTM D4951)*—This test can distinguish which of the lubricants contains metal components that can be harmful if they find their way into the biological food chain.

(5) *Resistance to water spray (ASTM D4049)*—This test serves as a relative indicator of how quickly the lubricant will be washed out of the bushings during field operations where it is subjected to high water pressure. One of the best ways to protect the environment is to simply put less grease into the waterways by using a lubricant that is not washed out easily and by adjusting greasing schedules accordingly.

(6) *Rust preventive characteristics (ASTM D665)*—Some of the "green" lubricants may not have adequate rust preventive additives needed for long-term performance.

(7) *Compatibility with mineral oil*—This is important since the "green" lubricants will, in most cases, be replacing mineral oil lubricants. Incompatibility of the new lubricants with the traces of mineral oil that will be left behind may cause formations of gums, varnishes, or other insoluble contaminants.

(8) *Water solubility*—This test can determine if the lubricant is absorbing water which comes into contact with it. If this tendency occurs, the lubricant may eventually become diluted with water, which will change its lubricating properties and may cause rust or premature breakdown of the lubricant.

(9) *Storage stability*—Biodegradable products may have a tendency to biodegrade on the shelf before they are put into service. This will test the tendency of the lubricant to biodegrade before use.

(10) *EP properties or Timken rating (ASTM D2509)*—This test determines the extreme pressure (EP) characteristics of the greases which are classified with a Timken load rating. One question that has arisen in selecting lubricants is whether a high Timken rating is required for wicket gate bushing applications. EP additives control wear rather than prevent wear. The EP additives react with the metal to form a compound which acts as a protective layer on the metal's surface, preventing metal to metal contact that can lead to scoring or failure. Under extreme pressure conditions, this layer is sacrificial and wears away, protecting the metal. As this layer is removed, the EP additive acts to form another layer. To prevent excessive corrosion, most EP additives are activated by excessive heat created during extreme pressure conditions but do not react at room temperature. Although there is a question as to whether the point pressure within the wicket gate bushings is ever high enough to activate the EP additive, the Timken ratings of greases currently being used in Reclamation facilities range from about 40 lb to 45 lb.

Table A-1.—Lubricant property tests
(Suggested by BC Hydro)

Test name	Test description	Test method
Biodegradability	Developed to determine the biodegradability of lubricants in water	CEC L-33-T-82
Toxicity	Rainbow trout will be exposed to lubricant-water dispersion	LC_{50}
Toxicity of degraded products	Same as above, except degradation products of lubricants will be used	LC_{50}
Element scan	Determines elemental concentrations	ASTM D4951
Copper strip corrosion	Determines lubricant's corrosiveness to copper	ASTM D4048
Rust preventive characteristics	Indicates the ability to prevent rust	ASTM D665
Resistance to water spray	Evaluates the ability of the lubricant to stick to a metal surface when subjected to direct water spray	ASTM D4049
Hydrolytic stability	Determines the stability of the lubricant in water	ASTM D2619
Compatibility with mineral oil	Determines the compatibility of the replaced mineral oil with the new lubricants	FTM 791C Method 3470.1
Water solubility	Determines water absorption of lubricant	In-house test
Storage stability	Determines breakdown of lubricant during storage	FTM 791C Method 3467.1
Categorize grease	Determines if composition agrees with specification sheet	Infrared scan
Compatibility with elastomers	Determines lubricant's effect on elastomers	ASTM D4289
Swelling of synthetic rubbers	Determines lubricant's effect on synthetic rubbers	FTM 791C Method 3603.5
EP properties - Timken	Determines EP characteristics	ASTM D2509
Wear characteristics	Determines relative wear preventive properties	ASTM D2266
Worked penetration	Determines consistency within NLGI grades	ASTM D217

REACH 11 DIKES MODIFICATION—A VERTICAL BARRIER WALL OF HDPE GEOMEMBRANE

by Mark Bliss, P.E., U.S. Bureau of Reclamation

Background Information

Introduction

The Hayden/Rhodes Aqueduct - Reach 11 Flood Detention Dikes (commonly referred to as the Reach 11 Dikes) are approximately 15 miles in length. The four dikes, with a maximum height of approximately 45 feet, are constructed of silty and clayey materials and parallel the aqueduct along the northern boundaries of Phoenix and Scottsdale, Arizona. The dikes were constructed to protect the canal from stormwater runoff (up to the probable maximum flood level - PMF) by temporarily storing and then gradually releasing stormwater into the canal. Thousands of homes in Phoenix and Scottsdale are located directly downstream of the dikes, making them high-hazard structures.

Since the project's completion in 1977, no releases from the dike outlet structures have been made into the canal. Any ponded water from local storms percolated into the ground or evaporated. These often intense rains have produced rills, gulling, and erosion tunnels. Severe cracking has occurred in both longitudinal and transverse directions. These cracks often are enlarged by erosion caused by downward percolation of rain water.

Geotechnical Deficiencies

Laboratory testing programs were developed in 1988 and completed in 1989 to address the severe cracking and erosion problems. The objective of this testing was to identify the causes of the cracks, determine if the dikes could be safely operated under normally expected loading conditions in their current state, and recommend alternatives to correct any deficiencies. The conclusions from these investigations are summarized below:

(1) Foundation pretreatment prior to placement of the dikes, consisting of prewetting and surface compaction of foundation materials, was only effective to relatively shallow depths. Undisturbed samples of foundation soils beneath the dike indicated that in-place dry densities are low and decrease rapidly with depth.

(2) Trenching on the dike crests and in-place moisture determinations showed that desiccation of dike materials had occurred to a depth of 3.5 - 4.0 feet, was relatively widespread, and cracks were generally hairline in width and consistently spaced.

(3) Large, deep, longitudinal, diagonal, and transverse cracks in Dikes 1, 2, and 3 were caused by differential settlement of foundation soils due to low in-place densities and infiltration of water. Sections of the dikes with longitudinal and transverse cracks can be correlated to areas having retained water during past rains.

(4) Historical rain events have shown that the dike embankment materials are highly erosive and dispersive. Laboratory testing on embankment samples indicated that approximately 40 percent of the materials tested were classified as D-1 dispersive.

(5) Failure of the dikes is likely in the event of either brief or sustained storage from rains. Transverse cracks extended through some of the crests to depths exceeding 16 feet, possibly into the foundation. Because of the erosive and/or dispersive properties of the materials, and based on the extensive erosion of the dikes during rainstorms, it is likely that seepage flowing through cracks would quickly erode the dike material and cause breaching.

Geology

The dikes trend southeasterly across Paradise Valley, a structural basin containing thick accumulations of quaternary fluvial and lacustrine sedimentary deposits. All four dikes are founded on basin fill and alluvial fan deposits. The basin fill deposits consist of silty sand, clayey sand, and sandy silt. The alluvial fan deposits are coarser, consisting primarily of silty gravel to gravelly sand containing cobbles to 12 inches in diameter.

According to research done on the prediction of field collapse of soils due to wetting, collapsible soil deposits are primarily fluvial and wind deposited, weakly cemented silty sands, sandy silts, and clayey sands of low plasticity. The majority of the basin fill deposits under Dikes 1, 2, and 3 fit these soil types. When the collapsible soil is allowed free access to moisture, the salt, clay, or silt binder that is providing the bonding mechanism between the larger particles will soften, weaken, and/or dissolve. These bonding materials reach a stage where they can no longer resist the existing overburden stress, and the soil structure collapses.

Evaluation of Alternatives

Since the probability of failure was high, Reclamation undertook a detailed alternative study. The length of the dikes indicated that repair costs were potentially very large. Therefore, cost and long-term performance were key factors in the evaluation of alternatives.

The following alternatives were evaluated:

(1) Cut-off wall options
  (A) Slurry walls of soil-bentonite or cement-bentonite mixtures
  (B) Soil mix wall (SMW)
  (C) Geomembrane barrier wall

(2) Filter options
 (A) Downstream filter zone
 (B) Modification of the dikes with filter and drain
 (C) Removal and reconstruction of the dikes with a filter zone

(3) Geomembrane options
 (A) Upstream geomembrane
 (B) Vertical geocomposite barrier wall

Each alternative was evaluated based on technical and economic merit. Because of the presence of erodible silts and dispersive clay soils, the design team eliminated options which did not include a soil filter. However, since surface erosion can create deep gullies along the slopes of the dike, a downstream filter was likely to be subject to eventual erosion. The downstream filter would also create the undesirable potential for high seepage gradients to develop at the downstream toe. The conclusion was that a vertically placed filter zone (drainage trench with finger drains) through the centerline of the dike offered the best protection from future erosion, would require the smallest volume of filter material for construction, and was the least costly solution. Based on these observations, a decision was made to place the filter within a narrow trench excavated vertically through the crest. The excavation and placement of filter zone materials could best be accomplished by the slurry trench method. The hydrostatic pressure of the slurry is the primary stabilizing force supporting the soils during trenching. To prevent contamination of the filter zone material with residual materials of the slurry (typically bentonite) after installation, a biodegradable slurry was selected. A biodegradable slurry, such as a natural guar gum or a synthetic biopolymer, can be chemically broken down and flushed out of the filter zone material, leaving a free-draining filter zone in place. Horizontal finger drains were excavated at 500-foot spacings along the dikes prior to excavation of the vertical trench.

There also was concern for high seepage gradients across the vertical filter zone that could wash filter material into a downstream crack. To prevent this, a vertical barrier wall was included against the upstream wall of the trench. The vertical barrier wall material selected for installation in the trench was an interlocking, jointed HDPE geomembrane, commonly referred to as a curtain wall. The curtain wall was installed in the biopolymer trench before placement of the filter zone material.

Test Section

A 50 foot deep x 1,100 linear foot representative test section was installed in a severely cracked section of Dike No. 1 to assess the constructability. The test section was then impounded with water for 30 days at a water level equivalent to the PMF to evaluate the performance of the design. The water was impounded behind the dike with the use of temporary berms constructed as part of the test section contract. The interior slopes of these berms were lined with 30 mil HDPE geomembrane to minimize seepage losses. The basin floor of the test pond was left undisturbed.

Description of Test Section

The test section was designed to model a typical "modified" dike cross-section representative of the approximate 12.5 miles of dikes needing repair. The test section was placed within a section of Dike 1, which had a history of large cracks and erosion features. The trench was also constructed to the maximum depth of 50 feet. Critical aspects of the construction process which were closely monitored included:

(1) Trench excavation, including performance of the contractor's equipment and stability of the biopolymer slurry.

(2) Slurry losses from the trench into the embankment and foundation and particularly through transverse cracks.

(3) Installation techniques of the curtain wall.

(4) Tremie placement methods of the zone 2 filter material.

Berms were placed around the test section to contain the temporary pond that would be filled to within 3 feet of the top of the dike. The water for filling the test section and maintaining the pond for 30 days was obtained from the Hayden/Rhodes Aqueduct paralleling the dikes. Monitoring of the test section was performed 24 hours a day, 7 days per week and consisted of frequent visual inspections and recording of data from a large array of instruments.

Evaluation of HDPE as the Preferred Material Choice

The geomembrane types selected for the design were evaluated based upon the following desired physical properties and installation characteristics:

(1) Low permeability
(2) High strain to failure (Tear strength)
(3) High puncture resistance
(4) Flexibility
(5) Sealability and verification of interconnected joints at depth

The HDPE geomembrane not only possessed these properties and characteristics but also had been successfully installed to depths greater than 100 feet at other sites. The proven methodology for the installation provided assurance of a full-depth vertical barrier wall, ease of installation, and a technique to maintain the curtain wall in place while backfilling was in progress. The barrier wall included these components:

(A) Curtain Wall—HDPE geomembrane placed vertically in the subsurface. The thickness utilized on this project was 80 mil. HDPE has very low permeability.

(B) Interlock, Joint, Panel—Multi-channel locking device made out of HDPE requiring two pieces to form the joint (figure 1). Joints were composed of 160-mil-thick HDPE. Utilization of this component with HDPE geomembrane forms a panel. The size of the panels used was 24 feet wide x 54.5 feet long. Length is the depth of the excavation plus flaps—in this case, 50 feet of excavation plus a top flap of 4.5 feet. The top flap served to secure the panels at the surface and prevent drag down from the backfill operation. The design of the joint will allow a vertical slip plane. This feature is important since the foundation is collapsible and differential settlements are likely.

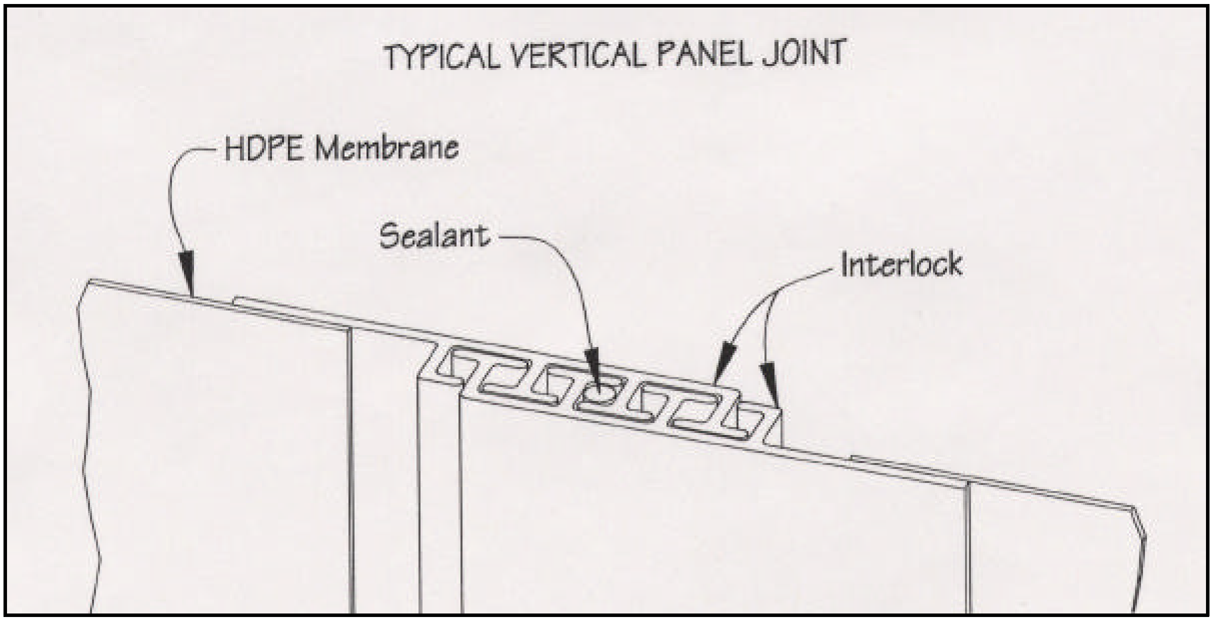

Figure 1.—Placement of sealant in interlocked joint.

(C) Sealant—Hydrophilic rubber sealant. Placement of this additional sealant was concurrent with insertion of an attaching panel. When wetted, the sealant is capable of swelling to 8 times its size in volume in an unconfined state. When installed in the middle chamber of the interlocking joint, the swelling is confined, and the resulting force causes the joint to compress or fit tighter.

(D) Electronic Joint Verification—A contact element and conductive wire is attached to each interlock. The contact elements are positioned on the bottom of the interlock so as to touch each other when two panels of curtain wall are properly installed.

(E) Frames—Insertion frame used for precise placement of panels. The angle iron attached to the bottom of the panel serves as ballast, since HDPE is buoyant in water. Two frames were used in an alternating fashion.

Instrumentation

In order to evaluate the performance of the design, the test section was heavily instrumented and monitored for the 30-day test period. Instrumentation consisted of porous tube piezometers, observation wells consisting of slotted pipe, settlement points, and miscellaneous *in situ* testing such as pressuremeter and cone penetrometer.

Installation Techniques - Test Section

The construction of the test section began in June 1993. A summary of the major work items and the proposed construction methods is outlined below.

Excavation

The contractor excavated the trench with a specially modified excavator designed to dig 56 feet. The trench was excavated under biopolymer slurry support to a maximum depth of 50 feet. Trench design width was 24 inches.

Zone 2 Filter Material "Backfill"

The backfill in the trench was an ASTM C-33 concrete sand designed to be tremied into the slurry filled trench. The sand met filter gradation requirements for the embankment and foundation materials.

Curtain Wall

The panels were fabricated at the manufacturer's production facilities and shipped to the site. Each panel was tested and certified to meet minimum value specification requirements (table 1). Verification testing of interconnection of joints at depth was done on every joint.

Finger Drains

One horizontal finger drain was excavated into the embankment at the center of the test section and the embankment backfilled prior to trenching. This drain extended from the downstream toe to intersect with the vertical trench and is composed of C-33 concrete sand.

Table 1.—Curtain wall panels specifications and performance requirements

Property	Test method	Minimum values
Thickness	ASTM D1593/D751	80 mil (2.03 mm) ± 5%
Density	ASTM D792	0.940 g/cc
Tensile properties		
Yield strength	ASTM D638	173 lb/in (31 kg/cm)
Yield elongation	ASTM D638	13%
Break strength	ASTM D638	324 lb/in (59 kg/cm)
Break elongation	ASTM D638	560%
Impact strength	ASTM D1822	381 ft-lb^2 (801 mj/mm^2)
Impact elongation	ASTM D1822 FTMS 101C Method 2065	100%
Puncture strength shrinkage, dimensional	ASTM D1204 (100°, 1hr)	98 lb (44 kg) ± 2%
Factory seams		
Leakage	Air pressure test each seam with 100 psi for 2 minutes	Must have less than 4 psi loss in 2 minutes
Shear test	Place seam in shear	160 lb/in (28 N/mm) of width
QA test Interconnection of adjoining panels at designed depth	Battery, creates an electrical circuit when contact elements from adjoining panels come in contact	Electrical resistance is measured confirming circuit

Piezometers/Observation Wells

All instrumentation was installed without the use of drilling fluids. Reclamation installed all instrumentation with the use of a CME hollow stem auger except for the observation wells located within the trench backfill. These wells where placed in the trench just prior to backfilling operations.

Evaluation of Test Section Construction

Vertical Trench

A 24-inch trench width was adequate for this work and provided sufficient width for the curtain wall installation.

Slurry

During evaluation of the contractor's originally proposed guar gum slurry, a synthetic biopolymer slurry was proposed. Based on trenching tests at the site, this slurry was approved and performed successfully during the test section by providing a cleaner zone 2 backfill.

Curtain Wall

No significant installation problems were noted during the test section construction. It was found that installing panels (54.5 feet x 24 feet) of this size during windy periods (greater than 20 knots) can be a problem. The interlocking mechanism of the panels and their construction, in general, was of high quality. Placement of one panel into the previously placed panel is easily done as long as verticality is maintained. The joint's interconnection can be verified by the use of an electronic circuit which is completed once the panel is fully installed to designed depth. In this case, panels were manufactured 4.5 feet longer than the specification depth in order to provide a flap that could be folded over and staked down. This helped to prevent slumping and maintain verticality.

Filter Material, Zone 2

During the first placements of zone 2, the contractor attempted to pump the saturated zone 2 through a concrete pump directly into the trench. This procedure resulted in significant mixing of the zone 2 backfill and slurry and created a high potential for segregation of zone 2 in the trench. Once a true tremie operation was established involving the introduction of water saturated zone 2 into a steel pipe embedded within the zone 2 backfill, the placement worked well.

Evaluations of the zone 2 tremie placement methods indicated the following:

(1) Saturated backfill using the tremie method produced very little slurry/zone 2 mixing.

(2) No significant segregation of the zone 2 - C-33 concrete sand backfill appears to have occurred.

(3) Water present in the zone 2 backfill drains naturally from the trench into the foundation within a few weeks.

Evaluation of Performance

Performance of the test section was monitored over a 30-day period in which a PMF level reservoir elevation was stored behind the dike. Evaluation was based upon data recorded from observation wells, porous-tube piezometers, settlement/deflection points, and visual inspections.

General Observations

The following observations resulted from evaluation of the test section construction and performance during ponding:

(1) The curtain wall is an effective watertight vertical barrier wall.

(2) Installation of a curtain wall can be done to depths of at least 50 feet.

(3) Deep vertical trenching, supported by biopolymer fluid, can be an effective means of placing both a HDPE barrier wall and filter zone materials.

(4) A test section is recommended to better define construction procedures and understand the behavior of the biopolymer fluid being used.

(5) Trenching with biopolymer fluids is most effective in finer grained soils. Where groundwater depths are shallow, the particular site conditions should be carefully evaluated to assure effective trench support.

Conclusions

Based on the results of the test section construction and monitoring, the contractor was awarded Part 2 of the contract to complete modifications to the full 12.5 miles of dike. Construction was begun in December 1993 and completed approximately February 1995, almost 6 months ahead of schedule. This method of installation proved to be a cost-effective and efficient way of creating a deep seepage barrier within existing ground.

Mission

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

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