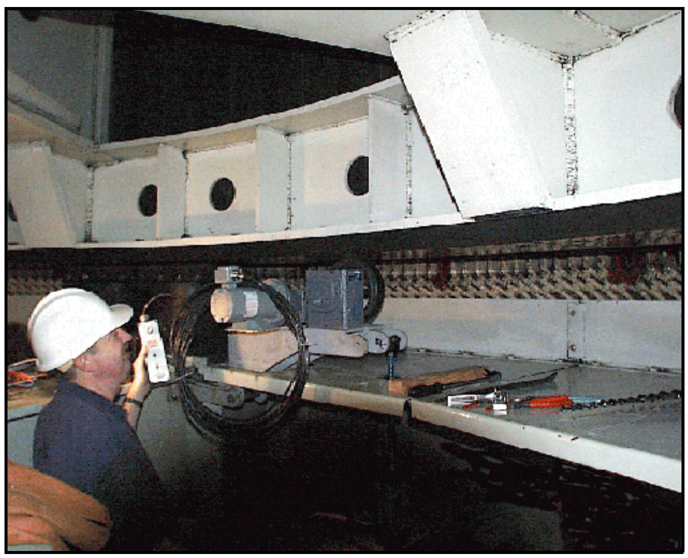
WATER OPERATION AND MAINTENANCE BULLETIN

No. 198 **December 2001**



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

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***Cover photograph**: Operation of rotor turning gear. The rim of the generator rotor is overhead.*

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CONTENTS

Page

Available on the Internet at: **http://www.usbr.gov/infrastr/waterbull**

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

by Ron Luehring[1], Nathan Snorteland[1], Mike Stevens[2], Lelio Mejia[3]

Abstract

*The use of stone columns, in combination with drainage wicks, can effectively mitigate the liquefaction potential of silty soils. This paper presents the results of using over 1,000 3.0- to 3.75-foot-diameter dry bottom-feed vibro-stone columns constructed in up to 60 feet of interbedded fluvial-lacustrine sandy and silty foundation materials beneath Salmon Lake Dam in north-central Washington. 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 (MCE). State-of-the-practice data conversions were used to perform the liquefaction analysis on the basis of clean sand equivalent blowcounts. Post-construction site characterization indicates: (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 using stone column treatment with measurable density increases, even in fine-grained silty soils. Key discussion is provided based on observations related to the effect of nonplastic fines on liquefaction mitigation, foundation pore pressure response during construction, and influences of sequencing during stone column construction. (**Note**: This article is printed as a followup to the original article printed in the December 1998 bulletin, No. 186).*

Background

Salmon Lake Dam is situated on a tributary of Salmon Creek about 15 miles northwest of Okanogan in north-central Washington and immediately upstream of the town of Conconully (figure 1). Completed in 1921, the dam consists of a 30-foot-high zoned earthfill embankment with a crest length of 1,260 feet and a combined spillway/outlet works structure.

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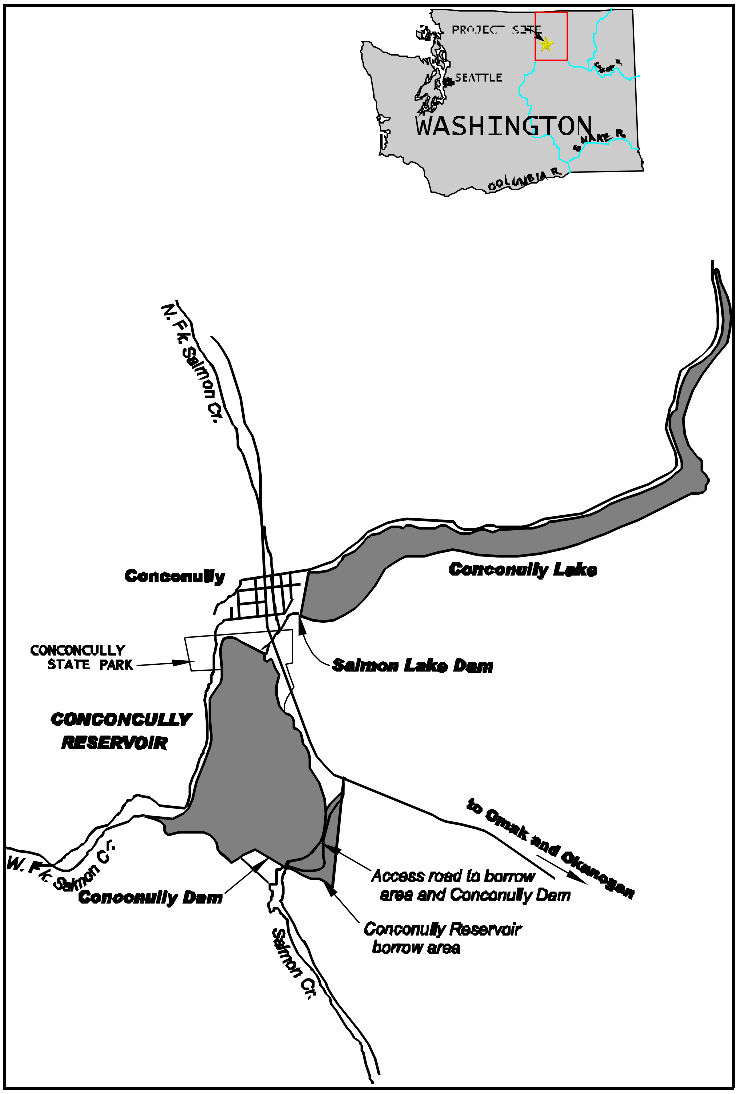

Figure 1.—Location/vicinity map.

The dam foundation consists of Quaternary fluvio-lacustrine sediments under most of the embankment to depths of up to 297 feet. These sediments are generally cohesionless, inter-bedded to laminated silty sand, with interbeds and lenses of silt with sand, sandy silt, poorly graded sand, and silty sand with gravel.

Analysis of the earthquake catalog led to the determination of a maximum credible earthquake (MCE) of M_L 6.5 for a random event at a distance of 29 kilometers [1]. The maximum peak horizontal bedrock acceleration for this source was estimated to be 0.26 g [2]. This MCE can produce high excess pore pressures and loss of shear strength in foundation layers susceptible to liquefaction. Significant foundation site characterization (SPT, CPT, Becker Penetration Tests, and Crosshole Shearwave Tests) was completed for the Corrective Action Studies (CAS). $N_{1(60)m}/N_{1(60)r}$ ratios were computed for various earthquakes. Triggering analyses (ratios less than 1.0 to 1.2) indicated widespread distribution of potentially liquefiable foundation materials.

Structural Modifications

A comparison of liquefaction mitigation alternatives was made during the CAS design phase of investigations and the selection of a preferred design alternative [2]. Alternatives were reviewed and checked by Woodward-Clyde Consultants and independently compared to other alternatives during a Safety of Dams Value Engineering Review [3].

Nonstructural modification alternatives included a permanent reservoir restriction and/or the potential use of an early warning system. Of the 10 structural Safety of Dams (SOD) modifications investigated, the use of vibro-stone columns ($3.8 million) was selected as the preferred alternative and ranked higher than dynamic compaction ($5.2 million) and excavate and replace ($5.9 million) methods. Factors influencing the selection of the preferred alternative included: foundation stratigraphy (sandy silt, silty sand, and silts with treatment depths to 60 feet), groundwater levels within 3 feet below original ground, dewatering requirements, a confined construction area, a limited area for excavation, and resident encroachment immediately downstream of the dam toe.

In addition to treating the foundation using the dry bottom-feed vibro-stone columns method of construction, the structural SOD modification included a 29-foot-high buttress and a 19-foot-high stability berm "sandwiching" a two-stage filter and toe drain system on the downstream excavated slope (figure 2). The downstream buttress was constructed to address upstream slope stability.

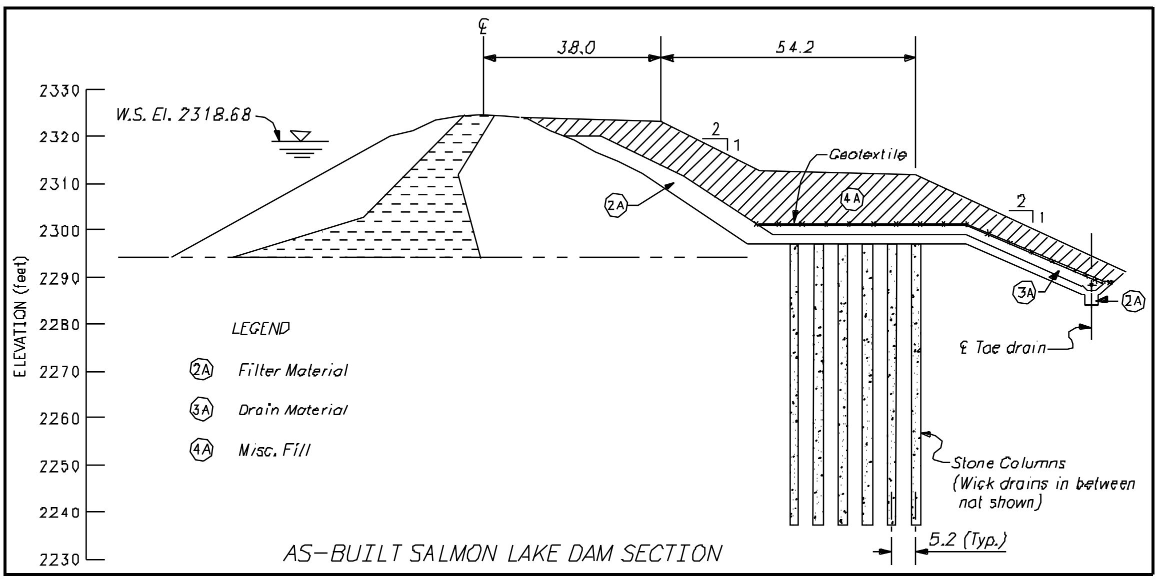

Figure 2.—SOD modifications post-construction section.

The effectiveness or degree of densification resulting from the installation of vibro-stone column systems is a function of soil type, silt and clay content, soil plasticity, pre-treatment soil density, vibrator type, volume of the stone, spacing between stone columns, and sequencing of stone column construction. The use of vibro-stone columns as a liquefaction mitigation alternative has several benefits (i.e., the vibrations created by the dry bottom feed displacement method densifies cohesionless sands and silty sands, added stone improves drainage characteristics of the treated soil for pore pressure release, and the stone column creates a reinforcing element that results in reducing cyclic shear stress in the weaker surrounding soils) [4]. Energy created by this method is confined to depths and locations of treatment and, thus, does not affect adjacent residences as other methods may.

Acknowledging the limits of the vibro-stone column equipment to densify silty soils, Reclamation recognized the need to test the equipment to ensure treatment capability within the gradational envelope of the foundation.

A test section was constructed in July 1997 to investigate the effectiveness of vibro-stone columns as a ground-improvement method in the heterogeneous silty to silty sand foundation and to help optimize stone column treatment (i.e., diameter, spacing, sequence) design by comparing pre- and post-test section foundation strengths (i.e., SPTs and CPTs) [4]. Before the SOD modification, additional verification testing was performed in the test section to evaluate the potential for strength increase over time (aging). Analysis of aging data is ongoing and incomplete at the time of this paper.

Wick drains were installed on 3- or 6-foot spacings (depending on the row) to full stone column depth and extended about 6 inches above the working surface. The SOD constructed foundation treatment configuration consisted of six rows of 3.75- and 3.0-foot-diameter stone columns constructed on 6-foot centers in six rows, forming equilateral triangles with a row-to-row spacing of about 5.2 feet (normal to the dam's centerline, as shown in figure 3). The two rows of stone columns furthest upstream and downstream were constructed at 3.75-foot-diameter and the interior two rows at 3.0-foot diameter. Between stations 11+50 and 13+00, all stone columns constructed were 3.75-foot diameter to account for a perceived finer-grained foundation in this area. The target design depth for construction of all stone columns was between 58 and 61 feet.

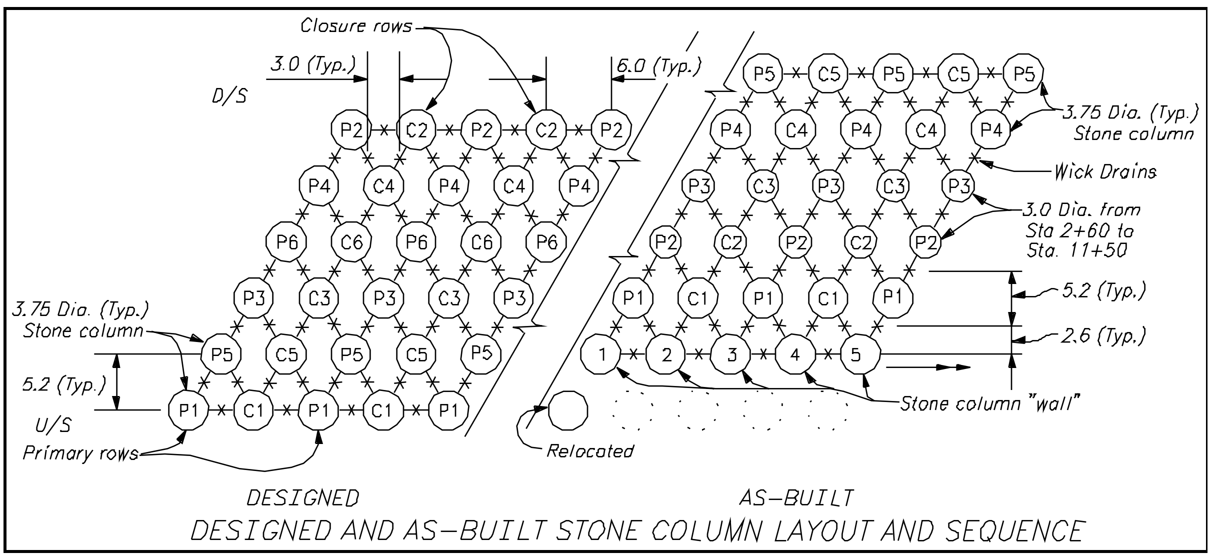

Figure 3.—Designed and as-built stone column layouts and sequence comparisons.

During construction of the first row of columns at the toe of the embankment, air was observed exiting through a crack on the downstream slope of the dam about 10 feet vertically above the downstream excavated toe, and a small "water spout" was seen flowing on the slope adjacent to where a closure column was being constructed nearest the toe. The contractor was required to cease construction, and the entire pattern was relocated downstream about 5 feet. This change provided a row of wick drains upstream of the first stone column row. The first row of stone columns was constructed in its entirety parallel to the centerline of the dam and excavation cut slope to create a "wall" of columns to reinforce the cut slope, protect the dam embankment, and alleviate excess pore pressure buildup. No pore pressure relief expressions were subsequently observed. A total of 1,020 stone columns (44,000 lineal feet of 3.75-foot-diameter stone columns and 16,800 lineal feet of 3.0-foot-diameter stone columns) were constructed by the specialty subcontractor, Hayward-Baker, Inc., requiring about 16,900 cubic yards of crushed stone.

Stone columns could not be constructed to the full design depth in some areas of the foundation. The largest area was about 50 feet long near the left end of treatment. The depth of treatment was limited by the nearby presence of bedrock and dense soils in the foundation as well as installation sequencing issues. Verification testing in the left abutment area indicated adequate treatment despite the inability to achieve the design depths. SPT blowcounts approaching or exceeding refusal and CPT tip resistance exceeding 300 tsf at depth reflect the presence of dense soils and influences of bedrock in the foundation near the left abutment.

Site Characterization (Pre- and Post-Construction)

Over the past 8 years, the foundation/embankment explorations of Salmon Lake Dam have progressed from a general geologic and materials investigation to a site-specific characterization geared towards quantitatively evaluating liquefaction triggering.

Two methods, SPT and CPT, were selected to provide site characterization before and after site remediation since both are considered reliable for sandy and low-plasticity silty soils. These methods are considered the most cost efficient and technically viable tools to provide information necessary to assess liquefaction "triggering" [5, 6, 7, 8].

Before foundation treatment, 11 SPT borings (2 SPT borings in the two test sections and 9 SPT borings distributed across the downstream toe) were used to characterize the site to depths approaching 70 feet.

Because of a CPT's ability to achieve a nearly continuous record of penetration resistance with documented repeatability in a very short time and low expense, they were used to supplement the SPTs site characterization. Forty CPT soundings (3 CPT soundings in each of the two test sections and 34 CPT soundings distributed across the downstream toe) were performed before treatment.

Because of the silty nature of the foundation soils, verification testing to characterize foundation improvement after stone column construction of "designated treated areas" was conducted a minimum of 2 weeks after the construction of the last stone columns at each site in an effort to allow pore pressures to dissipate before testing. Verification testing included 17 SPT borings and 93 CPT soundings (18 post-test section CPT soundings, 12 CPT soundings investigating the "aging" affect before construction, and 63 post-construction soundings) (figure 4).

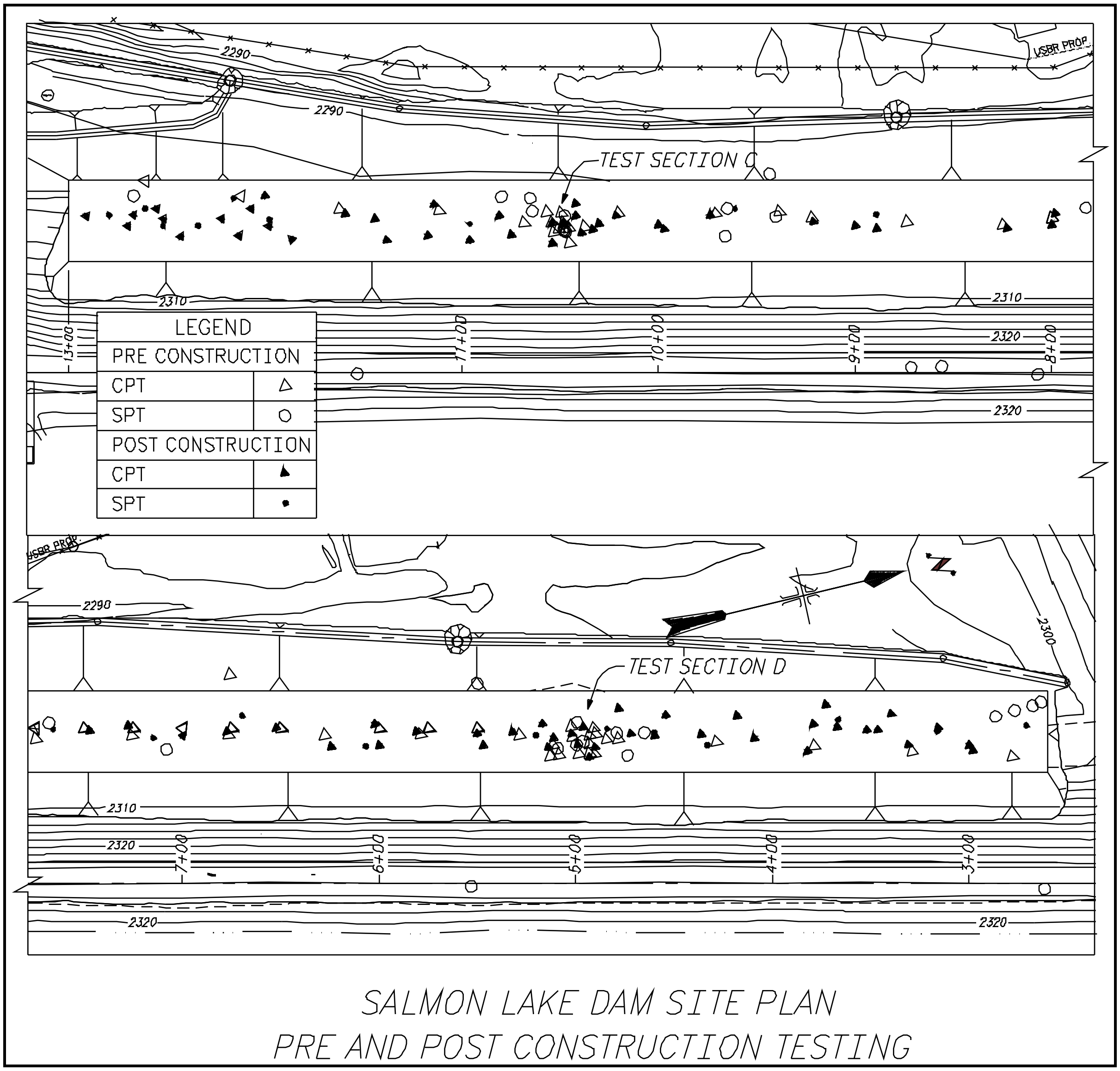

Figure 4.—Distribution of pre- and post-construction verification testing.

Behavior of Drainage Wicks and Piezometers

About 107,700 linear feet (1,982 wicks of varying length) of drainage wicks were located equidistant between the planned locations of the stone columns extending to the full design column depths. The number of wick drains surrounding any stone column varied (figure 3).

Installation of the drainage wicks was expected to enhance reduction of air and water pressures during the stone column construction process; the wicks protected other areas of the foundation and embankment from excess disturbance and/or hydraulic fracturing.

During the construction of any given stone column, up to 40 drainage wicks (roughly 15-foot- diameter average influence zone) actively vented water and air to the surface from initial ground penetration of the probe until after the column had been constructed. Drainage wicks near the right abutment produced significantly less discharge than those at the left abutment possibly because of a lower overall piezometric surface and differences in geologic conditions (soil composition). Connectivity between the drainage wicks and stone columns was observed during the 1997 test section, but the full extent was largely unknown. Continuous reading data loggers were employed to monitor piezometric fluctuations from instruments located through the crest and downstream of the construction area during construction of selected stone columns. The "best fit" line shown in figure 5 illustrates a basic foundation response (in terms of piezometric rise) related to the construction of over 200 stone columns. A trend can generally be observed, but there is significant data scatter, which precludes a good correlation.

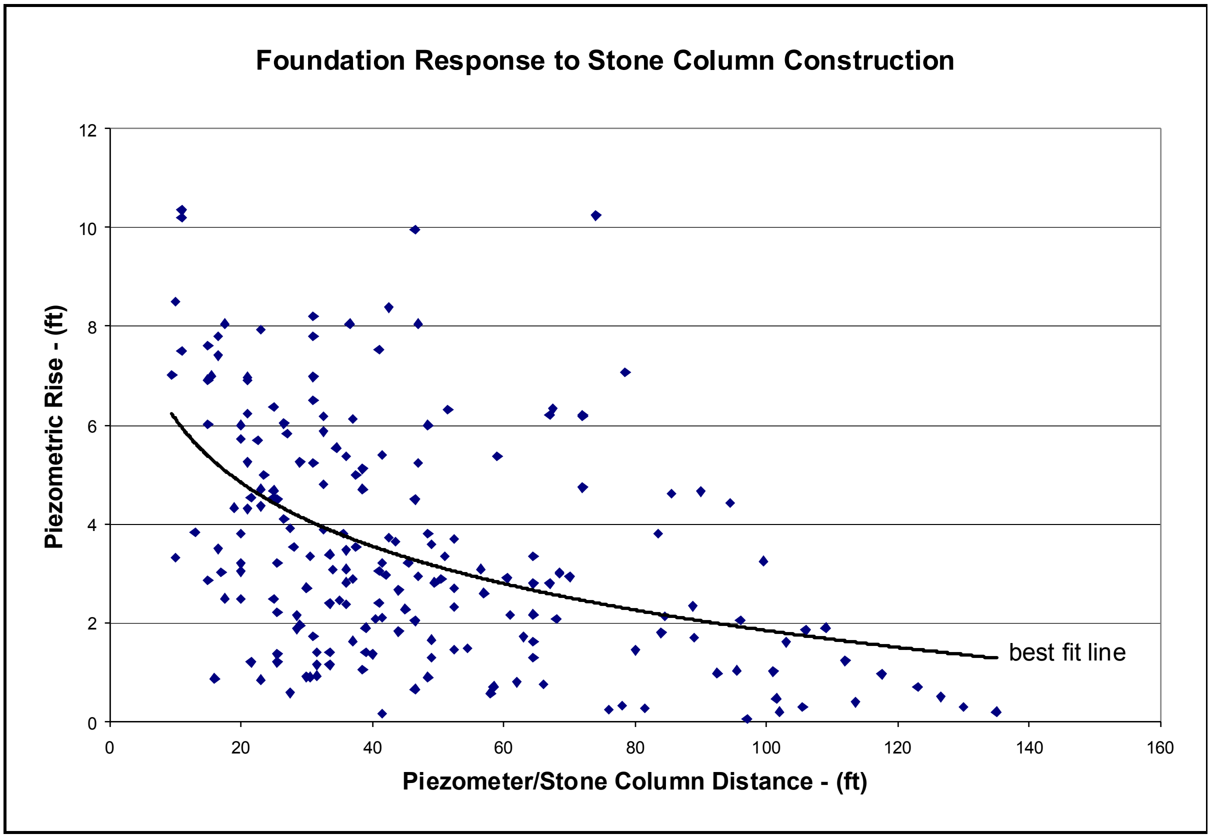

Figure 5.—Piezometric rise with piezometer/stone column distance.

The piezometric response during the construction of any given stone column is the maximum pore pressure rise observed at various distances. Pore pressure rise exceeding 10 feet was observed within wick drains closest to the stone column being constructed and up to a 2-foot rise over a distance of 100 feet. Foundation response to probe penetration and stone column construction is practically instantaneous, and pore pressure dissipation occurs concurrently with the completion of the stone column. Piezometer tip elevations are between 50 and 60 feet, which coincide with the depth of the stone column. An additional 25,300 lineal feet (466 wicks) of drainage wicks (two "lines") were installed downstream of the last row before adjusting the stone column pattern downstream one row.

Methodology for Evaluation of Liquefaction Potential

Liquefaction Evaluation Based on SPT Data

The liquefaction potential evaluation compared 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 significant percentage of materials with fines (minus No. 200), a fines correction, $\Delta N_{1(60)}$, is required and applied to the measured values. State-of-the-practice methodologies were employed which relate cyclic shear stress to required corrected blow-counts for clean sands [8, 9]. The criteria used to identify triggering was when the $N_{1(60)m}/N_{1(60)r}$ ratio $\leq$ 1.0 to 1.2.

Figure 6 compares representative pre- and post-construction SPT blowcount data $N_{1(60)m,(cs)}$ against required $N_{(60)r,(cs)}$ (liquefaction triggering threshold). Measured $N_{1(60)m,(cs)}$ values that fall to the left of the required $N_{1(60)r,(cs)}$ line indicate potential for triggering of liquefaction. The majority of hollow shapes (or pre-treatment data) lie near or to the left of the liquefaction triggering threshold line. The post-treatment data, represented by solid shapes, show significantly higher blowcounts and indicate foundation improvement by measured densification increase.

An area of significant fines (silts and silty sands) is present between elevations 2275 and 2260, whereas sandy lenses are apparent between elevations 2283 and 2280 and between elevations 2260 and 2248. The effectiveness of treatment of these areas is influenced by the fines content and is readily apparent.

Table 1 illustrates differences in foundation improvement (with material type) by comparing $(N_1)_{60}$ and $(N_1)_{60\text{-cs}}$. The percent of clay size materials (minus 0.005 m) was relatively low regardless of the soil classification. The largest strength increase was exhibited by the silty gravels followed by the silty sands, the poorly graded sands, and silts, respectively. Averaged across the site by elevation and weighted by the number of samples, the average amount of improvement $(N_1)_{60}$ was about 95 percent.

Table 1.—Pre- and post-construction SPT $(N_1)_{60cs}$ value comparison by material type

Soil type	Average percent fines[1]	Average percent clay (0.005m)[1]	Number of pre-/post-construction samples	Average Pre- $(N_1)_{60}$	Average Post- $(N_1)_{60}$	Percent increase
Silt	65	11	70/48	12	23	88
Silty sand	49	5	159/185	17	33	95
Poorly graded sand with silt (SP-SM)	10	2	51/35	21	40	92
Silty gravel with sand (GM)	12	3	4/6	15	52	236

[1] From post-construction SPT lab data only.

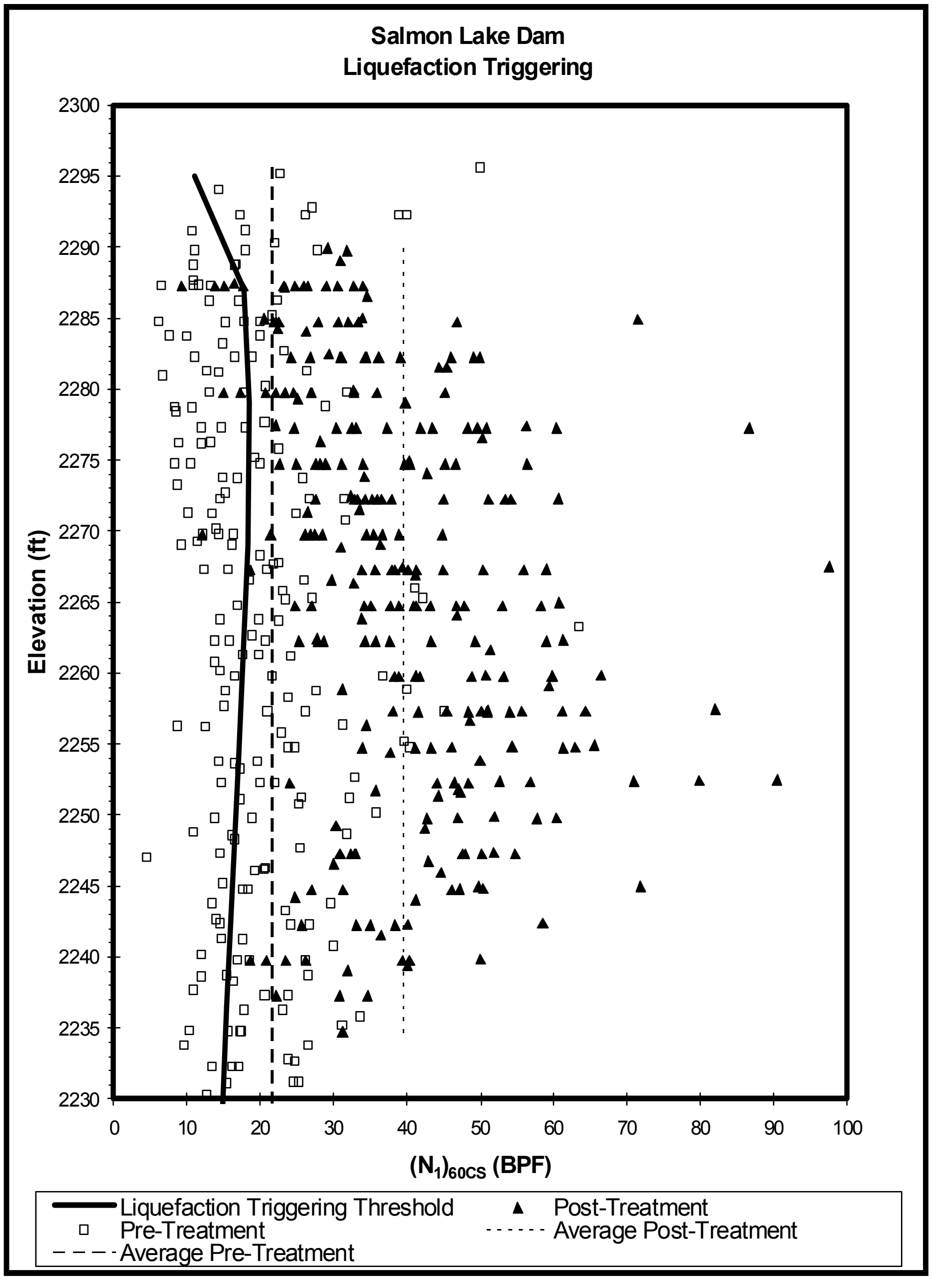

Figure 6.—Comparisons of pre- and post-construction SPT data.

CPT Methodology for Liquefaction Analysis

To evaluate liquefaction triggering in a method similar to that of the SPT, 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 [9], it is possible to correct the measured CPT penetration resistance to an equivalent clean sand value by estimating grain characteristics (apparent fines content [AFC]) directly from the CPT. However, it should be noted that estimates of the AFC from the CPT can be unreliable for determining the actual fines content in some cases and probably should not be used rigorously for this purpose. At this site, the method generally underestimates the laboratory measured fines content (figure 7).

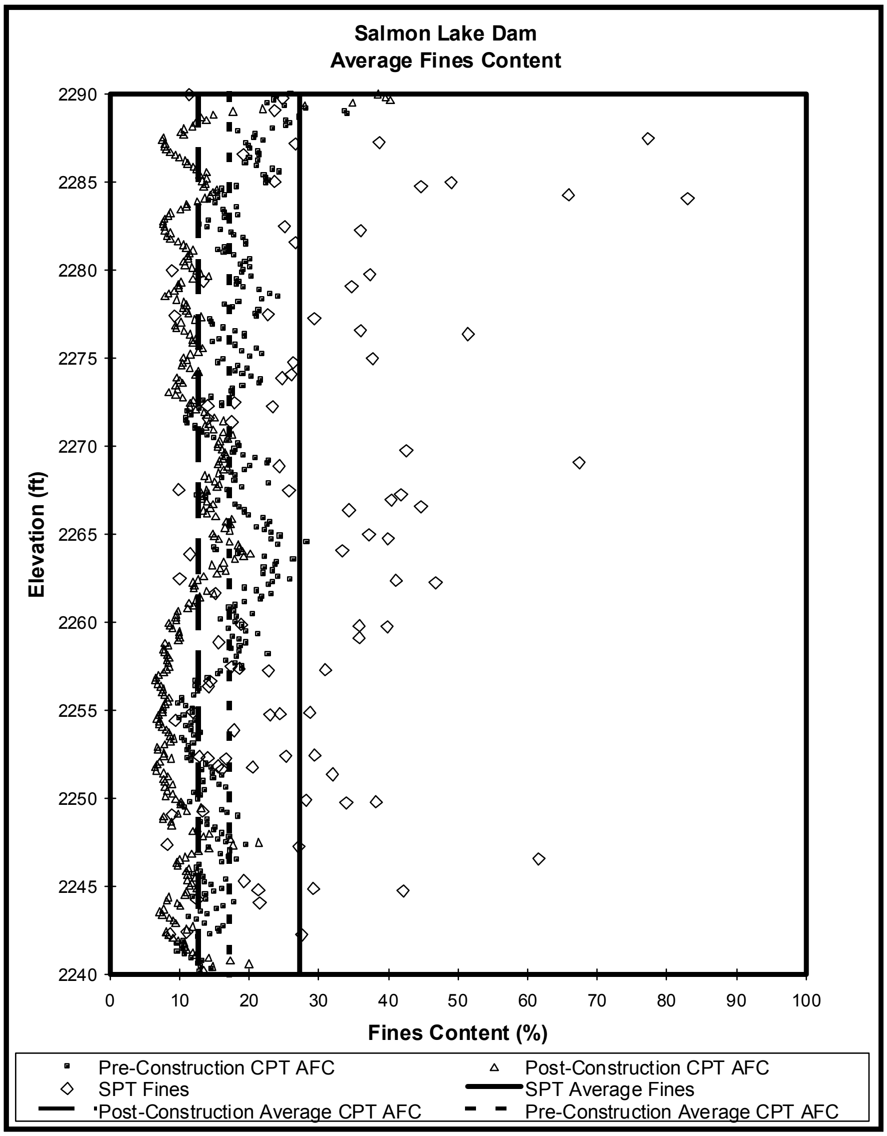

Figure 7.—CPT apparent/SPT laboratory fines content with elevation.

The CPT fines correction was computed from the equations proposed by Robertson and Wride using the CPT friction ratio. A decision was made to defer to the AFC derivation using the CPT friction ratio to maintain consistency with the liquefaction analysis method applied.

Comparisons of AFC on companion CPT holes (pre- versus post-construction) generally show a decrease in fines after stone column construction, which would imply the foundation has become coarser. Since the actual fines content of the materials is unlikely to have changed significantly during treatment, the change in AFC must be associated with changes in the treated foundation differing stress conditions and is clearly artificial [10]. This points out the difficulties in predicting the fines content from these CPT parameters.

Since the calculated AFC after treatment was used to correct the post-construction CPT tip resistance to equivalent clean-sand values, and the AFC decreased after treatment (average 5 percent), the corrected post-treatment CPT resistance values, q_{c1Ncs}, are on a relative basis biased on the conservative side. Thus, the actual level of foundation improvement is likely to be slightly higher than that inferred by comparing the pre- and post-treatment values of corrected equivalent clean-sand CPT tip resistance.

It is fortunate that the foundation improvement observed has been so significant that a difference in the measured stress condition (as reflected in the AFC) does not impact the conclusions on the adequacy of treatment (figure 8).

The level of foundation improvement can also be depicted by a comparison between average pre- and post-treatment values of normalized CPT tip resistance, q_{c1N} [10] (figure 9). Data shown is the average (at each elevation) of all applicable pre- and post-construction CPT data.

A factor which may have influenced the overall foundation treatment performance is the effect of grouping or confinement for large numbers of stone columns. According to Baez [4], "the effects of grouping (confinement) are evident in the resulting normalized penetration resistance values for the spacings evaluated." This conclusion suggests that the use of a small test section may underestimate full production foundation improvement results.

Influences of Sequence

Near-optimal foundation improvement observed during the 1997 test section was attributed to using an alternating row "advancing front" construction sequencing.

In this approach, alternate rows were constructed upstream to downstream sequentially (primary rows), with adjacent rows subsequently constructed (closure rows). Since closure sequencing in the test section appeared to have a beneficial effect in treatment, construction specifications for treating the entire foundation incorporated an additional sequencing requirement *maximizing* the closure effect. Columns were constructed initially using a "hop-scotch" pattern from the outside inward for each row (see figure 3).

At the beginning of construction, five primary rows were constructed before initiation of the closure rows followed by verification testing 10 days after completion of the closure rows. The specialty contractor voiced concern that by implementing the specified sequencing, the

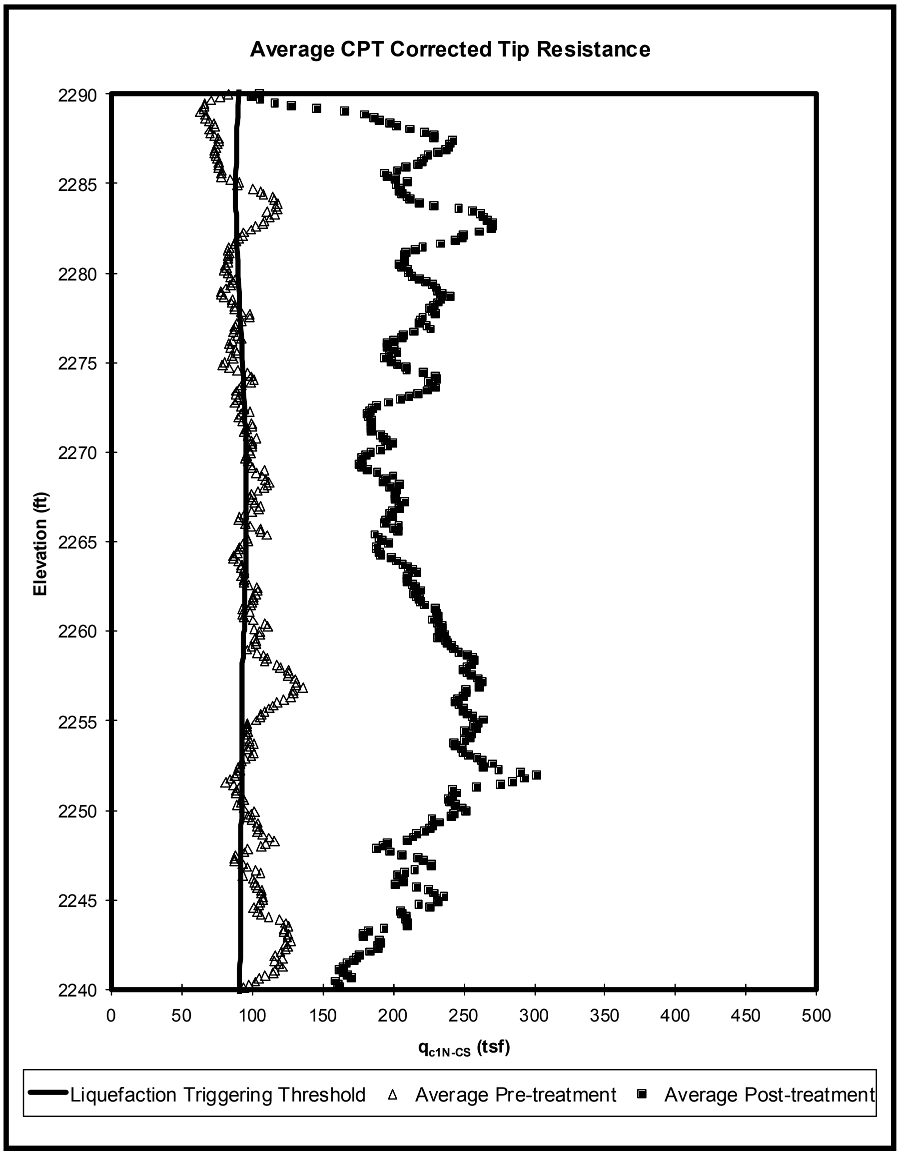

Figure 8.—Clean sand equivalent tip resistance ($q_{c1N\text{-}cs}$) with elevation.

stone column equipment was being worked beyond the maximum required effort, or 80 percent of equipment capacity. In addition, columns constructed in closure rows were more likely to be short of the specified design depth, produce a smaller average diameter column, and require less than the specified volume of stone.

A second sequencing plan was employed by constructing alternating primary and secondary rows but keeping the same internal sequencing intact. Even with the new sequencing plan, similar problems were experienced by the contractor, but to a lesser degree. Several alternative sequencing plans were proposed by the contractor with the intent to efficiently meet all of Reclamation's design/construction objectives. The final sequence methodology abandoned the use of internal "closure" within each row, and columns were constructed on an advancing front with alternating primary and closure rows (as depicted on figure 3). Full

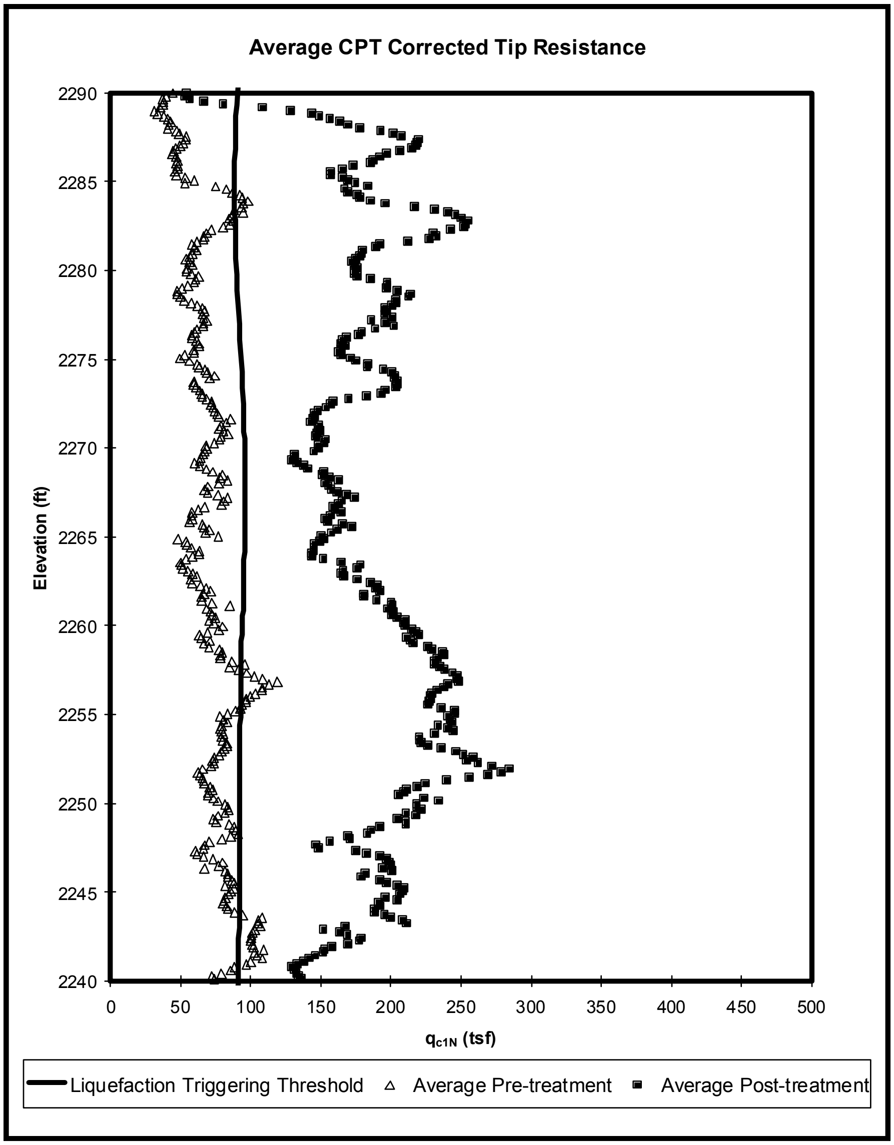

Figure 9.—Average corrected tip resistance (q_{c1N}) with elevation.

depth of both primary and closure row columns was obtained with this procedure. The closure columns generally took approximately 25 to 50 percent more time to construct than the primary row columns.

Based on the performance of the foundation to treatment, there appears to be a soil-specific densification limit that can be achieved when using vibro-stone construction for a given diameter and spacing. From an engineering standpoint, it is reasonable to attempt to approach this limit if the goal is to create a soil unit highly resistant to strength loss during a seismic loading. In an attempt to define such limits and their variance and/or distribution across the site, sequencing that maximizes the effect of closure should certainly be considered at the initiation of construction. If it can be determined that treatment is not optimally effective, sequence adjustments towards an "advancing front" can be made to ease the overworking of the foundation and the equipment. Adjustments should be made to maximize the energy imparted to the foundation with effective and optimal use of the equipment.

Conclusions

The verification testing indicates that construction of the stone columns increased the SPT blows per foot (average $(N_1)_{60}$ increase of 95 percent) and CPT penetration resistance (average increase of about 180 percent for (q_{c1N})). On the average, the penetration resistance of the treated soils is well above the threshold for liquefaction triggering during the design earthquake. In a few isolated and discontinuous soil intervals, the penetration resistance is at or near the threshold. However, the existence of these intervals is not considered detrimental to the overall seismic performance of the structure. On average, soils were treated well beyond liquefaction triggering levels.

Although a test section was initially performed to establish effective column spacing and diameter, the SOD foundation treatment performance appears to be magnified because of the effect of mass grouping or confinement.

An evaluation of foundation "aging" after 3 years of the test section construction is ongoing, and results are currently inconclusive.

The average AFC (associated with CPT tip resistance and friction ratio) decreased about 5 percent after the stone column treatment and is clearly artificial since it is unlikely that the material particle distribution between the columns actually changed during treatment. Since predicting AFC from CPT parameters can be unreliable in finer-grained soils (greater than 20 to 30 percent fines), the level of foundation improvement as depicted by the comparisons between the pre- and post-treatment values of normalized CPT tip resistance, q_{c1Ncs}, without the AFC adjustment (i.e., q_{c1N}), can be questioned.

The fact that up to 40 drainage wicks (in a roughly 15-foot-diameter area of influence) were actively venting air and water during the construction of any given stone column validates their use in protecting areas of the foundation and embankment from excess disturbance and hydraulic fracturing.

The effect of closure and proper sequencing can be highly beneficial in optimizing foundation improvement but can also present construction difficulties, especially in soils where the treatment is found to be highly effective. Maximizing closure within each row may not be warranted at this site because it appears to "overwork" the soils, resulting in excess pore pressures; unnecessarily overexercises the equipment; and increases production time. Modifying sequencing to approach an advancing front can reduce the construction difficulties while still achieving effective treatment. Future foundation treatment projects should consider construction of a strategic row or "wall" of stone columns parallel to the embankment toe and a reasonable and flexible construction sequencing strategy.

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WIRE ROPE PROTECTION AT ALTUS DAM

by Bill Bouley, Civil Engineer, Inspections and Emergency Management Group

In 1999, the Lugert-Altus Irrigation District in Altus, Oklahoma, replaced the wire ropes of the spillway radial gates at Altus Dam. When the stainless steel wire ropes were installed, they were encased to not only maintain the lubricant but also to prevent the radial gate steel skinplates from corroding as rapidly as they would if they were in direct contact with the wire ropes. The installation was made by spirally cutting a rubber water hose, which was then wrapped as an encasement around the lubricated cable. The encased cable and lubricant, which has been in place for over 2 years, is expected to increase the life of the protected parts far beyond what would normally be expected if left unprotected.

The gates can be raised up to an opening of 8 feet before the hoses come into contact with the hoist drums. During extreme flood events, the hoses can be removed from the wire ropes to allow the gates to be raised as required for flood operations.

RECLAMATION DEVELOPS NEW GENERATOR SAFETY DEVICE

by Bruce Lonnecker

The Bureau of Reclamation (Reclamation) owns and operates 194 hydroelectric generators at 58 powerplants in 11 of the Western States. These generators have a total capacity to serve about 14 million households. Hydroelectric generators have a massive rotating component, the rotor, which is connected to the turbine. The rotors in the eight generators at Glen Canyon Powerplant are typical of generators at many of the Reclamation powerplants. They are over 25 feet across and weigh over 110,000 pounds. As water is released and flows through the turbine, the rotor is turned, and electric power is generated.

A new safety device has been developed for use when maintaining Reclamation's hydroelectric generators. Reclamation developed the Generator Rotor Turning Gear, originally conceived by another utility, as a research project to improve the convenience, precision, and especially the safety of generator rotor turning operations.

Glen Canyon Plant Mechanic Rick Benzel demonstrates operation of the rotor turning gear. The rim of the generator rotor is overhead.

A potential safety hazard occurs when large, synchronous generator rotors must be turned slowly and stopped at precise positions for certain operations such as maintenance, inspection, mechanical alignment, and testing. Until now, rotors at Reclamation plants have generally been turned "by hand" or sometimes by using winches. These methods are difficult, cumbersome, and potentially dangerous. The rotor is made of steel, and its surface can be very slippery if any oil is present. Also, there are many projections on top of the rotor, such as fan blades, bolts, and nuts. When a generator rotor is turned by hand, several workers stand on the top of the rotor, brace their backs against some stationary structural component, and push with their feet on the projections, as if they were pushing a 110,000-pound stalled car. As the rotor begins to turn, the workers must reposition their feet to get a new purchase on the rotor and continue pushing.

Once turning, the inertia of the massive rotor tends to keep it in motion. It is not possible for a worker or even several workers to quickly stop the rotation of the rotor. Depending on its speed, the rotor may turn several times before it coasts to a stop. While moving, the rotor is a source of hazardous energy.

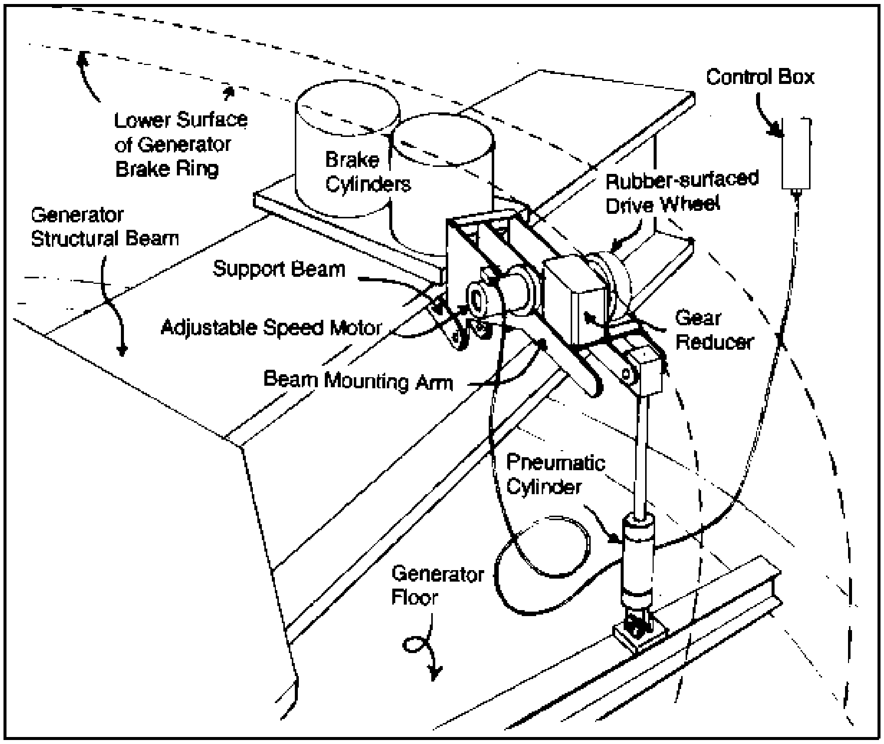

Diagram of the rotor turning gear.

The hazard is that a worker may be trapped or entangled with components of the turning rotor. The worker could slip or get into a position to be pinched or dragged by the rotor as it continues to turn. A worker at another utility died a few years ago in such an incident. Installation of the rotor turning gear eliminates this hazard.

The basic construction of the device incorporates a rubber wheel that is pressed up on the generator brake ring by a pneumatic cylinder. The wheel is turned by an adjustable speed motor. With this device, one worker can control the adjustable speed motor, and thereby the turning of the rotor, from a safe position within or near to the generator. Most importantly, the turning of the rotor can be stopped quickly. The rotor turning gear rotates down and out of the way during normal generator operation.

The first Reclamation rotor turning gear has been installed and successfully operated on a Glen Canyon generator. It was demonstrated to safely rotate the generator rotor and then quickly stop rotation. The rotor turning gear from one machine can be moved and used on any identical machine in the plant.

It is expected that the rotor turning gear will be adapted to many of Reclamation's hydroelectric generators to improve safety and make maintenance operations more convenient. For more information, please call Reclamation's Hydroelectric Research and Technical Services Group at (303) 445-2300.

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.

The purpose of this bulletin is to serve as a medium of exchanging operation and maintenance information. Its success depends upon your help in obtaining and submitting new and useful operation and maintenance ideas.

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