Use of Deep Blast Densification for Bridge Foundation Improvement on SR-504

DAVID V. JENKINS, ALAN P. KILIAN, AND JOSEPH E. HACHEY

A case history describing the use of blast densification by the Washington State Department of Transportation to densify a 40-m-deep, loose debris flow is presented. Debris flow from the 1980 eruption of Mount St. Helens would pose a high risk for liquefaction and ground settlement should a seismic event occur. A single-span bridge was to be constructed on the debris flow. It was determined that the only practical means of supporting the structure were spread footings founded on the debris flow, once improved by ground densification. Blast densification was chosen over more common means to improve the ground; it was considered the most cost-effective and feasible method of construction through boulder-laden debris flow. First, a test section was constructed to verify the blast design and to confirm its feasibility given the unusual geologic deposit. The goal was to improve the relative density of the deposit, as measured by standard penetration testing (SPT) and Becker penetration testing. Additionally, the site was instrumented to measure ground response. Instrumentation included surface and subsurface settlement devices, inclinometers, piezometers, ground-vibration instruments, and geophysical surveys. Blast densification successfully increased the SPT values of the deposit from an average $N_{1(60)}$ = 8 to $N_{1(60)}$ = 20 above 15 m, to $N_{1(60)}$ = 19 below 15 m.

The case history presented in this paper describes the design, construction, and test results of a deep-soil densification project. The project used explosives at approach-fill areas that were chosen for a new bridge structure. The densification work was performed by the Washington State Department of Transportation (WSDOT) as part of an 11-km SR-504 extension into the Mount St. Helens National Volcanic Monument; it was part the work to be performed in construction of Bridge No. 12 across South Coldwater Creek.

The Mount St. Helens National Volcanic Monument is situated in Cowlitz and Skamania counties in southwestern Washington State. The new 11-km extension begins at the outlet of Coldwater Lake at an elevation of 730 m and traverses eastward, crossing South Coldwater Creek. It then enters the South Coldwater Creek Valley, which was filled with up to 40 m of debris from the 1980 eruption of Mount St. Helens. South Coldwater Creek is bordered by the Coldwater Divide to the north and Johnston Ridge to the south. The new alignment will end near the summit of Johnston Ridge at an elevation of 1,400 m.

A blast densification project was used to mitigate the potential for liquefaction and dynamic settlement of the approach fills and bridge abutment footings at the new South Coldwater Creek bridge. Blast densification uses the shock and vibration resulting from the detonation of an explosive, aided by the weight of overlying soils, to rearrange soil particles into a denser state.

D. Jenkins, Washington State Department of Transportation, Headquarters Materials Laboratory, P.O. Box 47365, Olympia, Wash. 98504–7365; A. Kilian, FHWA, Western Federal Lands Highway Division, 610 East 5th Street, Vancouver, Wash. 98661; J. Hachey, Golder Associates Inc., 4104–148th Ave., N.E., Redmond, Wash. 98052.

SITE DESCRIPTION

The 1980 eruption of Mount St. Helens triggered a rockslide or debris avalanche and related lateral blast that devastated approximately 325 sq km^2 of ground north of Mount St. Helens (*1*). The north fork of the debris avalanche deposits formed blockages at the outlets to Coldwater Creek and South Coldwater Creek damming lakes with avalanche debris.

The new bridge structure that will span South Coldwater Creek will consist of a 60-m-long, single span, steel-plate girder bridge supported on low-capacity spread footings founded in debris avalanche deposits from the 1980 eruption.

When four borings were drilled during a foundation investigation of the bridge structure, WSDOT encountered loose-debris avalanche, consisting of a multicolored, heterogeneous mixture of sand and gravel with varying amounts of silt, cobbles, and boulders to depths in excess of 40 m. Corrected standard penetration test [SPT, $N_{1(60)}$] blowcounts were typically 8 or less. Pre-1980 deposits below the 40 m depth consisted of dense to very dense, nonstratified, fine to coarse sand, some gravel, and some silt.

Groundwater levels corresponded roughly with the level of water in South Coldwater Creek, which at the bridge site ranges from 2 to 5 m below the existing ground surface.

SEISMIC CONSIDERATIONS

Mount St. Helens Seismic Zone (*2*) is an interpreted 100-km-long, near vertical, right-lateral, strike-slip active fault zone. The zone trends north-northwest through the WSDOT project area. The maximum magnitude recorded for an earthquake in the zone is 5.5 on the Richter scale, for the earthquake measured on February 14, 1981. Its epicenter was near Elk Lake, approximately 5.2 km north of South Coldwater Creek.

Crustal earthquakes (3.3 to 16.6-km deep) greater than the 5.5-magnitude event are possible along the Mount St. Helens Seismic Zone (*3*). WSDOT designed the bridge to withstand a seismic event with a magnitude of 6.5, generating a 0.55 g peak bedrock acceleration. Because the soils at the project site are granular, loose, and saturated, liquefaction was potentially a high risk to the stability of the structure.

Liquefaction analyses were performed based on the SPT data and procedures developed by Seed et al. (*4*) and indicate that about two-thirds of the SPT results fall within the range where liquefaction is a moderate to high risk.

A major liquefaction failure in the vicinity of the bridge could significantly affect a large area. Such a failure could include loss of both vertical and lateral foundation ground support for the bridge, ground subsidence, and lateral spreading. Lateral spreading would be particularly damaging because it probably would displace the bridge laterally even if it were supported on deep piles. Localized liquefaction could induce differential settlement and possibly cause lateral movements that could damage the bridge.

Even if liquefaction did not occur, a seismic event would be likely to induce dynamic settlement of the loose debris avalanche deposit. Resulting ground settlement would cause unacceptable movement of the bridge.

Stability analyses indicate that ground improvement must be full depth (40 m) and conducted over the entire plan area of the approach fill in order to lessen significantly the probability of a deep-seated failure, the objective being to essentially create an "island" of stable soil. To protect against the maximum design event, the upper 15 m requires an $N_{1(60)}$ value of about 25, whereas below 15 m an $N_{1(60)}$ value of 20 is required. These $N_{1(60)}$ values correspond to relative density values of approximately 65 percent in the upper 15 m and approximately 55 percent below 15 m.

FOUNDATION OPTIONS AND DENSIFICATION METHODS

Foundation options were evaluated principally on the basis of seismic risk, cost, and constructability. Because of the extensive boring depth and loose nature of the site's soils, several significant design issues had to be addressed, including foundation support, area and foundation settlement, liquefaction potential, seismically induced settlement, and the advantages of ground modification.

Both shallow and deep foundation-support systems were considered. The site soils generally were not suitable for spread-footing support of a bridge. Yet deep foundation systems, such as driven piles or drilled shafts, would have to deal with downdrag forces from static or dynamic settlement of the recent debris avalanche deposit, lateral load and lateral spreading caused by liquefaction of the deposit, and construction problems related to the presence of boulders in the debris flow.

Static settlement of the foundation soils had a potential impact on the foundation system. Because of the young age of the debris avalanche deposit, its loose saturated nature, and the effects of buried organics, the deposit could still be undergoing natural settlement. This could cause downdrag loads on deep foundations and differential settlement between structure elements. Applying foundation loads and approach-fill loads to this nonuniform deposit could result in unacceptable settlement.

Based on the design earthquake, liquefaction potential was determined to be a high risk. Liquefaction would result in loss of foundation support, lateral spreading, and ground subsidence. All these liquefaction effects were unacceptable for the bridge design. Perhaps liquefaction would not occur at full depth, 40 m, at the site. Yet dynamic settlement of the deposit would likely result in large settlement at the site. In the case of a modest event, the settlement might be 0.2 m; if a large event were to occur, it could be a few meters. Seismically induced ground settlements were considered a controlling design constraint.

Considering the unique nature of the deposit and the need to keep costs in line for a moderate-sized bridge's ground modification, techniques to improve density and strength of soil became important considerations. The benefits of the right technique would be to allow the use of cost-effective shallow foundations and to reduce the risk of seismically induced liquefaction and ground settlement.

Numerous methods are available to improve the density and strength of a loose debris avalanche deposit, including deep dynamic compaction, vibro-compaction, stone columns, deep soil mixing, jet grouting, and blast densification. These methods were viable alternatives, but constructability risks associated with the presence of bouldery soil and related costs made blasting the preferred option.

Densification of granular soils requires first that the original soil structure be broken down so that soil particles can be moved to a new packing arrangement. In saturated, cohesionless materials this is accomplished most readily by inducing liquefaction using dynamic and cyclic loading. In the case of blasting or dynamic compaction, the compression wave generated by the sudden large energy release can give an immediate buildup in pore water pressure, which greatly reduces the shear strength. The compression wave is immediately followed by a shear wave that is responsible for failure of the soil mass. Passage of these two waves ultimately results in the soil particles settling into a denser, more stable position.

DENSIFICATION DESIGN

Densification by blasting differs from normal construction practices in that it has had limited usage, even though documented use of blast densification can be traced back 50 years. Reluctance to use blast densification relates to the lack of a theoretical design basis. Blast design is empirical, based on prior experience which is modified by site trials. To date, the Jebba project in Nigeria (*5*) was the only project documented to have used blasting to a similar depth, that is, in excess of 35 m. Theoretically, there does not appear to be any restriction on the depth of densification achievable.

An advantage of blasting at the WSDOT site was that the problem of the bouldery soil at the site was handled easily with the construction installation methods WSDOT used. Holes were advanced using the Becker Hammer, which experienced little difficulty in penetrating this deposit. The truck-mounted HAV-180 Becker Hammer Drill consists of a double-acting diesel hammer driving a double-walled casing into the ground.

The design of charge spacing and size was empirical, based on data from available case histories. This design was significantly influenced by the blast densification program conducted at the Molikpaq caisson-retained island in the Canadian Beaufort Sea (*6,7*). The Molikpaq data indicated that the maximum densification was achieved within about 3 m above and below the center of a given charge. Based on these results WSDOT decided to space charges at a nominal vertical spacing of about 6 m and locate the first charge about 1.5 m below the water table. Consequently, the charges were placed at depths of 5 m, 11 m, 17 m, 23 m, 29 m and 37 m below the ground surface. The spacing between the bottom two charges was increased to 8 m to allow densification to about 40 m.

The lateral spacing of charges was controlled for the most part by three factors:

1. The need to minimize the total number of holes to be drilled;

2. The decision to use a "two-pass" approach, which is the common approach at most other blasting sites; and

3. The desire to stay within the 5- to 15-m guideline for charge spacing that Mitchell presented (*8*).

The two-pass approach charges are laid out in a pair of superimposed grids. Each grid has the charges laid out in equidistant rows, with the charges for the second grid placed in the centers of the squares formed by the rows of the first grid (Figure 1). The spacing between rows was 5.3 m (resulting in a spacing of 10.5 m between rows in a single pass). The first grid is detonated in the first pass, and the second grid is detonated in the second pass.

The proposed area of densification consisted of two areas approximately 45 m by 25 m each. Using the two-pass approach design resulted in three rows in the first pass, followed by two rows in the second pass and an effective spacing between blast holes of 7.5 m.

The charge sizes were designed based on past experience where the powder factor was between 15 and 25 grams of explosive per cubic meter of treated soil. There was also concern about the potential for "cratering" and the potential for triggering slope failures in the adjacent slopes. Beginning from the top deck down, the initial plan called for 6 decks with 2.3 kg at 5 m, 4.5 kg at 11 m, 6.8 kg at 17 m, 9 kg at 23 m, 11 kg at 29 m, and 13.6 kg at 37 m. This resulted in a powder factor of approximately 15 g/m^3. The term "powder factor" means the mass of explosive used divided by the total volume of soil improved by blasting in one blast sequence or "pass."

The intent of blast densification is to produce settlement by inducing liquefaction. During earthquakes, liquefaction results from cyclic loading of the soil, and for a given soil density, the occurrence of liquefaction depends upon the magnitude of the cyclic load and the number of cycles experienced by the soil. There were two timing-design options available for testing whether the blast design would accomplish liquefaction. The first option was to detonate all of the charges at once, to increase the magnitude of the load at the expense of the number of cycles. The second option was to detonate a smaller number of charges at any one time and induce a greater number of cycles at the expense of reducing the magnitude of the loads. It was decided to use delays to create a larger number of cyclic loads. No case histories were found in the literature wherein the primary focus of the study was to evaluate the effects of blast densification by varying the delays between charges. WSDOT decided to use delays between charges, as charges were fired one row and one deck at a time, from the bottom deck up, with a 75-msec delay between rows and a 0.4 sec delay between decks.

Soil densification by inducing liquefaction requires the concurrent removal of water. To aid water removal, vertical drains were installed equidistant from the blast holes. The drains consisted of 76-mm diameter, Schedule 40 polyvinylecloride, with 3.0-mm-sized slots.

BLAST DENSIFICATION CONTRACTING

The technical objectives were to densify the soil at full depth and an area large enough to create a "stable island" that would withstand strong ground-shaking. Improvement in densification was measured by means of the SPT as an indicator. The goal was to increase the average SPT value [$N_{1(60)}$] from about 8 to 25 in the upper 15 m of ground and increase it to 20 below a depth of 15 m.

Using relatively new technology creates a lot of uncertainty when contracting. Consequently, one objective was to share the risk of the project by not including the explicit SPT blow counts in the contract. Also, an advisory specification was included in the contract describing the interpreted geologic conditions and expected difficulty in drilling the bouldery deposit.

An additional project constraint was the requirement to minimize damage to the surrounding terrain. The project is within the Mount St. Helens National Monument, and the existing topogra-

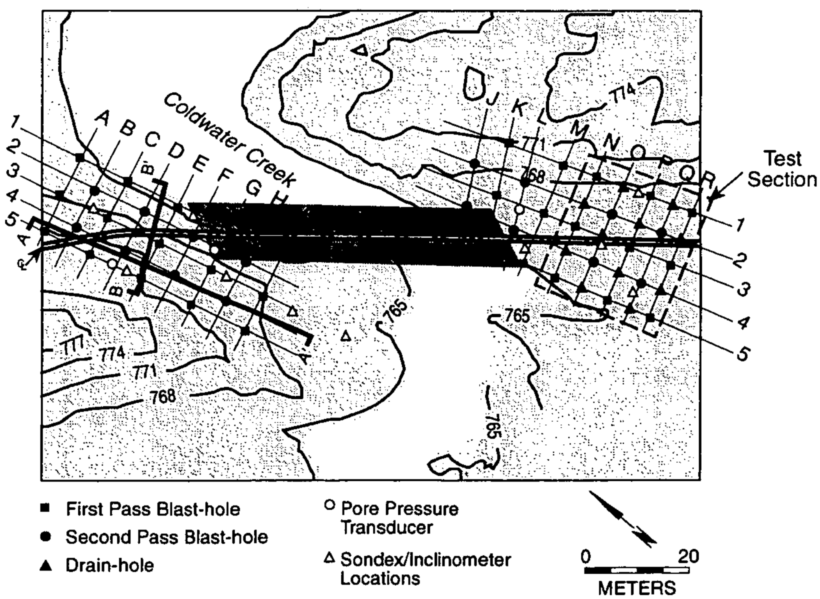

FIGURE 1 Blast-hole and instrumentation location plan.

phy is valued highly. The mandate was to do an absolute minimum amount of damage to the topography outside of the planned roadway.

The first step was to speed up the ground densification portion of the major highway project. This would allow time to evaluate the success of densification and not to risk using a new construction technique on the major project before making any needed change to the blasting plan.

A workable contracting method would include a prequalification requirement for contractors. The contract would specify a base program in terms of number of holes and spacing, construction sequencing, energy and blast depth. The contractor would bid on the base program with unit-price add/deducts for the actual program implemented. The base program would also include an initial "test section" phase that would involve varying selected procedures during the initial phase of work. In the unlikely event that the method was found to be unsuitable, the contract would provide for equitable, early termination of the work. Finally, the contract would specify the types of construction instrumentation required to control and monitor the densification effort.

The program did not specify drilling method or explosive type, but left selected details to the contractor. The actual production blasting program used was chosen by WSDOT based on the results of the "test section" phase. The bid items were intended to be flexible enough to provide for the actual program being implemented including any changes.

The blast densification contract consisted of three phases. Phase 1 consisted of drilling and blasting in a test section that amounted to approximately one-fourth of the total blast area proposed. Phase 2 was a 1-week evaluation during which WSDOT would study the results of the blasting in the test section. This phase also allowed WSDOT the option to cancel the contract or to proceed with the remainder of the densification program with possible modifications to the blast plan. Phase 3, the production phase, would be implemented to blast densify the remaining 75 percent of the proposed blast area.

Test Section Phase

Drilling for the test section took place between October 30, 1992, and November 8, 1992, and was conducted by Foundex Inc. of Bellingham, Washington. Instrumentation consisted of surface settlement hubs, two sondex casings installed to a depth of 38 m. Sondex rings were placed every meter at full depth, and borros anchors at 20 and 28 m below the ground surface.

During installation of the 40-m-deep vertical drains, it was observed that significant siltation was occurring within the drain pipes several days after installation. As much as 10 to 25 m of silt and fine sand was deposited in each of the drain holes. There was some discussion as to whether the slot size of the drains should be reduced to decrease siltation. It was thought that if the slot size was reduced significantly to prevent siltation, the drain slot would then be too small to move water effectively. Also, the silt in the drains probably was sufficiently loose that it would be dislodged by the fluid pressure generated during blasting.

The first pass of blasting in the test section consisted of detonating a total of nine blast holes in a 3-by-3 array. A 0.4-sec delay was used between the 6 decks and a 0.75-msec delay between rows. Nitropel, which is a pelletized form of TNT, was used as the explosive charge.

Surface settlement from the first blast averaged about 0.28 m within the blast zone. It was anticipated that there would be 1 to 1.5 m of settlement from the two passes of blasting. There did not appear to be any signs of cratering from the blast, nor were there any signs of large slope movements in the adjacent slopes. Minor slope movement had occurred as evidenced by the development of several tension cracks. On the basis of these results, it was decided that larger charges were warranted, but that the top charge would remain at 11 kg. The new charge profile consisted of 2.3 kg, 9 kg, 11.4 kg, 15.9 kg, 15.9 kg, and 27.3 kg at the 5-m, 11-m, 17-m, 23-m, 29-m and 37-m levels. This increase in charge resulted in a powder factor of $25 g/m^3$.

The new blast profile was used for the second pass at the test section. A total of four blast holes on a 2-by-2 array were detonated. Settlements from the second pass averaged about 0.21 m for a total settlement in the blast zone of 0.49 m. Again, there was little evidence of cratering, and there were no large slope movements. The fact that almost the same amount of settlement was achieved on the second pass, despite the fact that fewer blast holes were used and the ground was already somewhat denser from settlement after the first pass, confirmed the larger charge sizes were warranted.

Evaluation Phase

The contract allowed for a 1-week evaluation period during which a decision would be made about whether to proceed with the remainder of the densification program and potential modifications to the contract blast plan could be examined. The contract had been bid on a unit-price basis that gave WSDOT flexibility to alter quantities and procedures. On the basis of the results of the test section, WSDOT elected to proceed with the contract, but developed modifications to the blast plan that would be incorporated into the blast plan in Phase 3, the production blasting.

The following modifications to the blast plan were made:

- The charge profile used in the second pass of the test section was used for the production blasting. This resulted in a powder factor of $25 g/m^3$.
- The vertical drains were deleted. Visual observations indicated that the blast-holes drained more water than the vertical drains and that sand boils developed in areas where there were no drains or blast-holes.
- The 75-msec delay between rows was deleted and the 0.4-sec delay between decks was reduced to 0.3 sec. It was postulated that damping at the site could have reduced vibration levels more than anticipated and that this could also have reduced settlement.

Production Phase

Drilling at the site resumed on November 23, 1992, and blasting was completed on December 15, 1992. The blast densification resulted in vertical settlements of up to 1.5 m and significant increases in liquefaction resistance, as measured by SPT and BPT results.

TESTING AND INSTRUMENTATION

Instrumentation and testing were conducted as part of the blast densification project. Instrumentation locations are shown in Fig-

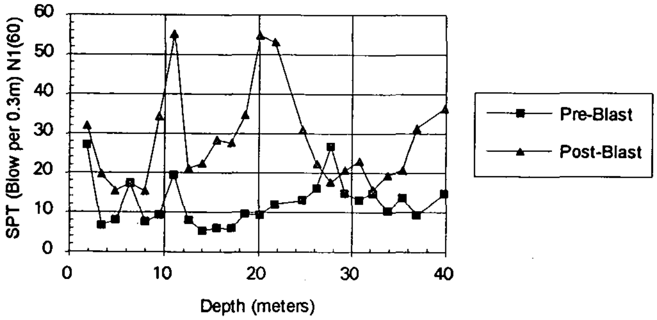

FIGURE 2 SPT pre- and post-blast penetration data (location D2).

ure 1. Penetration test location descriptions (e.g., D2) refer to the intersection of grid lines as shown in Figure 1. A location designation of L3/M4 indicates a location that is approximately midway between grid points L3 and M4.

Penetration Testing

Two types of penetration tests were performed on this project: SPT and Foundex mudded Becker penetration tests (FBPT). Results of SPT and FBPT testing at Pier 1 (D2) are summarized in Figures 2–4.

Standard penetration testing was conducted in accordance with ASTM D-1586, Penetration Test and Split-Barrel Sampling of Soils. Energy transfer was found to average about 43 percent of theoretical during testing, resulting in a 28 percent reduction of SPT N-values during normalization.

The preblast SPT data from the four boreholes drilled in 1991 during the foundation investigation of the bridge were used to compare with the postblast SPT testing conducted in January 1993. The SPT results for Pier 1 (site D2) are shown in Figure 2. There is significant scatter in the SPT results however, and many of the higher blow counts may have been affected by gravel. The presence of gravel reduces confidence in the SPT tests; however, the difference between 1991 and 1993 results clearly indicate a significant increase in density.

The BPT is similar in concept to SPT, and correlations between the tests have been published by various authors (*9*). The major differences between the tests are the tip diameter and the fact that skin friction increases with depth in the BPT, as the casing extends the full depth of the hole. BPT casing used in this test was 168 mm in diameter, driven closed end. The scale of the BPT has a significant benefit in coarse soil deposits, as the BPT results are less influenced by the presence of gravel particles; however, the

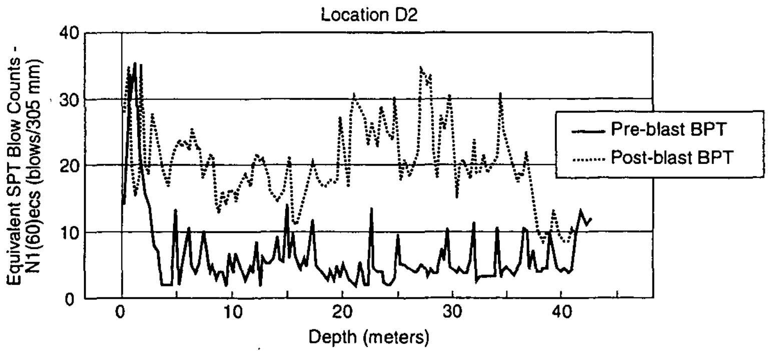

FIGURE 3 Pre- and post-blast penetration data.

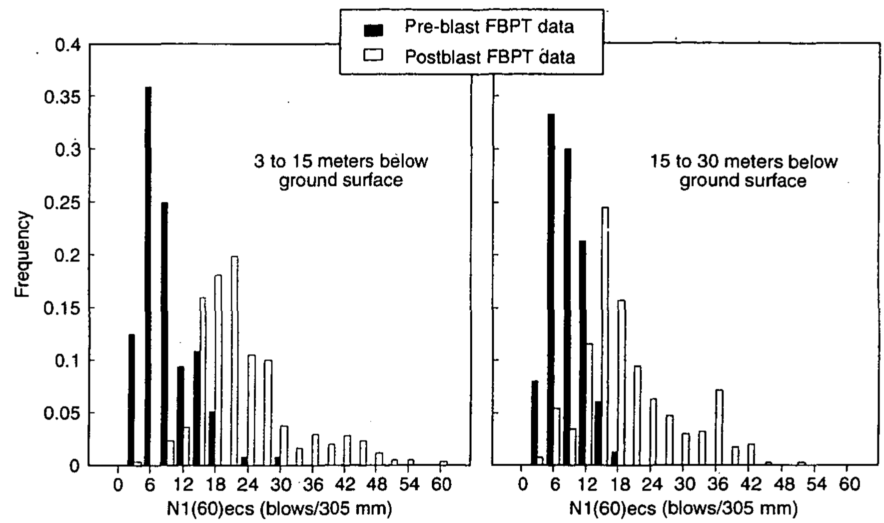

FIGURE 4 Distribution of $N_{1(60)ecs}$ values.

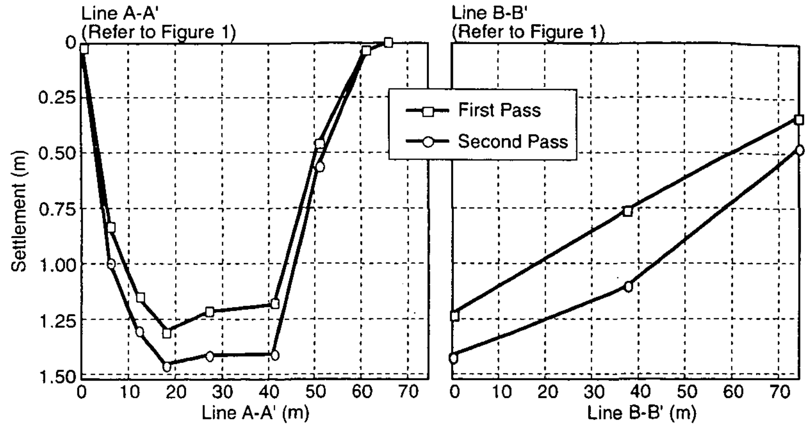

FIGURE 5 Ground settlement, west abutment.

increase in skin friction with depth makes SPT/BPT correlations less reliable below about 13 m in depth. The FBPT was developed by Foundex Inc. to reduce the problems associated with skin friction in developing SPT/BPT correlations.

A study by Foundex (*10*) for the Canadian government showed that the FBPT showed better correlations to the SPT than did the standard BPT, and the correlation was not significantly affected by depth below the ground surface.

The contract required three series of tests composed of four FBPTs in each series. The first series of FBPTs was conducted before blasting to develop a preblast baseline data base. The second series of FBPTs was done approximately 3 weeks after production blasting was complete. The last series of FBPTs was completed approximately 4 months after blasting to study the affects of blast aging.

Figure 3 shows a typical pair of replicated BPTs from the west abutment (site D2), conducted before and after blasting. The blow counts are presented as $N_{1(60)cs}$ values. The Liao and Whitman (*11*) method was used to correct for overburden stress, and a silt-content correction of 2 blows/0.3 m was used. It is obvious that there has been a significant increase in penetration resistance over the entire depth of the deposit. Figure 4 presents all of the FBPT data in the form of histograms showing penetration resistance. The histograms reveal that loose zones still exist within the debris flow, but the average blow count has increased by about 12 blows/0.3 m.

FBPT testing was conducted over a 4-month period to evaluate the effects of aging and it did not indicate a significant increase in penetration resistance. It was concluded, therefore, that the full affects of aging occurred within 3 weeks after blasting.

Settlement Measurements

Surface settlement measurements were conducted using wood survey hubs and settlement plates. The steel plates consisted of 0.09-m^2 plates buried 0.3 m below the ground surface, and a steel post extending above the ground surface to serve as a survey stake. Settlement cross sections at the west abutment are shown in Figure 5. The settlement data indicate vertical strains of about 4 percent.

Subsurface settlement monitoring was considered to be important for this project because of the depth of the zone of loose materials. An important issue was whether liquefaction could occur below 15 m and, if not, whether densification was required below this depth. Deep settlement devices were installed to determine where settlement was occurring and whether it was occurring uniformly. This monitoring was of particular importance during the test phase of the project, during which final decisions on the blast plan were to be developed.

The Sondex tube is a corrugated plastic pipe capable of compressing or extending in length during ground movement. Steel rings are placed in the groves of the pipe at 1-m intervals. The steel rings can be detected by a probe lowered down the center of the pipe, to determine their elevations. Sondex casing was the preferred deep-settlement instrument because it provides a number of measurement points in a single bore hole and can be used to obtain a settlement profile with depth.

Sondex data (Figure 6) indicated that the vertical strain was fairly uniform with depth, as indicated by a fairly linear data plot on the graph. The settlement remained uniform, even when surface settlements of 1.5 m were achieved.

Slope Inclinometers

Slope inclinometers were installed with the Sondex casing at selected locations within and outside the perimeter of the blast zone.

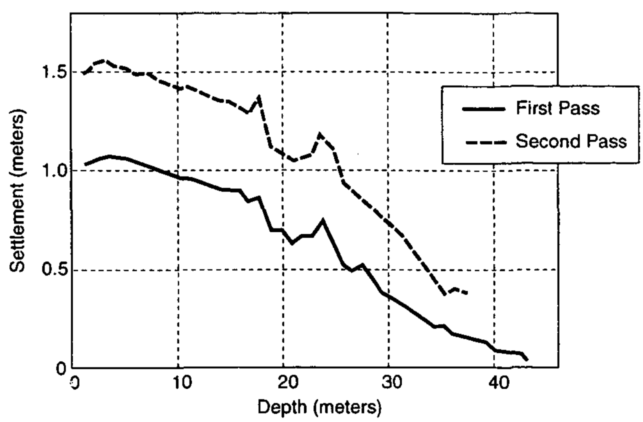

FIGURE 6 Settlement results—Sondex D/E5, west abutment.

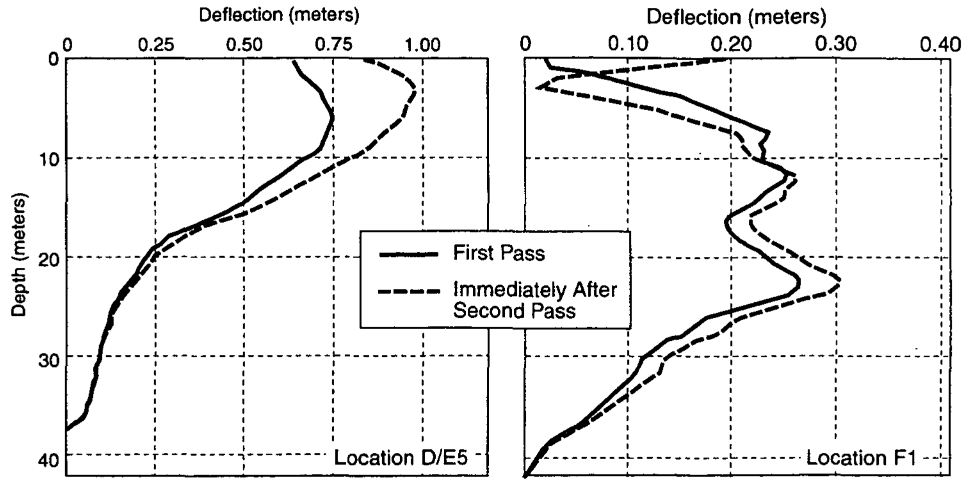

FIGURE 7 Slope indicator results, west abutment.

The most interesting data from the slope inclinometers came from the west abutment. The inclinometer located in the vicinity of D/E5 showed up to 1 m of lateral movement inward toward the blast zone (Figure 7). The slope inclinometer in the vicinity of F1, which is approximately 180 degrees from the inclinometer at D/E5, showed about 0.25 m of movement toward the center of the blast zone. These inclinometers are roughly 15 m apart. Thus the average lateral compressive strain was about 4 percent. When added to the vertical strain, this resulted in a total volumetric compressive strain of about 8 percent.

Pore Pressure Measurements

Series of four pore-pressure transducers located at various depths were installed at three locations in the blast area. The transducers were located at depths of 14 m, 20 m, 26 m, and 35 m. Figure 8 shows pore-pressure measurements from a piezometer group on the west abutment. Complete liquefaction appears to have occurred at all depths, as indicated by normalized pore pressure, *Ru*, values of unity shown in Figure 9. Pore-pressure dissipation to near static conditions was complete 24 hr after blasting.

Physical manifestation of the pore pressure was evidenced by sand boils and a high volume of water migrating to the surface approximately 30 min after detonation. Water continued to flow to the surface several hours after blasting.

Shear-Wave Velocity Survey

Down-hole shear-wave velocity, *Vs*, tests were conducted by Palmer before and after blasting was complete (*12*). Results from the survey indicate the *Vs* in the upper 6 m did not change after blasting from its nominal value of 150 m/s. *Vs* between 6 and 12 m increased from 161 m/s to a post-blast value of 247 m/s. *Vs* between 12 and 24 meters increased from 213 m/s to a post-blast value of 253 m/s. Below 24 m, no significant increase in shear-wave velocity was measured.

SUMMARY

The WSDOT project showed ground densification using blasting could improve soil density sufficiently to mitigate the high liquefaction potential at the test site and the probability of extreme ground settlement if there were a seismic event. The site of the Mount St. Helens National Monument was improved enough to support a bridge structure on spread footings.

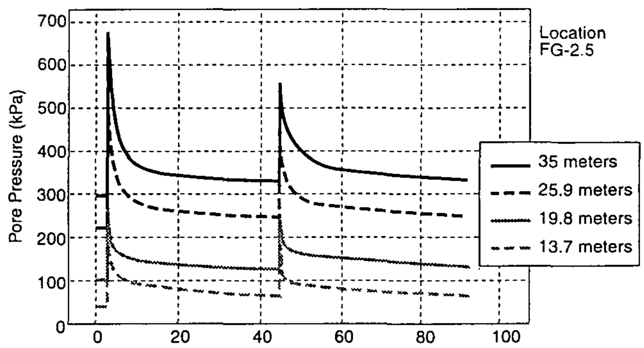

FIGURE 8 Pore water pressure, west abutment.

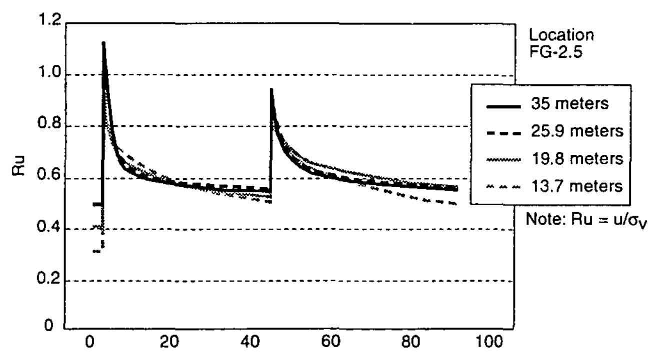

FIGURE 9 Normalized pore water pressure, west abutment.

REFERENCES

1. Voight, B., H. Glicken, R. J. Janda, and P. M. Douglass. Catastrophic Rockslide Avalanche of May 18. In The 1980 Eruptions of Mount St. Helens, Washington, (P. W. Lipman and D. R. Mullineaux, eds.), U.S. Geological Survey Professional Paper 1250, Reston, Va., 1981.
2. Weaver, C. S. and S. W. Smith. Regional Tectonic and Earthquake Hazard Implication of the Crustal Fault Zone in Southwestern Washington. *Journal of Geophysical Research*, 1983.
3. Meyer, W., M. A. Sabol, H. X. Glicken, and B. Voight. The Effects of Groundwater, Slope Stability, and Seismic Hazard on the Stability of the South Fork Castle Creek Blockage in the Mount St. Helens Area, Washington. U.S. Geological Survey Professional Paper 1345, Reston, Va., 1985.
4. Seed, H. B., I. M. Idriss, and I. Arango. Evaluation of Liquefaction Performance Using Field Performance Data. *Journal of Geotechnical Engineering*, ASCE, Vol. 109, No. 3, March 1983.
5. Solymar, Z. V. Compaction of Alluvial Sands by Deep Blasting. *Canadian Geotechnical Journal*, Vol. 21, 1984.
6. Rogers, B. T., C. A. Graham, and M. C. Jefferies. Compaction of Hydraulic Sand in Molikpaq Core. *Proc., 43rd Canadian Geotechnical Conference*, Quebec City, Quebec, Canada, Oct. 10–12, 1990.
7. Stewart, H. R. and W. E. Hodge. Molikpaq Core Densification with Explosives at Amauligak F-24. *Proc., 20th Offshore Technology Conference*, Houston, Tex., May 2–5, 1988.
8. Mitchell, J. K. Soil Improvement, State-of-the-Art Report. *Proc., 10th International Conference on Soil Mechanics and Foundation Engineering*. Stockholm, Sweden, 1981, pp. 509–565.
9. Harder, L. F. and H. B. Seed. Determination of Penetration Resistance for Coarse-Grained Soils Using the Becker Hammer Drill. Earthquake Engineering Research Center Report No. UCB/EERC-86-06, Berkeley, Calif. 1986.
10. *A Testing Technique for Earthquake Liquefaction Prediction in Gravelly Soils, Improvements to the Becker Penetration Test for Estimation of SPT Resistance*. Report to the National Research Council of Canada, Industrial Research Assistance Program, Report No. IRAP-M 40401W. Foundex, Vancouver, British Columbia, Canada, 1992.
11. Liao, S. C. and R. V. Whitman. Overburden Corrections for SPT in Sand. *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 3, March 1984.
12. Palmer, S. P. *Final Report SR 504 Blast Densification Project Surface-to-Downhole Shear Wave Velocity Surveying*. Report to the Washington State Department of Transportation, Seattle, July 6, 1993.