BLAST DENSIFICATION FOR MITIGATION OF DYNAMIC SETTLEMENT AND LIQUEFACTION

WA-RD 348.1

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16. ABSTRACT

A loose, debris avalanche deposit, resulting from the 1980 eruption of Mt. St. Helens, was encountered during the foundation geotechnical investigation for Bridge 12 on the Spirit Lake Memorial Highway (SR 504) in the Mt. St. Helens National Volcanic Monument, Washington. The deposit was determined to be at risk of experiencing liquefaction and/or dynamic settlement during the design seismic event for the structure (Richter magnitude 6.5 and peak ground acceleration 0.55g). The experimental use of deep blasting was selected by the Washington State Department of Transportation, and approved by the Federal Highway Administration, to densify the deposit full depth (up to 43 meters) for the purpose of mitigating the liquefaction and dynamic settlement risks. The contract for the work utilized a test section to evaluate the blast design. Revisions were made to the blast design based on the results of the test section blasting, and production blasting was completed in December of 1992. Numerous instruments were utilized to monitor and quantify the results of the blast densification.

Standard Penetration Testing (SPT) before blasting indicated average, corrected blowcounts of about eight full depth in the deposit. Post-blasting SPT blowcounts increased to above the goal of about 25 in the upper 15 meters of the deposit, and to above about 20 below 15 meters. Modified Becker Penetration Testing corroborated the SPT results. Total ground surface settlement of up to 1.5 meters was observed. Volumetric compressive strains on the order of about eight percent were determined on the basis of surface settlement and slope inclinometer measurements.

The total project costs were $599,000. On a unit cost basis, the treatment cost about $2.50 per cubic meter of densified ground. A cost savings of approximately $300,000 was realized over alternative ground improvement methods such as stone columns. Blast densification also allowed the use of cost-effective, shallow spread footings for the 60 meter, single-span bridge.

Even greater cost savings were realized when the blast densification method was used on the East Creek section of the final 6.2 km of SR 504, from Coldwater Lake Outlet to Johnston Ridge.

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BLAST DENSIFICATION FOR MITIGATION OF DYNAMIC SETTLEMENT AND LIQUEFACTION

EXPERIMENTAL FEATURES PROJECT WA92-08

performed as part of Construction Project
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Bridge 12, SR 504, Coldwater Lake to Johnston Ridge
Mt. St. Helens, Washington

FINAL REPORT

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DISCLAIMER

The contents of this report reflect the views of the author who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Washington State Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

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EXECUTIVE SUMMARY

The Washington State Department of Transportation used the ground modification technique of densification by blasting at the Bridge 12 site on the Spirit Lake Memorial Highway (SR 504). The technique successfully mitigated the potential for damage to a newly constructed bridge structure that could be caused by future earthquake induced liquefaction or dynamic settlement of the upper thirty-five to forty meters of loose, debris avalanche materials, deposited as a result of the May 1980 eruption of Mt. St. Helens.

The blasting produced an increase in relative density (D_R) of the foundation soils from a pre-blast range of twenty to forty percent to a post-blast range of fifty to seventy percent. Ground surface settlements were observed to range from about three-tenths to one and one-half meters. Total volumetric strains were indicated to be about eight percent within the deposit.

In addition to mitigating the seismic risks, the improved ground conditions made it possible to utilize more cost-effective, shallow, spread footings for support of the moderate size, single span structure. Further cost savings (estimated at $300,000) were realized since the unit cost of the blast densified soil was up to fifty percent lower than alternative methods of ground improvement.

A wide array of instrumentation was used during the project to evaluate and quantify the level of ground improvement achieved by blasting. Much of this data is summarized herein. Further research and evaluation of blast densification technology should include the data collected during this project.

Blast densification has had little previous usage. The project was therefore determined to be experimental. This report constitutes the deliverables of the Experimental Features Workplan prepared for the project.

INTRODUCTION

Densification by deep blasting was used by the Washington State Department of Transportation (WSDOT) to mitigate liquefaction and dynamic settlement caused by potential future strong earthquake ground motion at a new bridge site constructed as part of the Spirit Lake Memorial Highway into the Mt. St. Helens National Volcanic Monument. An experimental features workplan was prepared by WSDOT and approved by the Federal Highway Administration (FHWA) for evaluation and research to be conducted as a part of the project. This final report constitutes the deliverables of that workplan, and summarizes the results of the project and the associated ongoing research.

SITE LOCATION AND DESCRIPTION

The site of the new bridge construction is referenced as Bridge 12 on SR 504 (Spirit Lake Memorial Highway) in the Mount St. Helens National Volcanic Monument. The Bridge 12 structure is a portion of the final 11 km segment of the Highway to be constructed from Coldwater Lake (elevation 730 m) to Johnston Ridge (elevation 1400 m). The proposed bridge will cross South Coldwater Creek above its outflow into Coldwater Lake (Figure 1, Photos 1 and 2). The bridge will be a single-span, 60 m, steel plate girder with retaining walls at all four corners. The project area is within the avalanche debris flow deposits resulting from the May 18, 1980 eruption. The eruption triggered a rockslide/debris avalanche, and associated lateral blast, which devastated approximately 325 square kilometers of ground north of Mt. St. Helens. The debris avalanche deposits formed blockages at the outlets to Coldwater Creek and South Coldwater Creek creating avalanche debris dammed lakes.

PRELIMINARY GEOTECHNICAL INVESTIGATION

Golder Associates, Inc., Seattle, Washington, prepared the geotechnical report for the Bridge 12 site. Four borings drilled during the foundation investigation encountered loose avalanche debris materials associated with the 1980 eruption to depths of 37 to 43 meters below the existing ground surface. The avalanche debris materials consisted of a multi-colored, heterogeneous mixture of sands and gravels with varied amounts of silt, cobbles and boulders up to two meters in diameter. Below these materials, dense to very dense silty sands and gravels (tephra) were encountered, representing the pre-1980 ground surface. Average, corrected Standard Penetration blowcounts, $(N_1)_{60}$, of about 8 were observed in the post-1980 avalanche debris deposit. The relative density (D_R) of the deposit was determined to be approximately 20 to 40 percent.

Groundwater levels corresponded roughly with the level of water in South Coldwater Creek which varies from two to five meters below the existing ground surface at the bridge site.

Due to the loose, saturated and unconsolidated nature of the post-1980 deposit, it was determined that there was a high probability for dynamic settlement and liquefaction under the ground motions produced by the design maximum credible earthquake event. It was further determined that a low to medium risk of liquefaction, but a high risk of seismically-induced dynamic settlement, was present under a more likely, less severe event.

PLANNING AND DESIGN PHASE

SEISMIC CONSIDERATIONS

The Mt. St. Helens Seismic Zone (Weaver & Smith, 1983) is an interpreted 100 km long, near vertical, right lateral, strike slip, active fault zone. The maximum recorded earthquake was a 5.5 Richter magnitude event which occurred on February 14, 1981. The epicenter was near Elk Lake, approximately 5.2 km north of South Coldwater Creek. The occurrence of crustal earthquakes (3 to 17 km deep) larger than the measured 5.5 event are possible along the Zone (Meyers, et al., 1985).

The proximity to the Zone resulted in WSDOT adopting a maximum credible design seismic event (MCE) of Richter magnitude 6.5 with a peak ground acceleration of 0.55g.

LIQUEFACTION AND SETTLEMENT POTENTIAL

Liquefaction analyses were performed based on the standard penetration test (SPT) data observed during the investigation and procedures developed by Seed (1983). The analyses indicated that about two-thirds of the SPT results fall within the range where liquefaction is a moderate to high risk under the design 0.55g ground acceleration.

The effects of a major liquefaction failure of a large area in the vicinity of the bridge could include loss of vertical and/or lateral bridge foundation ground support, ground subsidence and lateral spreading. Lateral spreading could be particularly damaging since it would probably displace the bridge laterally, even if supported on deep piles or shafts. The effects of local liquefaction could induce differential settlement and possibly lateral movements which could damage the bridge.

Dynamic settlement on the order of a few tenths of a meter could be expected under seismic loading conditions for a modest seismic event. Under the loading conditions of a large earthquake, such as the MCE, dynamic settlements could exceed several meters.

In summary, the effects of seismically-induced liquefaction would be loss of foundation support, lateral spreading and ground subsidence. Under strong ground motion seismicity which failed to induce liquefaction, effects would still include large and differential settlements. All of these effects presented unacceptable risks for bridge design.

FOUNDATION OPTIONS

The foundation options for the Bridge 12 site were principally evaluated based on seismic risk, cost and constructability issues. Due to the extensive depth and loose nature of the site soils, several significant design issues had to be addressed. These included foundation support, liquefaction potential, static and/or dynamic settlement and the advantages of ground modification. Both shallow and deep foundation support systems were considered.

The existing condition of the natural foundation materials were generally not suitable for bridge support on shallow spread footings. Due to the very young geologic age of the debris avalanche deposit and its loose, saturated nature, coupled with the effects of buried organics, the deposit may still be experiencing natural settlement. This condition presented a potential for differential, and essentially un-quantifiable, static settlement under the loadings of approach fills and foundations. Seismic induced liquefaction and/or settlement would increase the risk for unacceptable movement of the bridge. Shallow spread footings, without some form of ground modification, were therefore not considered viable.

Shallow spread footings, founded on modified ground of limited depth (say, depths of 10 to 15 meters), were considered to mitigate the low bearing capacity of the near surface materials, as well as the potential for near surface liquefaction and static or dynamic settlement. However, the condition of the lower 25 to 30 meters of the 1980 debris avalanche deposit raised questions regarding not only static and dynamic settlement, but for the potential for deeper liquefaction. In general (as a rule of thumb), the potential for liquefaction at depths greater than 10 to 15 meters is not considered possible. This generalization is based on observed seismically-induced liquefaction in locations such as Anchorage, Alaska (1964) and Nigata, Japan (1964). However, the debris avalanche deposits at the Bridge 12 site are dissimilar to those observed to liquefy in Alaska and Japan, especially with regard to the depositional environment (debris flow/avalanche vs. fluvial/deltaic) and material constituents (avalanche scoured materials including boulders, cobbles, sands, ash, organics and ice vs. fluvially sorted sands and gravels). Based on these observations, it was determined that there was insufficient evidence to conclude that liquefaction could *not* occur at depths greater than 15 meters within a relatively young and loose deposit such as encountered at Bridge 12. Shallow foundation options, without deep ground improvement, were therefore rejected.

Deep foundation systems such as driven piles or drilled shafts would need to resist either, or a combination of, downdrag forces due to static or dynamic settlement, or lateral loading due to liquefaction induced lateral spreading of the deposit. Further, the presence of boulders in the deposit could result in costly constructability problems for any deep foundation system. High costs and questionable design and construction feasibility made these options undesirable.

Considering the unique nature of the deposit and the desire to keep costs in line for the moderate bridge size, ground modification techniques to improve the density and strength of the deposit became important considerations. The benefits would be to allow the use of cost-effective shallow footings founded on improved ground conditions, and to reduce seismic risks.

Several ground modification methods are available to increase the density and strength of a granular soil deposit. Examples include vibro-compaction, vibro-replacement (stone columns), deep soil mixing, jet or compaction grouting, deep dynamic compaction and blast densification. Any of these methods are viable alternatives for ground modifications, but the presence of boulders in the deposit posed a high risk for damage to the equipment used in vibro-replacement, jet grouting and deep soil mixing operations. These methods were also estimated to be at least twice as costly as compared to blast densification. Deep dynamic compaction is capable of ground improvement to a limited depth of about 10 to 12 meters which made this method incompatible with the requirements of this project. Thus, the need to modify the loose debris avalanche deposit full depth (40 meters), and the desire to keep costs down made blast densification the preferred ground improvement option for the Bridge 12 site.

PREFERRED FOUNDATION CONFIGURATION

Spread footings founded on material modified by deep blasting were selected as the preferable foundation system for Bridge 12. The goals of deep ground modification included mitigation of both static and dynamic settlement concerns. The modification also needed to be full depth within the post-1980 deposit and be of sufficient areal extent to robustly resist lateral spreading due to liquefaction under strong earthquake ground motions. The abutment fills would be retained at all four corners with mechanically stabilized earth structures to minimize impact to the surrounding terrain.

OVERVIEW OF BLAST DENSIFICATION

Densification of granular soils requires first that the initial soil structure be temporarily broken down so that the particles can be re-arranged into a more compact condition. In saturated, cohesionless soils this is most readily accomplished by applying dynamic and cyclic loadings thereby inducing liquefaction. In the case of blasting or dynamic compaction, the compression wave generated by the sudden large release of energy results in an immediate increase in pore water pressure. When the pore water pressures equal the maximum total overburden stress (i.e., $R_U = u_o/\sigma_{V_o} \geq 1$) liquefaction occurs, accompanied by a sudden drop in the shear strength of the soil. The shear wave, which travels slightly slower than the preceding compression wave, then fails the soil by exceeding the reduced soil shear strength. The soil particles are then free to re-arrange into a more compact condition of increased density.

Densification by blasting differs from ordinary construction practices in that it has had limited usage, particularly on U.S. highway projects, even though documented use of blast densification can be traced back as far as 50 years. The reluctance to employ blast densification is largely due to the absence of a theoretical design basis. Blast design is empirical, based on prior experience, and modified by site trials. The Jebba dam project in Nigeria (Solymar, 1984) was the only documented project where blast densification was utilized to a maximum depth similar to the Bridge 12 site. However, the Bridge 12 site required densification full-depth in the deposit, while the zone of treatment at the Jebba dam site was confined to a single, loose soil layer at a depth of approximately 35 m. Theoretically, there does not appear to be any restriction on the depth of densification achievable by blasting methods.

DENSIFICATION OBJECTIVES

Specific "densification criteria" or "acceptance criteria" were not written into the contract for this project. The reasons for this are discussed below under Contracting. However, it was acknowledged that certain "benchmarks" needed to be achieved in order to be assured that potential settlement and liquefaction were mitigated. The following criteria were established to evaluate the effectiveness of the blast densification.

- Standard Penetration Test - In order to mitigate the potential for earthquake induced liquefaction, it was determined, using Seed's criteria, that corrected Standard Penetration Blowcounts, $(N_1)_{60}$, would need to be above about 25 within the upper 15 meters of the deposit and above about 20 below 15 meters.

- Settlement - Estimates of settlement which would be induced by the blast densification predicted that as much as 4 meters of settlement was possible. It was generally felt that settlements on the order of 1.5 to 3.0 meters would be necessary to mitigate the dynamic and static settlement concerns for the bridge structure. These types of settlements were also felt necessary to represent the desired increase in relative density.

CONTRACTING

OBJECTIVES

The contract for the ground modification at the Bridge 12 site was let separately from the main contract for the major highway construction of the Coldwater Lake to Johnston Ridge extension. This was done to expedite the work and allow for evaluation of the success of the blast densification method. By contracting separately, the use of the experimental construction technique, with possible changes required to the final blasting plan, would not impact the major new alignment project.

The technical objectives were to densify the soil full depth and to sufficient areal extent to create a "stable island" to withstand strong ground shaking. Generally this was controlled by measuring the improvement in density by the methods described above. Use of an alternative, infrequently used, technology may create uncertainty in contracting. Consequently, it became an objective to share the risk of the project by not including an explicit performance specification based on SPT results in the contract.

A workable contracting method was developed to include:

- A pre-qualification requirement of contractors,
- The contract specified a base program in terms of number of holes and spacing, construction sequencing, energy and blast depth, etc. 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 which 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 an equitable early termination of the work.
- The contract specified the types of instrumentation required to control/monitor the densification effort and to evaluate the results.
- The contract did not specify drilling method, explosive type, etc., but left selected details up to the contractor. The actual production blasting program implemented was determined 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 implemented, including any changes.

The blast densification contract was divided into three phases:

- Phase 1 consisted of a test section which amounted to approximately one-fourth of the total proposed densification area.
- Phase 2 was a one-week evaluation period during which WSDOT studied and evaluated the results of the test section blasting. This time period also allowed WSDOT to exercise the

option of proceeding with the production blasting, including any modifications to the blast plan, or to cancel the contract.

- Phase 3 consisted of the production blasting to densify the remaining three-fourths of the proposed blast area, to proceed only upon a determination by WSDOT that the results of the test section were acceptable or could be made so by modifications to the blast plan.

An advisory specification was included in the contract which described the interpreted geologic and site subsurface conditions. The specification alerted bidders as to the potential for difficult drilling due to the presence of boulders in the post-1980 avalanche debris deposit.

ENVIRONMENTAL SENSITIVITY

The site is located within the Mt. St. Helens National Monument which is under the management of the United States Forest Service (USFS). Several environmentally sensitive issues impacted the blast densification project. The surface morphology is highly valued and includes topographical features, known as "the hummocks," which are peculiar to the ground surface of the avalanche debris deposits. Protection of these features to the maximum extent possible was mandated (i.e., settlement outside of the right-of-way limits was not acceptable). Site access was restricted to specific roads. Silt fences were required to preclude silt run-off.

INITIAL BLAST PLAN

The design charge spacing and size was empirical, based on data available from case histories. This design was significantly influenced by the blast densification program conducted at the Molikpaq caisson retained island in the Canadian Beaufort Sea (Rogers, et al, 1990; Stewart and Hodge, 1988). 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, it was decided to space charges at a nominal vertical spacing of about 6 m with the first charge located about 1.5 m below the water table. Consequently, the charges were placed at depths of 5, 11, 17, 23, 29 and 36 m below the ground surface. The spacing between the bottom two charges was increased to 7 m to allow densification to about 40 m.

The lateral spacing of charges was largely controlled by three factors:

- The need to minimize the total number of holes to be drilled,
- The decision to use a "two-pass" approach, which has been the common approach at most other blast densified sites, and
- To stay within the 5 to 15 m guideline for charge spacing presented by Mitchell (1981).

The "two-pass" approach lays out charges 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. The first grid is detonated in the first pass, and the second grid is detonated in the second pass. The grid layout for the blast holes, etc. is shown on Figure 2 and in the Contract Provisions and Plans, Appendix A.

The proposed area of densification consisted of two areas, each approximately 25 by 45 m, and roughly centered around each bridge abutment location. Using the two-pass approach design resulted in three rows in the first pass with an effective spacing between rows of 11.5 m.

The term "powder factor," as used below, is the mass of explosive utilized divided by the total volume of soil improved by blasting in one blast sequence ("pass"), in g/m^3. The total volume was calculated as the plan area plus one-half hole spacing outside the perimeter of the blast holes times the depth of treatment. Powder factors noted in the literature (La Fosse, et al, etc.) appear to be calculated similarly. Other methods of calculation could be used, for instance, dividing the total mass of explosive used in all passes divided by the total volume treated. However, it is felt that the "powder factor per pass" calculation is more pertinent to the blast design.

The charge sizes were designed largely on past experience where the powder factor was between 5 and 30 g/m^3 of treated soil. There was also concern about the potential for "cratering" of the ground surface, and concern about the potential for triggering slope failures in the adjacent slopes if high charge weights were placed too near the ground surface. Beginning from the top deck of charges down, the contract blast plan called for six decks with 2.3 kg at 5 m, 4.5 kg at 11 m, 6.8 kg at 17 m, 9.1 kg at 23 m, 10.9 kg at 29 m, and 13.6 kg at 36 m. This resulted in a powder factor of approximately 15 g/m^3. Blast drillhole diameters were 170 mm O.D. (see Drilling, below) to allow for the specified 76 mm O.D. diameter PVC explosive casing.

The intent of blast densification is to produce settlement by temporarily inducing liquefaction. During earthquakes, liquefaction results from cyclic loading of the soil. 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 accomplished 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 the charges at any one time and induce a larger 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. There were no case histories found in the literature where the primary focus was to evaluate the effects of blast densification by varying the delays between the charges. For this project it was decided to use delays both between decks, fired from the bottom up, and between rows. The charges were fired one row at a time, with a 75 ms delay between rows, and a 400 ms delay between the vertically spaced decks.

Soil densification by inducing liquefaction requires the concurrent removal of water from the decreasing pore space. Vertical drains were installed equi-distant between the blast holes. The drains consisted of 7.6 cm diameter, Schedule 40 PVC, with 2.5 mm slot size to aid in the removal of the water.

Evaluation periods were detailed in the contract to allow WSDOT to review the results of the blast densification. One week was allotted for review of the test section results. Following the production blasting, a four month period was allotted to evaluate "aging" effects, if any. "Aging" refers to an observed phenomenon where soil deposits densified by blasting show increases in relative density over time following the blasting. The phenomenon is poorly understood, but has been observationally verified on previous blast densification projects.

BID ITEMS

The bid items were structured to allow flexibility should changes in the blast patterns or design be deemed necessary. Boreholes for installation of explosives, vertical drains, instrumentation and BPT/SPT testing were bid per foot of hole drilled. Explosives were bid by unit weight. Instrumentation was bid per each unit, and other ancillary items were either per unit or lump sum. Contract items and estimated quantities are noted in the excerpted Contract Plans and Provisions, Appendix A.

INSTRUMENTATION

The opportunity to collect valuable information regarding this technology prompted both WSDOT and FHWA to allocate additional funding for project instrumentation. Due to the uncertainties of the actual displacements, accelerations and pressures which may be induced by the blasting, a wide range of instruments were used for monitoring. The instrumentation included devices to quantify surface and subsurface displacements and distortions, pore water pressure changes and ground accelerations due to blasting.

The entire suite of instrumentation was not installed prior to the test section detonation. In the event that the contract was cancelled following the test section, only the minimum instrumentation deemed necessary to evaluate the test section results was installed. The locations of the instrumentation installations are shown on Figure 3, and in the Contract Plans and Provisions, Appendix A.

The locations of instruments are referenced to the blast hole grid notation. For example, piezometer KL-3.5 is located between lines K and L and between lines 3 and 4.

TEST SECTION

SITE GRADING

The areas for the blast densification grids (east and west abutments) were graded roughly level to allow for drill rig access. The work included excavation from adjacent hummocks to build up to 3 m high fills at the proposed bridge ends. This work was done under force account by the general contractor who was then completing the previous section of SR 504.

DRILLING

The contract was let to Foundex Inc. of Bellingham, Washington, and drilling for the test section commenced on October 30, 1992. The contractor mobilized a Becker Hammer drilling rig to advance the holes for the explosives, vertical drains and instrumentation. The rig was capable of advancing holes to the required depth of about 40 m in approximately 1½-hours. Depending on the down-hole installation (e.g. PVC pipe for explosive placement, instrumentation, etc.), start-to-finish time per hole averaged about 2½ to 3 hours. All drillholes advanced with the Becker rig for installation of explosives, vertical drains, etc., were 170 mm, outside diameter.

One advantage of blast densification was that the problem of the bouldery soil at the site was handled easily with the construction installation methods used. The Becker Hammer experienced little difficulty in penetrating this deposit. The truck mounted HAV-180 Becker Hammer Drill consists of a double-acting diesel hammer (rated energy: 10.9 kJ) driving a double-walled casing into the ground. There is no rotation of the drill string. Drill cuttings are exhausted to the surface through the inner annulus by compressed air charged to the outer annulus.

Under separate contract, and at about the same time as the initiation of the Becker drilling, a rotary drill was mobilized to advance the holes necessary for the down-hole seismic survey work to be performed by the Washington State Department of Natural Resources (DNR).

VERTICAL DRAINS

An initial modification to the blast plan was deletion of the vertical drains along two sides of the test section in order to evaluate whether the drains were necessary for the densification method to be effective. Vertical drains were installed in the remainder of the test section. The drain pipe was installed to depths equal to the drill holes (40 m) with no filter pack or other backfill (i.e. the holes were allowed to collapse around the drain pipes).

During installation of the drains, it was observed that significant siltation was occurring within the pipes within several days following installation. As much as 10 to 25 meters of silt and fine sand was discovered in each of the drain holes. There was some discussion as to whether the slot size (about 2.5 mm) of the drains should be reduced to decrease siltation. It was felt that if the slot size was reduced sufficiently to inhibit siltation, the slots would then be too small to remove water effectively. It was also felt that the silt in the drains was probably loose enough to be dislodged by the fluid pressure generated by the blasting.

LOADING, BLASTING AND RESULTS

On Monday, November 9, 1992, the first pass of the test section was detonated. The pattern consisted of nine blast holes on a three-by-three grid loaded and sequenced as described above and specified in the contract. See photos 3 through 6. Note that the grey/white spouts are explosive exhaust, while the brown geysers are water and silt from the vertical drains. The geysers appeared almost immediately following detonation, and continued to exhaust for about 5 seconds. After the site had been cleared for entry, a surficial reconnaissance was made. Approximately 10 to 15 minutes after detonation, several hydrofractures opened in the ground surface and fairly large volumes of water began to exit the subsurface. The locations of the hydrofractures seemed independent of the locations of the vertical drains.

Surface settlement from the test section first pass blasting ranged from 0.17 to 0.38 m and averaged about 0.28 m within the blast zone. Between 1 and 1.5 m of settlement had been expected. Subsurface settlements, as measured by two Borros anchors installed at depths of 19 and 27 meters, were 0.14 and 0.09 meters, respectively. These data indicated that densification was occurring full depth within the deposit, and that vertical strain due to settlement was fairly constant with depth. A Sondex tube showed a similar linear increase in vertical settlement from the bottom to the top of the deposit. There did not appear to be any signs of cratering from the blast, nor were there any signs of large movements in the adjacent slopes of the hummocks. Minor slope movements were evidenced by several tension cracks associated with the settlement. Based on these results, it was decided that larger charges were warranted, but that the top charge would remain at 2.3 kg. The new charge profile consisted of 2.3 kg at 5 m, 9.1 kg at 11 m, 11.4 kg at 17 m, 15.9 kg at 23 m, 15.9 kg at 29 m, and 27.3 kg at 36 m. This resulted in a powder factor of approximately 25 g/m^3 (versus the initial 15 g/m^3). The initial and revised explosive charge profiles are shown on Figure 4.

The second pass consisted of four blast holes on a two-by-two grid centered on the first pass grid, and was detonated on the following day, November 10, using the revised charge profile. Surface settlements from the second pass averaged about 0.21 m for a total settlement of about 0.49 m within the test section blast zone. 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, even though fewer blast holes were used and the subsurface was already somewhat denser due to the first pass blast, strongly indicated that the larger charge sizes were effective and warranted.

EVALUATION OF TEST SECTION RESULTS AND MODIFICATIONS TO INITIAL BLAST PLAN

Based on the results of the test section, the following modifications were made to the blast plan:

- The increased charge profile (powder factor) used in the second pass of the test section would be used for the production blasting.

- The vertical drains were deleted. Visual observations indicated that the blast holes drained more water than the drains, and sand boils developed due to hydrofracturing of the ground irrespective of the locations of blast holes or drains.

- The 75 ms delay between rows was deleted, and the 0.4 second delay between decks was reduced to 0.3 seconds. It was postulated that damping at the site could have reduced vibration levels more than anticipated. This may have resulted in reduced settlement.

- A third round of blasting was added for the test section area to provide the same minimum amount of energy to be applied to the remainder of the production blast areas.

PRODUCTION BLASTING

Drilling at the site resumed on November 23, 1992. The remainder of the blast schedule was as follows:

- Third pass, test section, detonated at 4:59pm on December 11, 1992
- First pass, remainder east abutment, detonated at 1:24pm on December 12, 1992
- First pass, west abutment, detonated at 4:27pm on December 13, 1992
- Second pass, remainder east abutment, detonated at 12:56pm on December 14, 1992
- Second pass, west abutment, detonated at 10:03pm on December 15, 1992

The contractor's crews were re-mobilized in early January, 1993, and again in April, 1993, to perform the contract specified post-blast testing.

Visual reconnaissance around the vicinity of the abutments following blasting revealed that some ground cracking, with vertical differential movement, occurred in the ground surface surrounding the blast area. These effects were limited to a distance of about 30 or 40 meters from the densification areas. The cracks were most notable atop the hummock on the west side of the west abutment, where the job site trailer and staging area was located (Photo 7). It was noted that this hummock seemed to apply a surcharge load to the west side of the densification area which increased the magnitude of settlement in that area. This ground cracking included some vertical displacements of up to 0.3 meters and caused tilting of the contractor's trailer.

Much greater volumes of water came to the surface following the primary blast of the west abutment than observed following previous blasts. A pool of water about 1 meter deep collected in the depression created by the settlement (Photo 8). This made reading some of the instrumentation difficult.

Following some of the blasts, insufficient wind was blowing to clear the gases ejected from the blast holes. These gases were noxious and necessitated evacuation of the blast area until the gases had cleared.

Some surface cratering was noted following the second round of blasting on the remainder of the east abutment. These craters formed around the blast holes 5 to 15 minutes following detonation and were collapse features up to about 5 meters in depth (Photo 9). The ground would give way rather suddenly, and care was therefore needed when walking around the area. Due to the cratering it was determined that the spread footings should bear at a depth below the observed cratering.

SITE SAFETY

Site safety was the responsibility of the contractor. Procedures for safe blasting were observed, including blast area clearance and warning signals.

TESTING AND INSTRUMENTATION RESULTS

INCREASE IN RELATIVE DENSITY

Two types of penetration tests were performed to evaluate the increase in relative density (D_R) produced by the blasting: Standard Penetration Tests (SPT) and Foundex mudded Becker Penetration Tests (FBPT).

Standard Penetration Testing

Standard Penetration testing was conducted in accordance with ASTM D-1586, "Penetration Test and Split-Barrel Sampling of Soils". Energy transfer was evaluated by PDA (see Pile Driving Analysis, below) and was found to average about 43 percent of theoretical during testing, resulting in a 28 percent reduction of SPT 'N' values during normalization.

The pre-blast SPT data from the four boreholes drilled in 1991 during the foundation investigation for Bridge 12 were used to compare with the post-blasting SPT testing conducted in January 1993. Post-blast data was obtained in January 1993 following completion of the production blasting. Before and after blowcount values, $(N_1)_{60ecs}$, are plotted on Figures 5 through 7 as a function of depth. These values have been normalized to an effective overburden stress of 100 kPa (N_1), and corrected for hammer efficiency (N_{60}) and an estimated average silt content of 15 percent (N_{ecs}). A clear increase in relative density is indicated by these plots. In general, it appears that the blowcount "benchmarks" of at least 25 in the upper 15 meters and 20 below 15 meters was achieved. There is significant scatter in the SPT results, however, and many of the higher blowcounts may have been affected by gravel or larger particles. The presence of gravel reduces confidence in the SPT tests, however the difference between the 1991 and 1993 results clearly indicate an increase in density and a general attainment of the densification goals.

Becker Penetration Testing

The Becker Penetration Test (BPT) was identified as a more reliable means of measuring the increase in density produced by the blasting. The BPT is similar in concept to the SPT, and correlations between the tests have been published by various authors (e.g. Harder and Seed, 1986). The BPT consists of plugging the Becker Hammer drill string tip and performing what is essentially a small, closed-end, pile driving test. The major differences between the BPT and the SPT 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 these tests is 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 gravels. However, the increase in skin friction with depth along the side of the drill string makes correlation of BPT results with SPT results less meaningful below about 13 meters.

The BPT has the following advantages over the SPT:

- It provides continuous blowcount data;
- It minimizes the effects on blowcounts of gravels and cobbles due to the larger annulus at the tip;
- It is much more rapid than SPT's advanced by rotary drilling methods, and
- It is more easily monitored by pile driving analyzers for evaluation of driving efficiency and the effects of skin friction.

The Foundex mudded Becker Penetration Test (FBPT) was developed by Foundex Inc. to reduce the problems associated with skin friction in developing SPT/BPT correlations. A study by Foundex (1992) for the National Research Council of Canada showed that the FBPT showed better correlations to the SPT than the standard BPT. Further, the correlation was not significantly affected by depth below the ground surface.

The FBPT is a modified version of the BPT which uses an injected mud to reduce the side shaft friction of the drill string to essentially zero. Improved correlations to the SPT (N_{60}) values were found when the drill string plug was modified to a larger, oversized shoe (220 mm diameter), and the blowcounts were corrected for hammer efficiency based on observed bounce chamber pressures. For further details on the Becker Hammer drilling rig, the BPT and the FBPT, please refer to the excerpted portion of the National Research Council of Canada report: "A Testing Technique for Earthquake Liquefaction Prediction in Gravelly Soils" included as Appendix B. This report provides a statistical basis for correlation of SPT values with BPT and FBPT values.

The contract required three series of tests comprised of four FBPT tests in each series. The first series of FBPT testing was performed from November 27 to 30, 1992. Three tests were located within the production blast area prior to blasting. One test was performed in the area of the test section following the second pass blast. These tests developed a pre-blast base-line database for comparison. The second series of tests was performed approximately three weeks after production blasting was complete (January 8 to 12, 1993). The last series of tests was performed about four months after completion of blasting to study the effects of blast aging (April 5 to 8, 1993).

The pre-blast FBPT tests at the western abutment (locations D2 and I3) indicated average penetration resistances of about 4 and 8, respectively. The pre-blast FBPT tests at the eastern abutment (locations L4 and P2) indicated average penetration resistances of about 9 and 22, respectively. Note the higher average blowcount at the location P2 is due to the test being performed after the second pass of blasting in the test section.

For comparison, the pre-blast, post-blast and four month "aging" data are summarized on Figures 8 through 11, for each of the four test locations. These blowcounts have been corrected for bounce chamber pressure. A correction for the barometric pressure measured at the time of the test is also included. This is because the original FBPT/SPT statistical correlation was developed at sea level, and the measured bounce chamber pressure is dependant on atmospheric conditions.

The correlation of FBPT blowcounts is to SPT blowcounts, corrected for hammer efficiency only (i.e. N_{60}). However, evaluation of liquefaction potential (based on Seed's criteria) is established using SPT values which have been normalized and corrected for hammer efficiency and silt content (i.e. $(N_1)_{60cs}$). Applying analogous thinking regarding the two tests, it seems logical to apply similar adjustments to the FBPT data in order to evaluate liquefaction potential. It is acknowledged that no statistical basis for these adjustments exists, but an alternative method of liquefaction evaluation based on the collected data does not exist. The normalized, corrected FBPT data are shown on Figures 12 through 15.

Note that the FBPT data indicate that densification has occurred, although the magnitude of the increase is less than that indicated by the SPT data. In general, the densification goals appear to have been achieved, or nearly so. The effects of aging are noticeably indicated by the tests performed four months after blasting. The exception is the four month data from location L-4. Note that there is a sharp drop in FBPT blowcounts during this test. The blowcounts in fact drop below the *pre*-blast values, even though the test preformed soon after blasting indicated that densification objectives had been achieved. The cause for this anomalous data is unknown.

PILE DRIVING ANALYSIS

The FBPT test performed during the first and second series (November and January) were monitored using a pile driving analyzer (PDA) provided by Goble Rausche Likins and Associates, Inc. (GRL). A test was also performed on a regular BPT test (not mudded, and without the oversize shoe).

The test on the regular BPT indicated that side friction contributed significantly to the driving resistance. The PDA measurements of the FBPT tests confirmed that skin friction was reduced during driving so as to be negligible at all depths. Key elements of the report by GRL are included in Appendix C.

GRL also made dynamic measurements on the SPT test performed at location P2 in the second series of tests (November) to evaluate the SPT sampler efficiency. This data was used to correct the blowcounts to a reference efficiency of 60 percent which was assumed for the SPT blowcount data obtained during the 1991 bridge foundation investigation. The results of this test are included in Appendix C.

SETTLEMENT AND LATERAL DEFORMATIONS

Settlement was monitored both on the ground surface and subsurface utilizing:

- Survey hubs and settlement plates - Forty settlement plates, along with numerous other survey hubs were utilized to develop the surface settlement pattern induced by the blasting. Total settlements of up to 1.5 meters were observed as shown on cross sections A-A′ and B-B′, Figures 16 and 17. The locations of the section lines are shown on Figures 3 and 18.

Since the amount of settlement produced by the first pass was substantially greater than the second pass, the increased charge weights appeared to increase densification efficiency at the Bridge 12 site. Note the larger amount of settlement on the left (west) side of section B-B′. This is interpreted to be a result of the surcharge load imposed by an 8 to 10 meter high "hummock" present on this side of the densification area. A settlement contour map of the west abutment area was developed from the settlement plate data and is included as Figure 18. This contour plot also shows the greater amount of settlement along the west side of the blast area which was attributed to the "hummock" on that side of the abutment. The settlement data indicate vertical strains of about 4 percent within the deposit.

- Sondex tubes - Sondex tubes are flexible and compressible, corrugated pipe which is fitted every vertical meter with a steel ring. In order to prevent collapse under confining pressures the flexible pipe is installed with a smaller diameter PVC pipe down the annulus. The steel rings are sensed with a down-hole magnetic probe, thus providing data on subsurface settlements approximately every meter to the depth of the installation. Two Sondex tubes were installed in the test section, with an additional eight tubes installed for the production phase of work. Five of the Sondex installations included slope inclinometer tubes placed in the annulus for support and to provide lateral deformation data. The Sondex tubes were installed to the maximum depth of the post-1980 deposit. Results of the Sondex monitoring are shown on Figures 19 to 28. The data indicates that vertical strains were fairly uniform and consistent over the full depth of the deposit. Settlements in the range of 0.3 to 1.5 meters are indicated which agree with the surface settlement measurements. Note that Sondex S-3 (Figure 21) was located approximately 15 meters away from the west abutment blast area and indicates that settlement was negligible at this distance from the blasting.

- Borros anchors - Two Borros anchors were installed within the test section at depths of 19 and 27.5 meters. These devices consist of a trio of steel prongs which are expanded into the ground at the depth where settlement data is desired. Prong expansion is accomplished by driving the retracted prongs over a conical tip. These devices are relatively robust and were selected to provide a minimum amount of data upon which the blast densification results could be evaluated should other subsurface instrumentation be damaged or unreadable following detonation. The Borros anchors are limited by the fact that deformation data is only provided for a single subsurface point. The results corroborate the Sondex readings.

- Slope inclinometers - As mentioned above, five slope inclinometers were installed in the same holes drilled for five of the Sondex tubes. These instruments provided information on lateral deformations by measuring angular deflections from the vertical using a down-hole accelerometer probe. Depths of the installations were the same as the Sondex tubes. Slope Inclinometer SI-1 was damaged during blasting, and therefore provided no post-blast data. Plots of lateral deformations are shown on Figures 29 through 32. The most interesting data from the slope inclinometers comes from the west abutment. Slope Inclinometer 4 (SI-4, in the vicinity of D5) showed up to 1 meter of lateral movement toward the center of the blast zone. Slope Inclinometer 5 (SI-5, in the vicinity of E/F1) also showed movement toward the center of the blast zone (direction of movement about 180 degrees opposite of SI-4), with maximum movements of about 0.3 meters. These inclinometers are about 15 meters apart. Thus the average lateral compressive strain in the deposit is about 4 percent. When added to the vertical strain, this results in a total volumetric compressive strain of about 8 percent. The larger lateral movement in SI-4 is also attributed to the nearby "hummock" surcharge on this side of the west side of the west abutment densification area.

PORE WATER PRESSURES

Piezometric transducers were installed at depths of 7.5, 14, 26 and 35 meters at two locations within the west abutment area and one location in the production phase area of the east abutment to monitor pore water pressures during blasting. The pore pressures were monitored by a data logger which yielded the results shown on Figures 33a, 34a and 35a. Complete liquefaction appears to have occurred at nearly all depths, as indicated by normalized pore pressure parameter values, (R_U), of unity and above as shown on Figures 33b, 34b and 35b. The attenuation with distance of pore pressures induced by blasting may be noted by the readings in the west abutment piezometers when primary charges in the east abutment production blast area (about 60 meters away) were detonated. There is a barely discernable "blip" at about 20 hours in the readings (Figures 33a and 34a) indicating when this blast occurred. The logarithmic dissipation of pore pressures with time can also be seen as values approach hydrostatic levels within a day or two following blasting. Note that the R_U values show less scatter with depth in the deposit as blasting proceeds which may be indicative of an increased uniformity of relative density throughout the depth of the deposit.

Physical manifestation of the pore pressures induced by the blasting were sand boils and high volumes of water migrating to the ground surface within approximately 30 minutes following detonation. Water continued to flow to the surface for several hours after blasting.

Overall, subsurface instrumentation survival was good, with one piezometric transducer failing during primary blasting, one piezometric transducer failing during secondary blasting, one slope inclinometer being damaged and one Sondex tube partially silting up. Otherwise the instrumentation was readable and provided data throughout the project.

GEOPHYSICAL MEASUREMENTS

During blasting, ground accelerations were monitored by Geo Recon International with geophones located on the ground surface. Excerpted portions of Geo Recon's report are included as Appendix D. Peak particle velocities of 233 mm/s were recorded within the middle of the west abutment blast area during the 15 hole (primary) blast sequence. The peak velocity attenuated to about 77.5 mm/s at a distance of 53 meters from the center of the blast hole grid. However, the frequency of the blast vibrations substantially fall below about 40 Hz. "Rock Blasting and Overbreak Control," lists a variety of criteria for evaluation of the likelihood of plaster or drywall damage. In general, for vibrations lower than about 40 Hz, peak particle velocities should be below about 1.25 to 2.0 mm/s. Clearly, the use of blast densification would not be appropriate very close to a facility or structure sensitive to ground motions. Further, careful blast monitoring would be imperative if the technique was employed within a populated area.

The Washington State Department of Natural Resources provided the equipment and personnel to measure sonic velocities of shear waves induced at the ground surface and propagating downward into the deposit. Surface-to-downhole shear wave velocity surveying was performed both pre- and post-blasting in the west abutment area. Post-blasting surveying only was performed in the east abutment area. The survey results indicated that above 6 m in the deposit, the shear wave velocity (V_S) did not change appreciably due to the blasting. This leads to the conclusion that the blasting did not densify the deposit above the topmost deck of explosives. V_S increased from 20 to 50 percent in the depth interval from 6 to 24 meters. The surveys did not indicate an appreciable increase in V_S below a depth of 24 meters. Post-blasting velocities in both the east and west abutments were comparable. Excerpted portions of the report prepared by DNR are included as Appendix E.

Washington State University provided the personnel and equipment for downhole measurements of natural gamma radiation and neutron density. Measurements were collected prior to blasting by lowering the instrument down a vertical casing installed in a drill hole. The casing, unfortunately, was destroyed by the blasts, and no post-blast data was collected.

COSTS

The construction contract award was for a total cost of $460,040, which included $101,150 for instrumentation. Due to the deletion of the vertical drains, the final contract payment was $392,390 (the instrumentation costs did not change). The total project cost, including the geotechnical consultant and other costs was $599,000. The total volume of soil treated was approximately 118,000 m^3, calculated based on the estimated radius of improvement around a single blast hole. This translates to a ground improvement unit cost of about $2.50 per cubic meter, not including instrumentation or consultant costs.

For comparison, the estimated cost of vibro-replacement (stone columns) was about $5.00 per cubic meter for the same volume of treated soil. Thus, a cost savings of approximately $300,000, or 50 percent, was realized for this project.

CONCLUSIONS AND RECOMMENDATIONS

The loose, avalanche debris deposit beneath the Bridge 12 site was sufficiently densified by the use of blasting to mitigate the high potential for liquefaction and the probability of extreme ground settlement due to a seismic event.

For the soils at the Bridge 12 site, the use of vertical drains for subsurface drainage was determined to be ineffective. The silt content of the soils was such that the drain pipes silted up too rapidly to provide significant drainage. It was realized that in order to prevent siltation, either a graded filter would need to be placed around the drain pipe, or the slot opening size would have to be reduced by an order of magnitude or more. The former would increase the unit cost of the drains so as to be prohibitive, while the latter would reduce permittivity to levels so low as to negate the ability of the drains to remove the necessary quantities of water. The silt content of a soil deposit should therefore be considered when developing a blast densification plan.

Due to the ground vibrations produced during blast densification, the use of the technique would not be appropriate very close to a facility or structure sensitive to ground motions. Blast monitoring should be considered essential for any similar project proximal to structures or populations.

Site safety considerations should include provision for limiting personal exposure to concentrations of noxious explosive gasses expelled from blast holes.

The use of blast densification reduced overall bridge construction costs by allowing the use of spread footings for Bridge 12, and by providing densification at substantially lower cost (50% savings) relative to alternative ground improvement techniques. Blast densification was proved to be both cost effective and expedient on this project.

The WSDOT has determined, based on the success of this experimental features project, that it will not hesitate to use blast densification again in the future on comparable ground improvement projects.

The database of instrumentation results obtained during this project should enhance the state of knowledge regarding blast densification. Future research on this subject should make use of the data collected at Bridge 12.

ACKNOWLEDGEMENTS

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Particular gratitude and congratulations are granted to Mr. Dave Jenkins of the Washington State Department of Transportation and Mr. Al Kilian, formerly of the Washington State Department of Transportation, now with the Federal Highway Administration, for their willingness to experiment, their diligence in application of the observational method and commitment to the finished product.

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Lastly, thanks to Mr. Ron Chassie, Federal Highway Administration Regional Geotechnical Engineer, for his reviews and support of the significant time and effort required for preparation of this report.

EPILOGUE

EAST CREEK BLAST DENSIFICATION SUMMARY

During the early construction stage of the Coldwater Outlet to Johnston Ridge project, partially loaded scrapers hauling over an area just below East Creek induced am unusual ground failure. The failure occurred on gently sloping ground (5 to 10 percent grade), and involved an area measuring nearly 120 meters wide and 60 meters in length. Based upon observations and subsurface investigation, the foundation soils liquefied due to the hauling operation. Embankments to 12 meters in height were planned to be constructed over the area of failure and continuing for an additional several hundred meters further west.

The typical subsurface section within the area consisted of an approximate five foot cap of medium dense to dense, well graded soil with cobbles and boulders (debris flows, colluvium); underlain by 3 to 10 meters of loose to mostly medium dense, saturated silty (20 to 30 percent silt) fine to coarse sand (fluvially deposited ash); underlain by 3 to 6 meters of dense to very dense, well graded soil with cobbles and boulders (glacial till, colluvium); then bedrock. Subsurface investigation within the failed section and for an additional area 120 meters east and 300 meters west indicated that in situ densities to a depth of 12 to 15 meters had decreased substantially from the initial 1991 geotechnical investigation. In two areas average SPT "N_{60}" values dropped from 12 to 2, and from 4 to 1, respectively.

A variety of options were considered to remediate these, now, very loose, foundation soils. These included sub excavation, dewatering, dynamic compaction, compaction grouting, stone columns, and blast densification. The only methods for remediating the poor foundation conditions given the site conditions, rapid delivery required and reasonable costs were judged to be stone columns and blast densification. The costs for the two operations for remediating a section 550 meters long by 20 meters wide and 12 meters deep (typical depth) are provided below. Blast densification was selected based on both the cost comparison and the dramatic reduction in time required for ground modification.

- Stone Columns (estimated time required: 5 to 6 months)
  Assuming: 11,000 m^2 treated area, with columns on 2.5 m spacing, 12 m deep, with an estimated 18,000 kg of rock per stone column
  Total Estimate $1,340,000
  Cost per m^3 of treated soil $10.15/$m^3$

- Blast Densification (time required: 3 weeks)
  132,000 m^3 of soil treated
  As Constructed Cost $459,000
  Cost per m^3 of treated soil $3.48/$m^3$

Average "N_{60}" values were improved to 15 to 20 within approximately 10 days after the secondary blast. The cost savings reflected above, although significant, is small as compared to the savings realized by the relatively short delay to the prime contractor (estimated on the order of millions of dollars).

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FIGURES

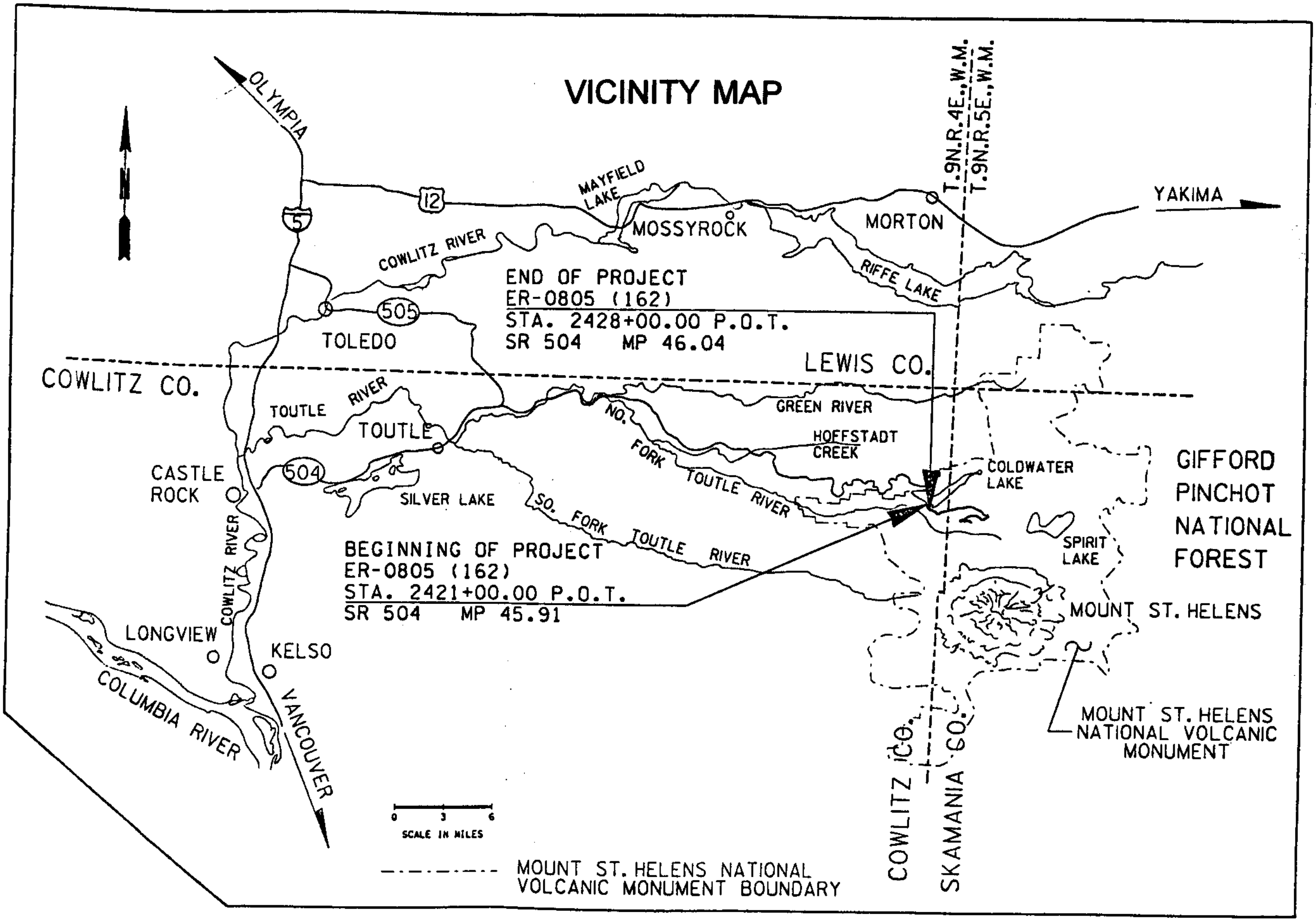

VICINITY MAP
OLYMPIA
5
12
MAYFIELD LAKE
MOSSYROCK
MORTON
YAKIMA
T.9N.R.4E.,W.M.
T.9N.R.5E.,W.M.
COWLITZ RIVER
RIFFE LAKE
END OF PROJECT
ER-0805 (162)
STA. 2428+00.00 P.O.T.
SR 504 MP 46.04
505
TOLEDO
LEWIS CO.
COWLITZ CO.
TOUTLE RIVER
TOUTLE
GREEN RIVER
NO. FORK TOUTLE RIVER
HOFFSTADT CREEK
COLDWATER LAKE
GIFFORD PINCHOT NATIONAL FOREST
CASTLE ROCK
504
SILVER LAKE
SO. FORK TOUTLE RIVER
SPIRIT LAKE
BEGINNING OF PROJECT
ER-0805 (162)
STA. 2421+00.00 P.O.T.
SR 504 MP 45.91
MOUNT ST. HELENS
COWLITZ RIVER
LONGVIEW
KELSO
COLUMBIA RIVER
VANCOUVER
MOUNT ST. HELENS NATIONAL VOLCANIC MONUMENT
COWLITZ CO.
SKAMANIA CO.
0 3 6
SCALE IN MILES
MOUNT ST. HELENS NATIONAL VOLCANIC MONUMENT BOUNDARY

FIGURE 1

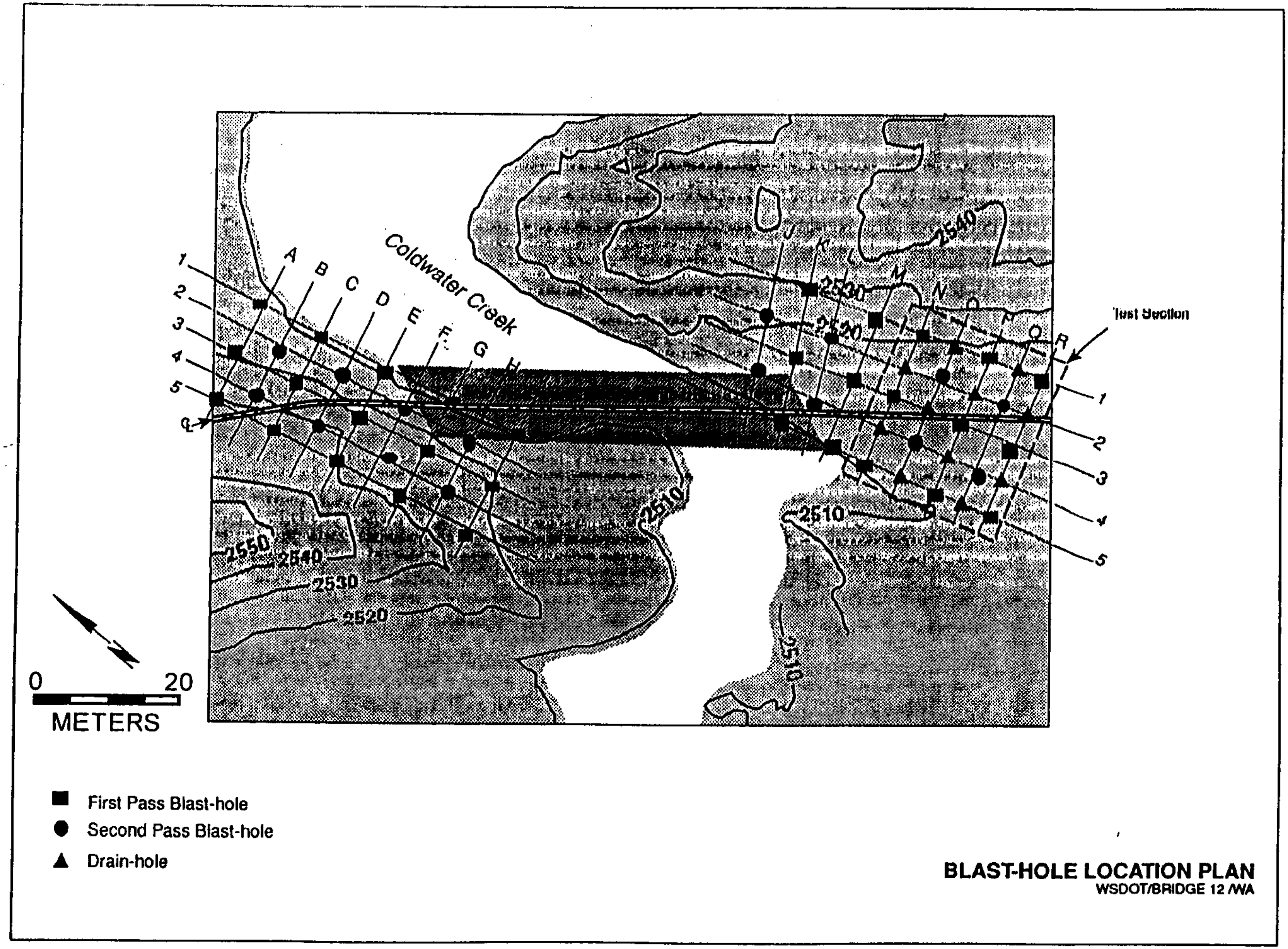

Coldwater Creek
Test Section
A
B
C
D
E
F
G
H
J
K
M
N
O
Q
R
1
2
3
4
5
2550
2540
2530
2520
2510
0
20
METERS
First Pass Blast-hole
Second Pass Blast-hole
Drain-hole
BLAST-HOLE LOCATION PLAN
WSDOT/BRIDGE 12 /WA

FIGURE 2

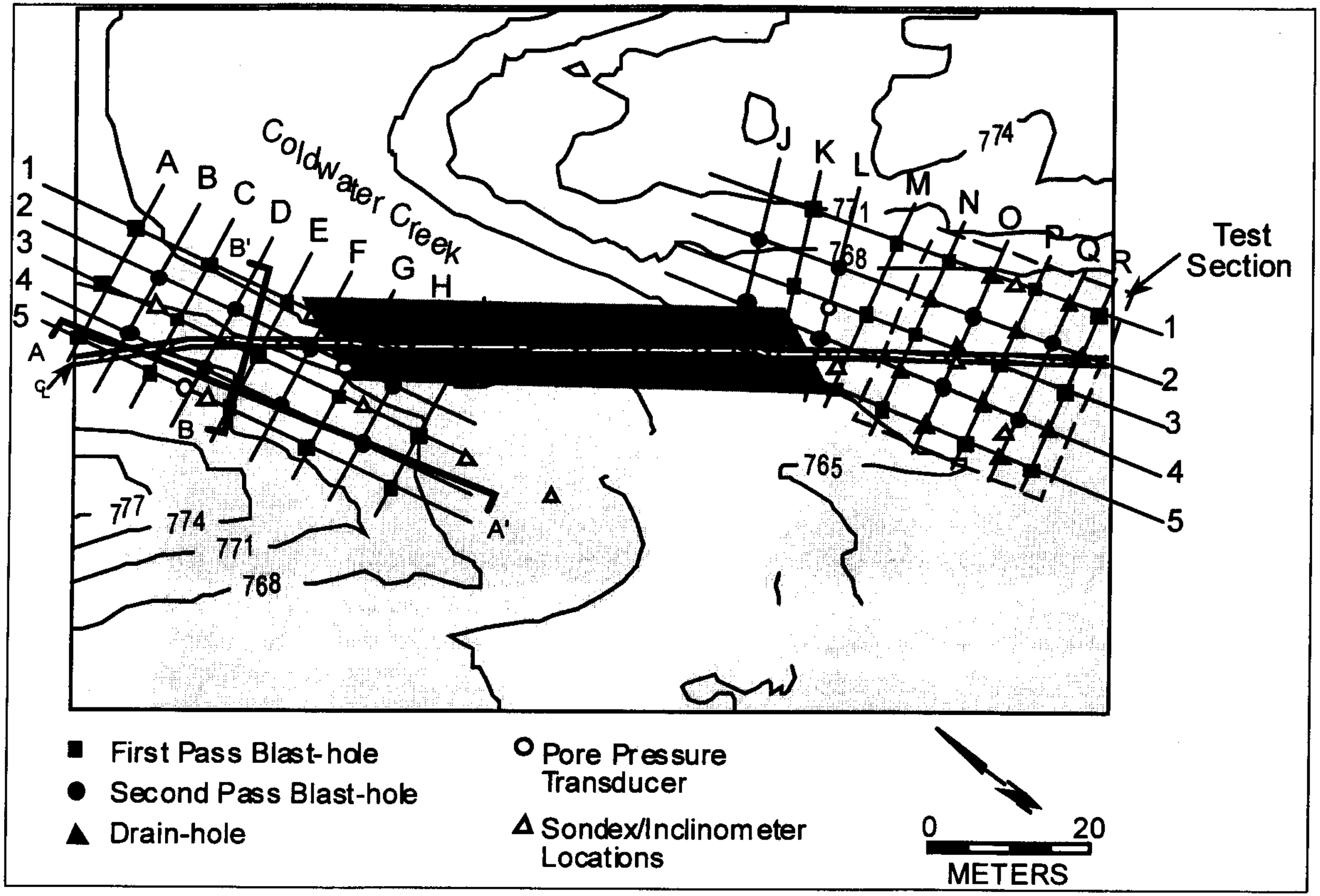


Figure 3 - Instrumentation Locations

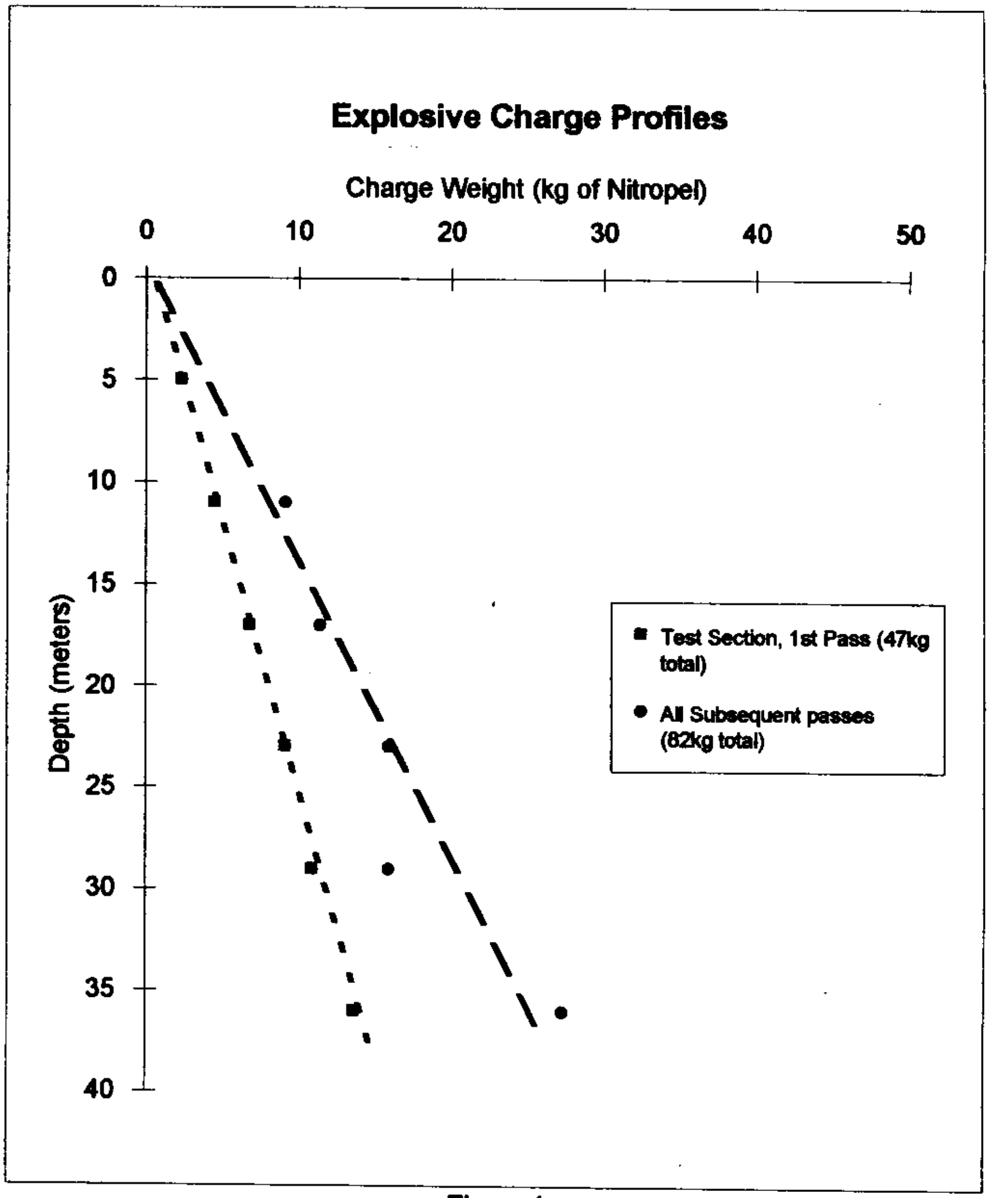

Explosive Charge Profiles
Charge Weight (kg of Nitropel)
0
10
20
30
40
50
Depth (meters)
0
5
10
15
20
25
30
35
40
Test Section, 1st Pass (47kg total)
All Subsequent passes (82kg total)

Figure 4

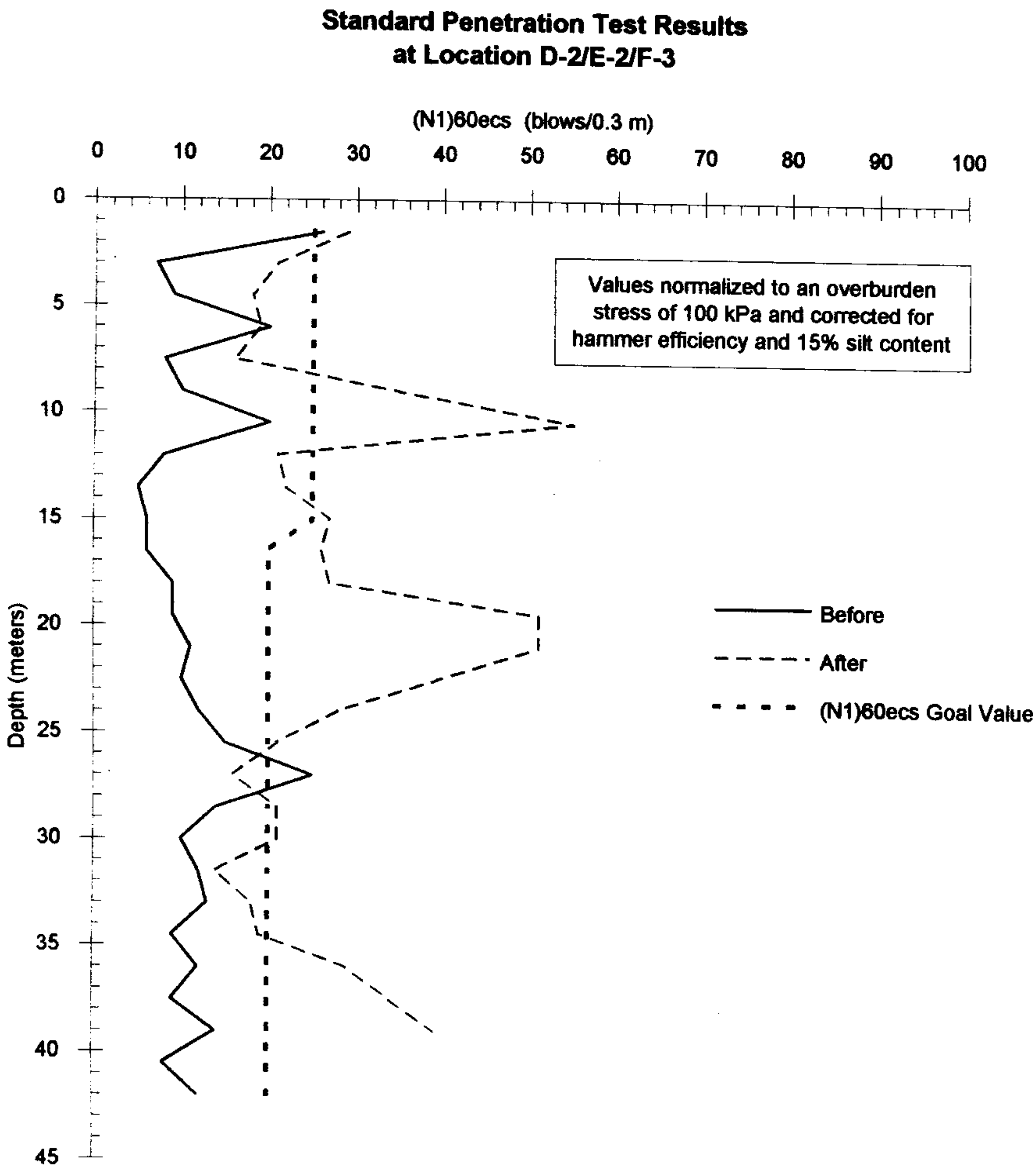

Standard Penetration Test Results
at Location D-2/E-2/F-3
(N1)60ecs (blows/0.3 m)
0
10
20
30
40
50
60
70
80
90
100
Depth (meters)
0
5
10
15
20
25
30
35
40
45
Values normalized to an overburden stress of 100 kPa and corrected for hammer efficiency and 15% silt content
Before
After
(N1)60ecs Goal Value

FIGURE 5

Standard Penetration Test Results at Location L-4/M-4

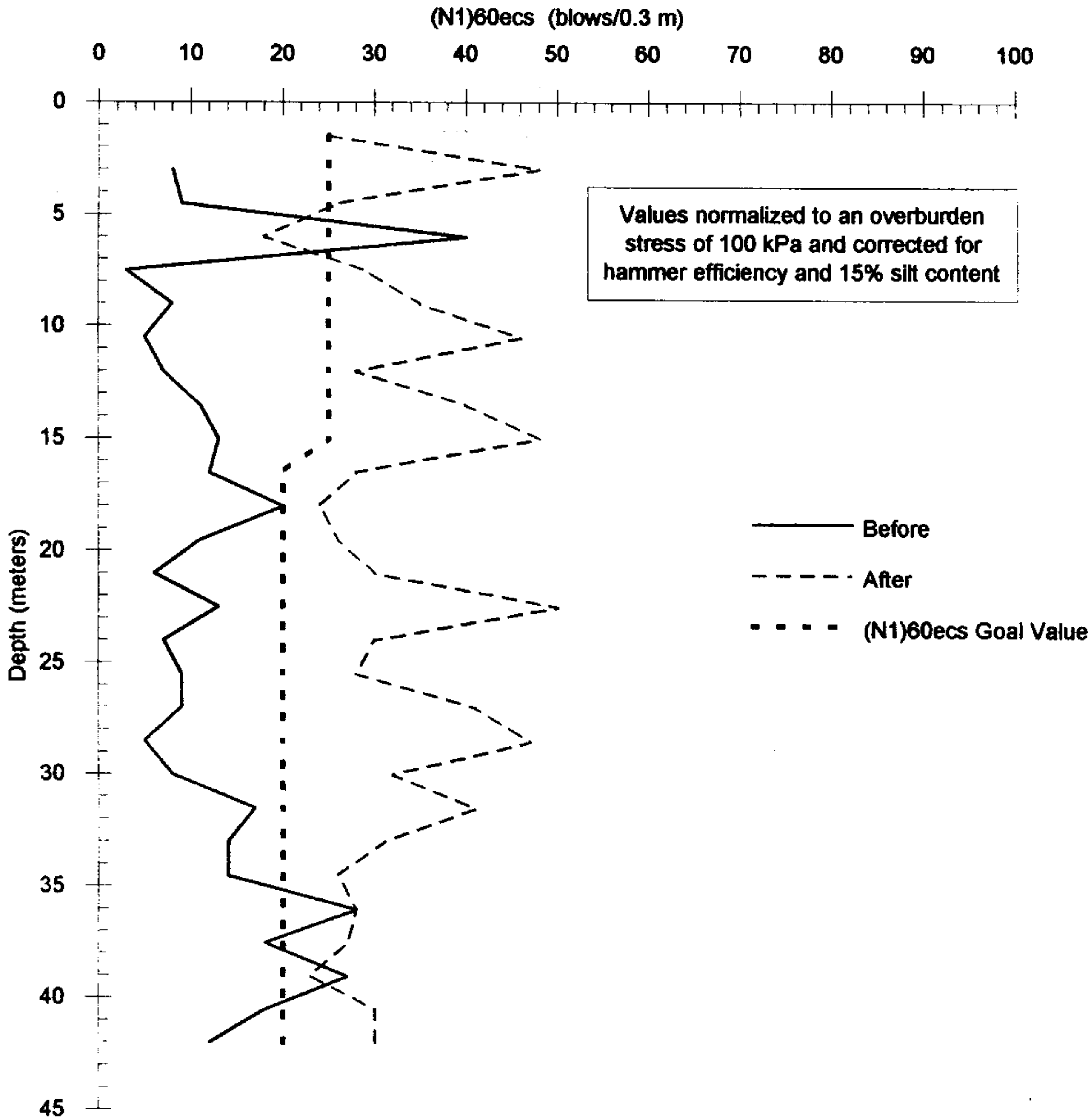


FIGURE 6

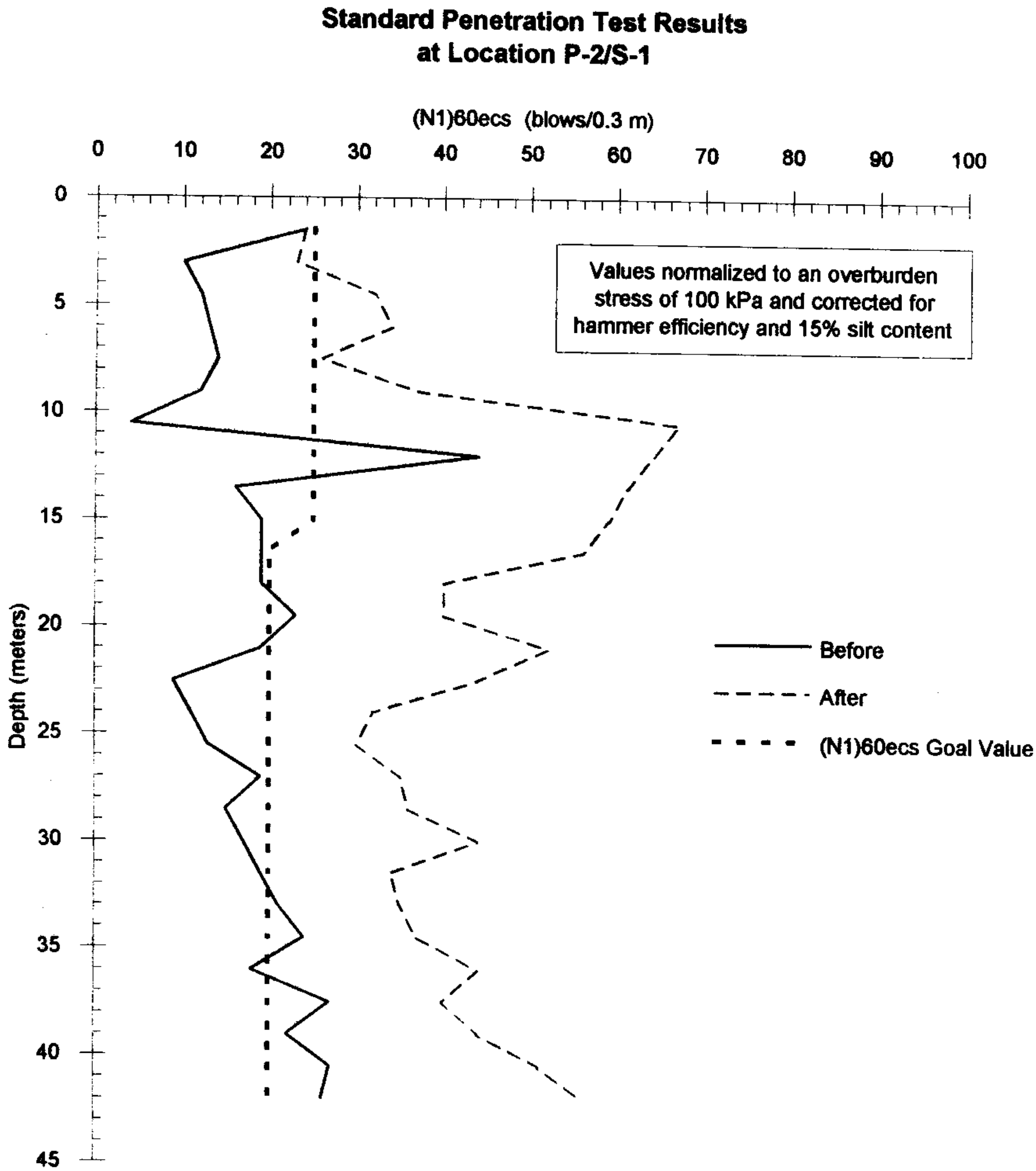

Standard Penetration Test Results
at Location P-2/S-1
(N1)60ecs (blows/0.3 m)
0
10
20
30
40
50
60
70
80
90
100
Depth (meters)
0
5
10
15
20
25
30
35
40
45
Values normalized to an overburden stress of 100 kPa and corrected for hammer efficiency and 15% silt content
Before
After
(N1)60ecs Goal Value

FIGURE 7

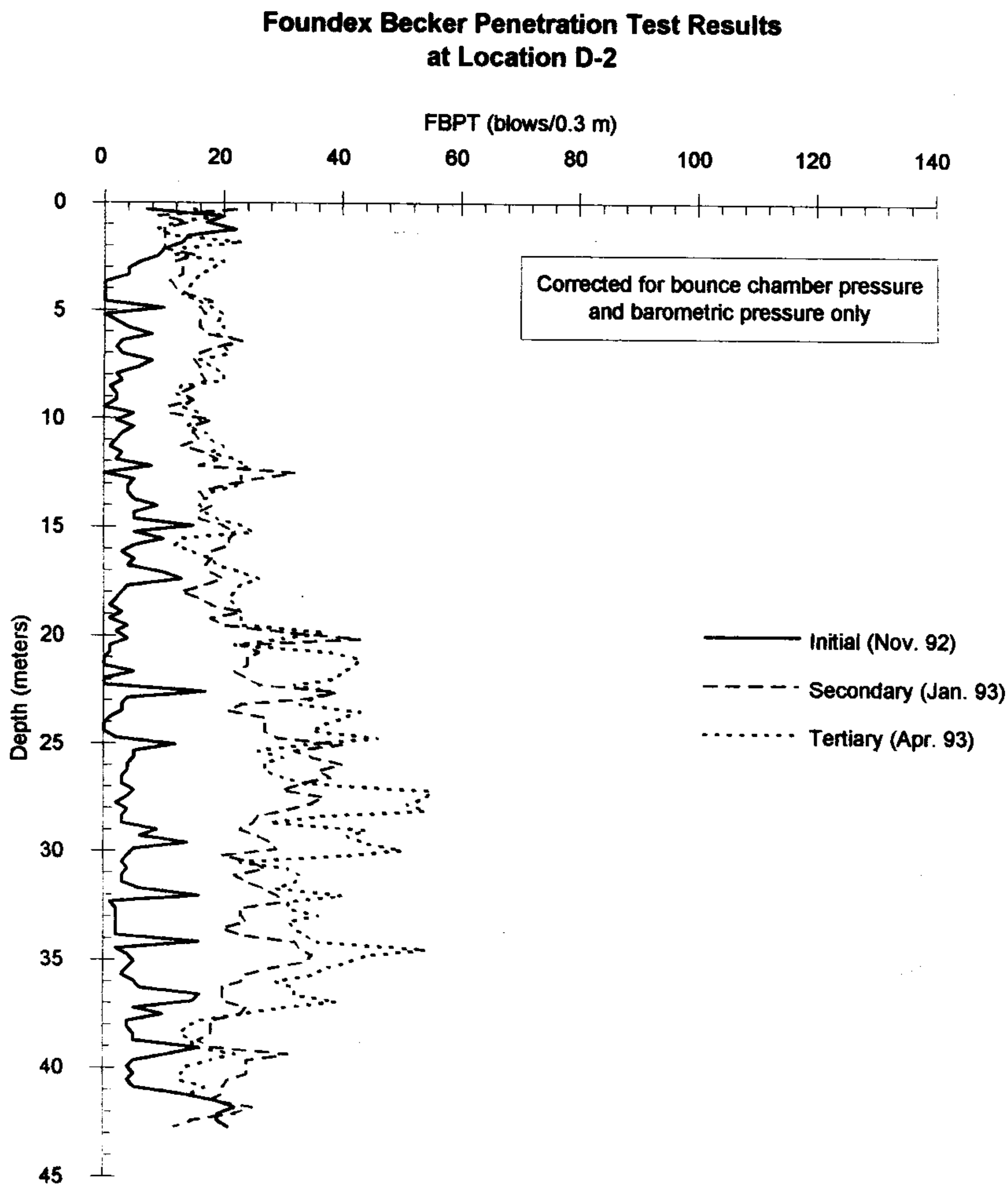

Foundex Becker Penetration Test Results
at Location D-2
FBPT (blows/0.3 m)
0
20
40
60
80
100
120
140
Depth (meters)
0
5
10
15
20
25
30
35
40
45
Corrected for bounce chamber pressure
and barometric pressure only
Initial (Nov. 92)
Secondary (Jan. 93)
Tertiary (Apr. 93)

FIGURE 8

Foundex Becker Penetration Test Results at Location I-3

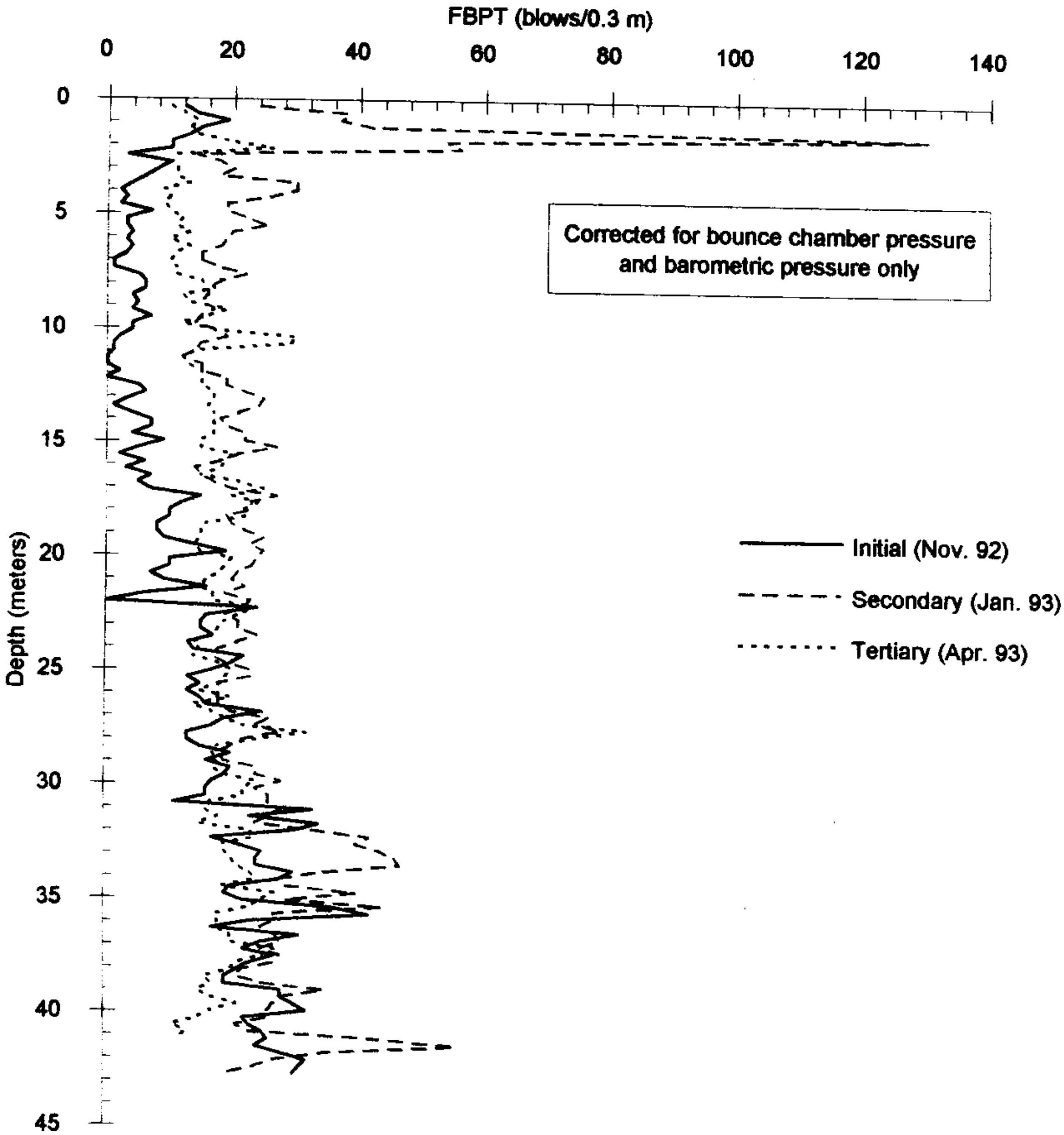


FIGURE 9

Foundex Becker Penetration Test Results at Location L-4

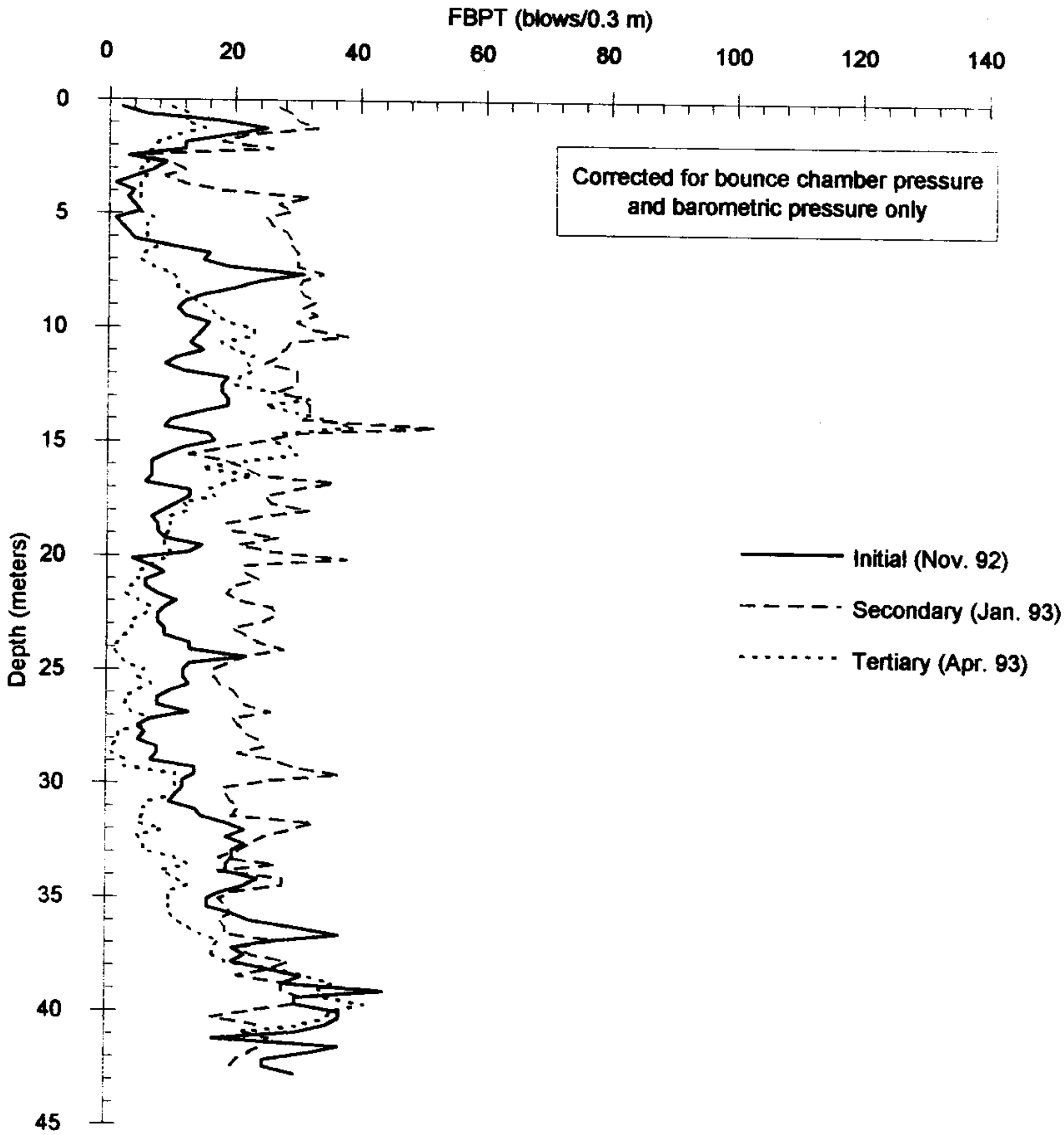


FIGURE 10

Foundex Becker Penetration Test Results at Location P-2

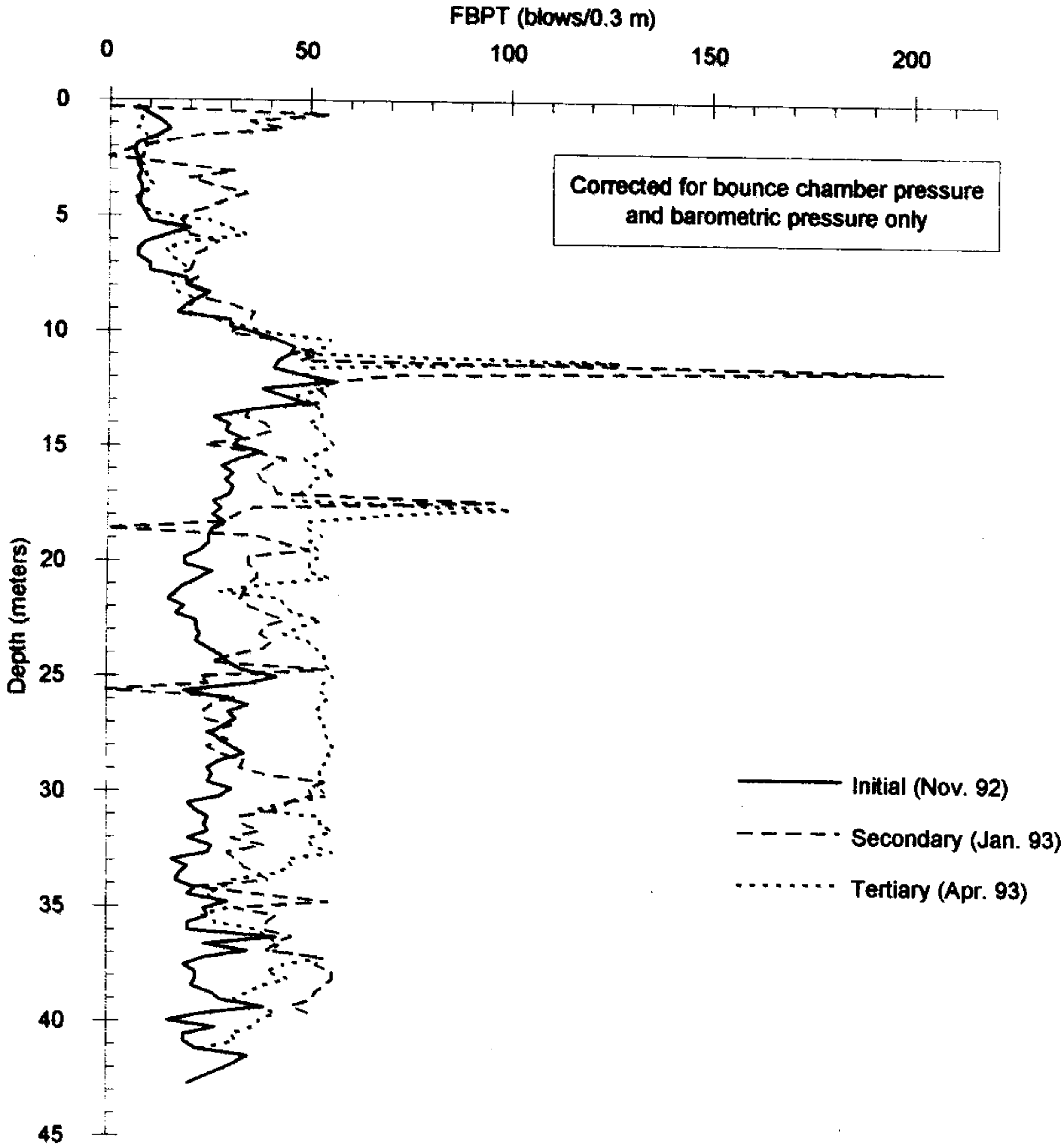


FIGURE 11

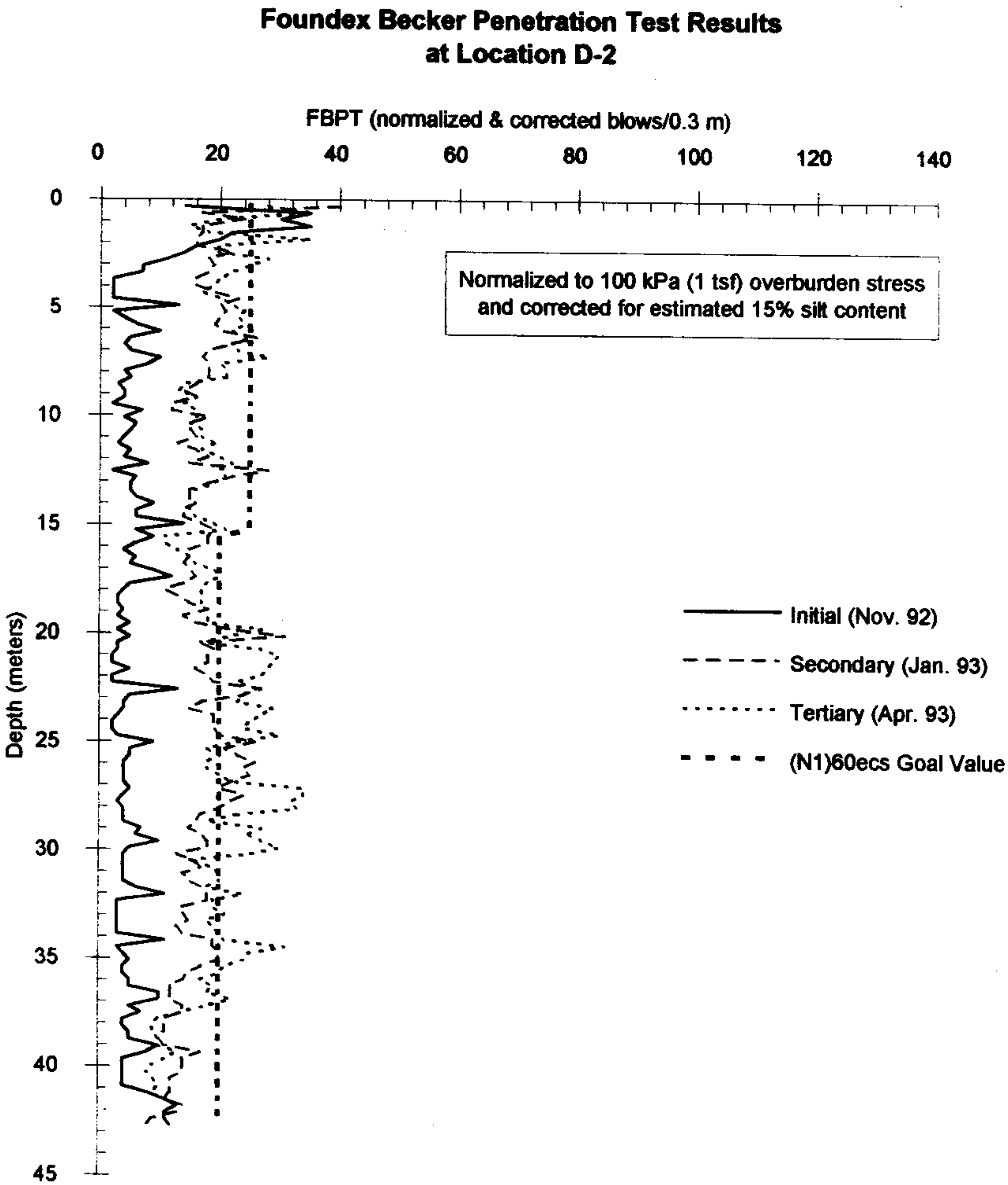

Foundex Becker Penetration Test Results
at Location D-2
FBPT (normalized & corrected blows/0.3 m)
0
20
40
60
80
100
120
140
Depth (meters)
0
5
10
15
20
25
30
35
40
45
Normalized to 100 kPa (1 tsf) overburden stress
and corrected for estimated 15% silt content
Initial (Nov. 92)
Secondary (Jan. 93)
Tertiary (Apr. 93)
(N1)60ecs Goal Value

Figure 12

Foundex Becker Penetration Test Results
at Location I-3

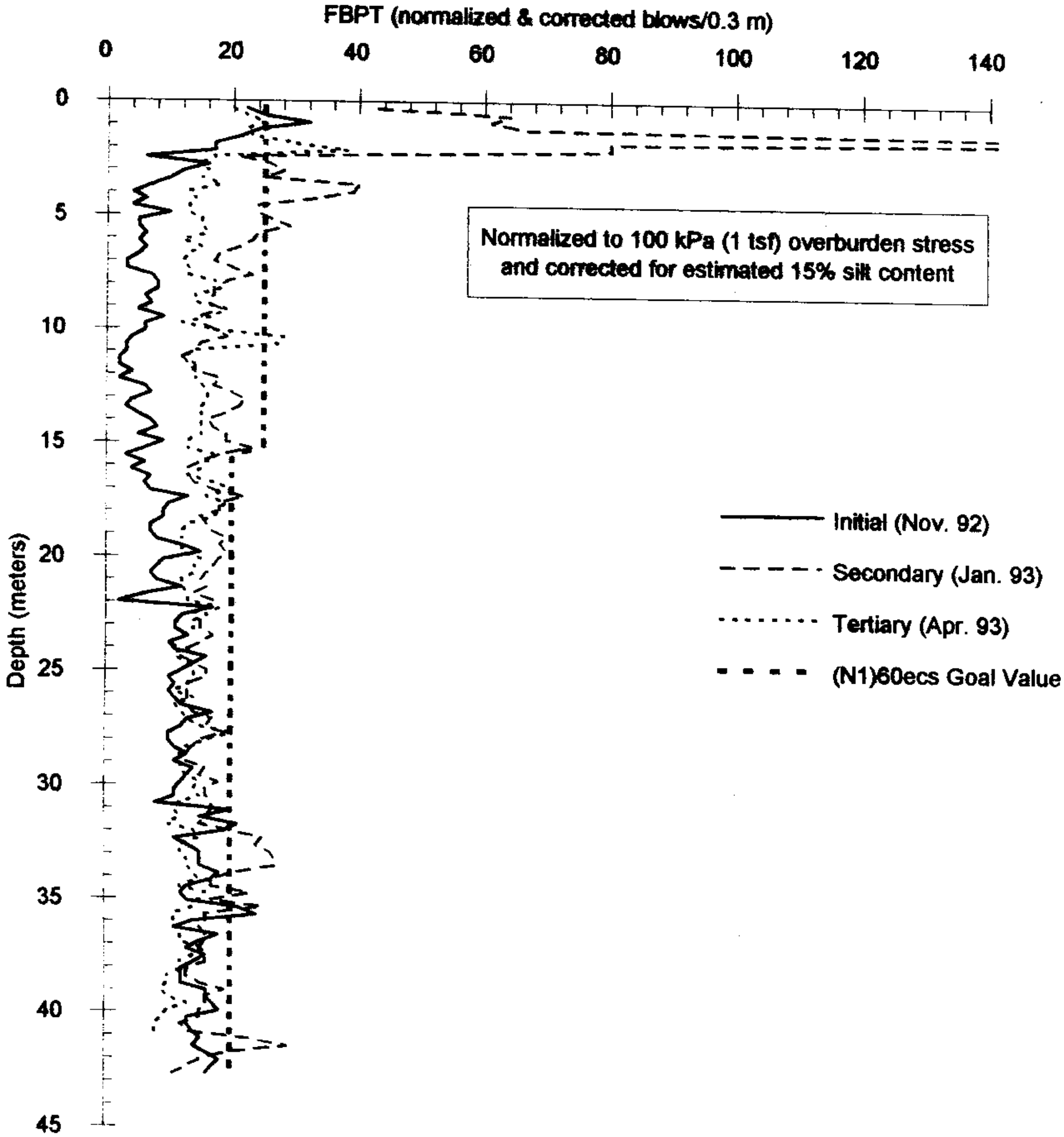


Figure 13

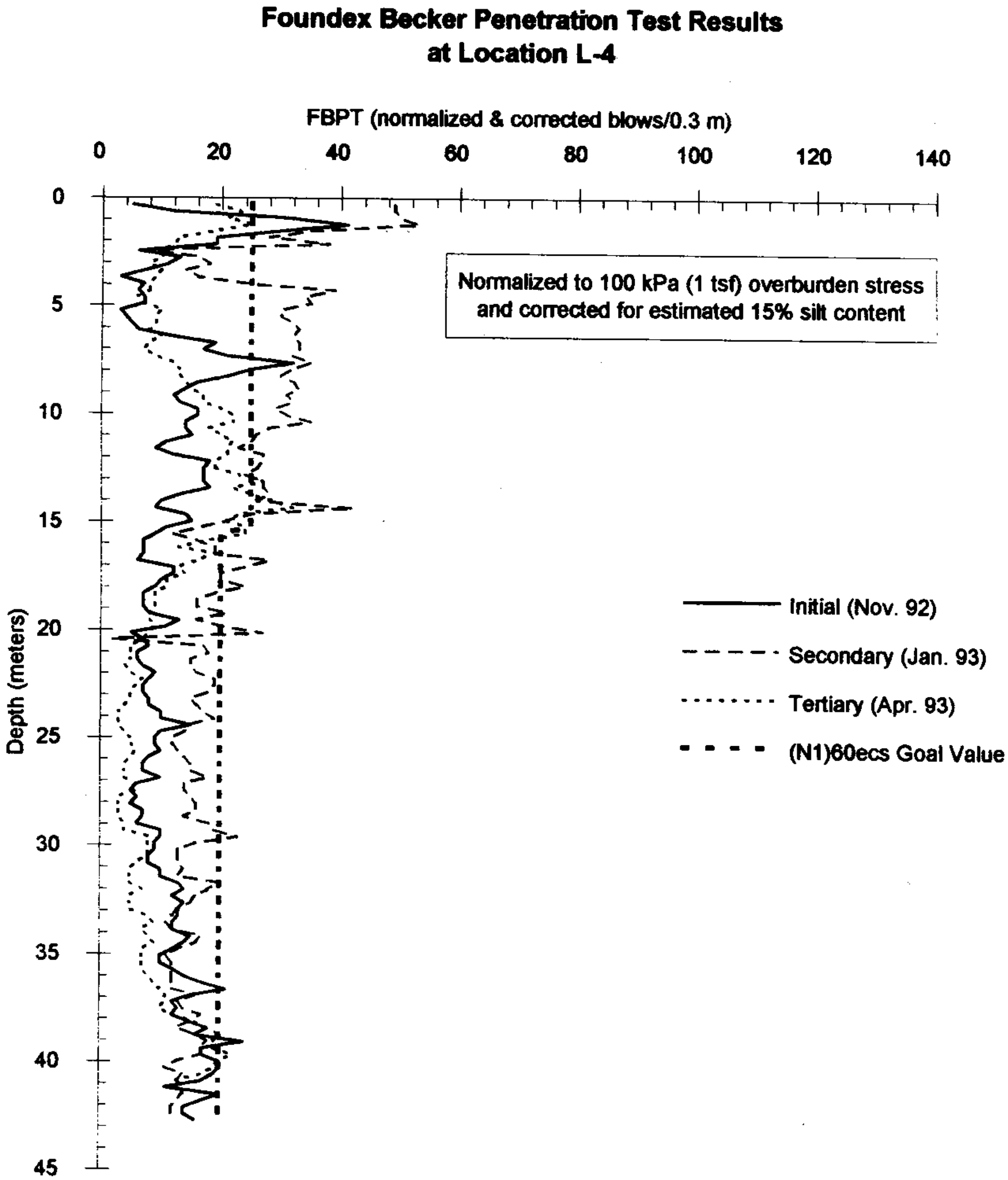

Foundex Becker Penetration Test Results
at Location L-4
FBPT (normalized & corrected blows/0.3 m)
0
20
40
60
80
100
120
140
0
5
10
15
20
25
30
35
40
45
Depth (meters)
Normalized to 100 kPa (1 tsf) overburden stress
and corrected for estimated 15% silt content
Initial (Nov. 92)
Secondary (Jan. 93)
Tertiary (Apr. 93)
(N1)60ecs Goal Value

Figure 14

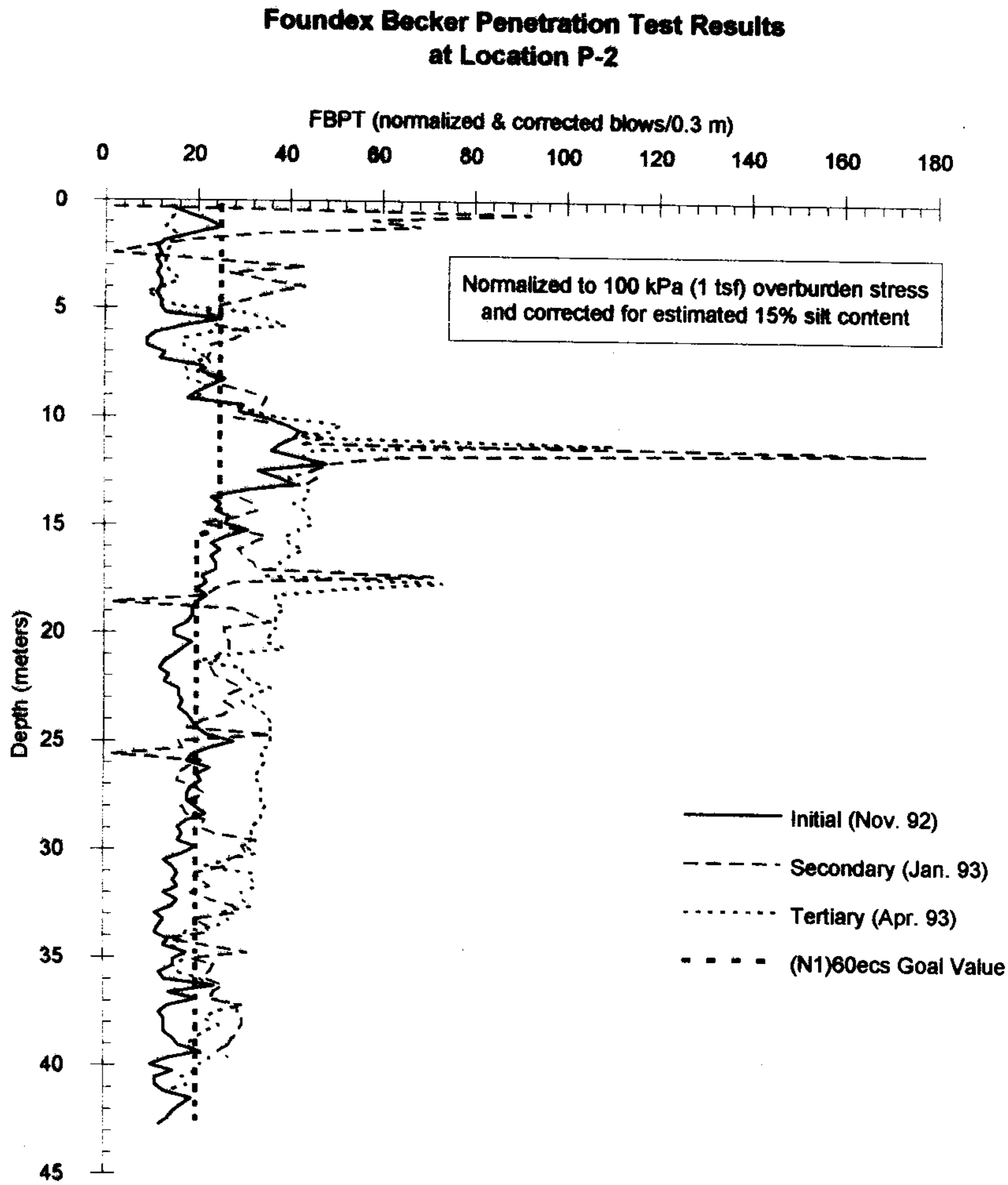

Foundex Becker Penetration Test Results
at Location P-2
FBPT (normalized & corrected blows/0.3 m)
0
20
40
60
80
100
120
140
160
180
Depth (meters)
0
5
10
15
20
25
30
35
40
45
Normalized to 100 kPa (1 tsf) overburden stress
and corrected for estimated 15% silt content
Initial (Nov. 92)
Secondary (Jan. 93)
Tertiary (Apr. 93)
(N1)60ecs Goal Value

Figure 15

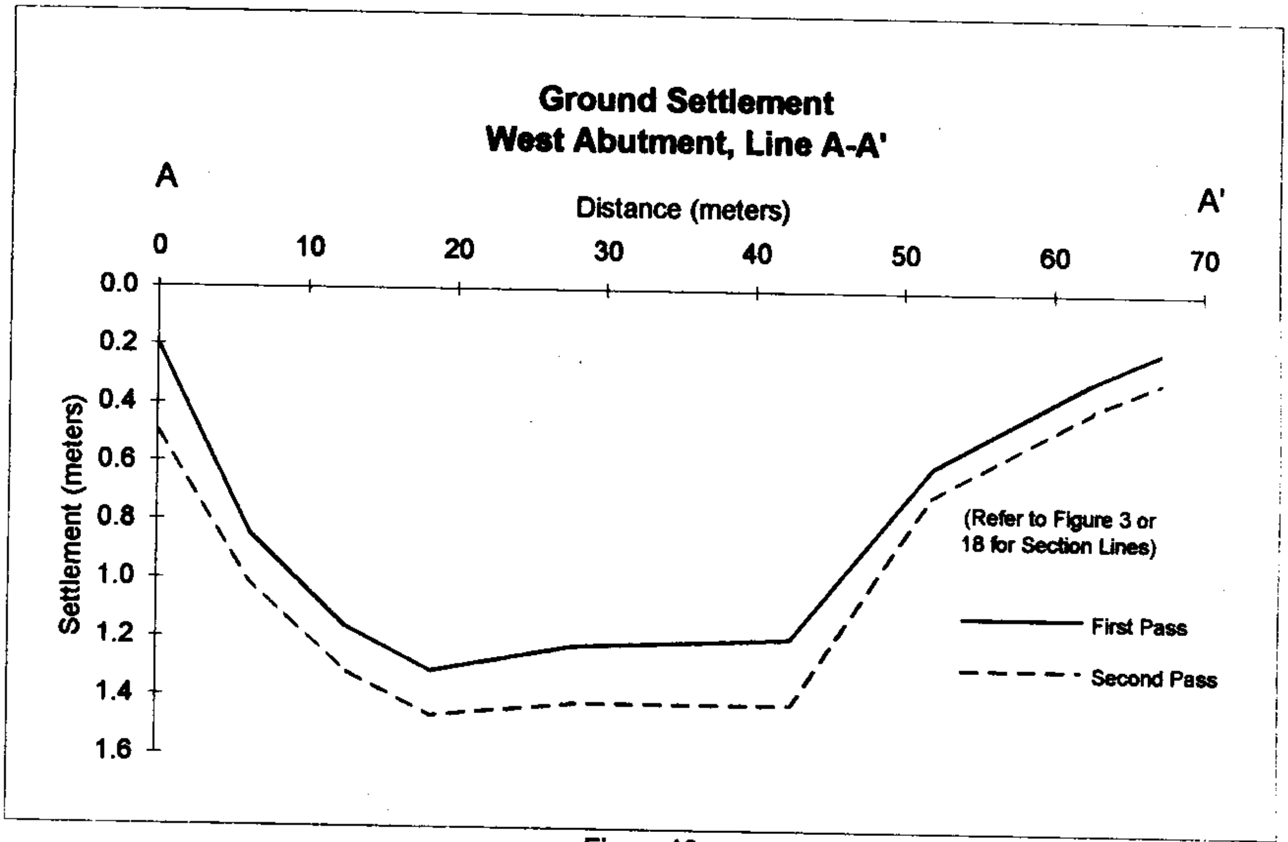


Figure 16

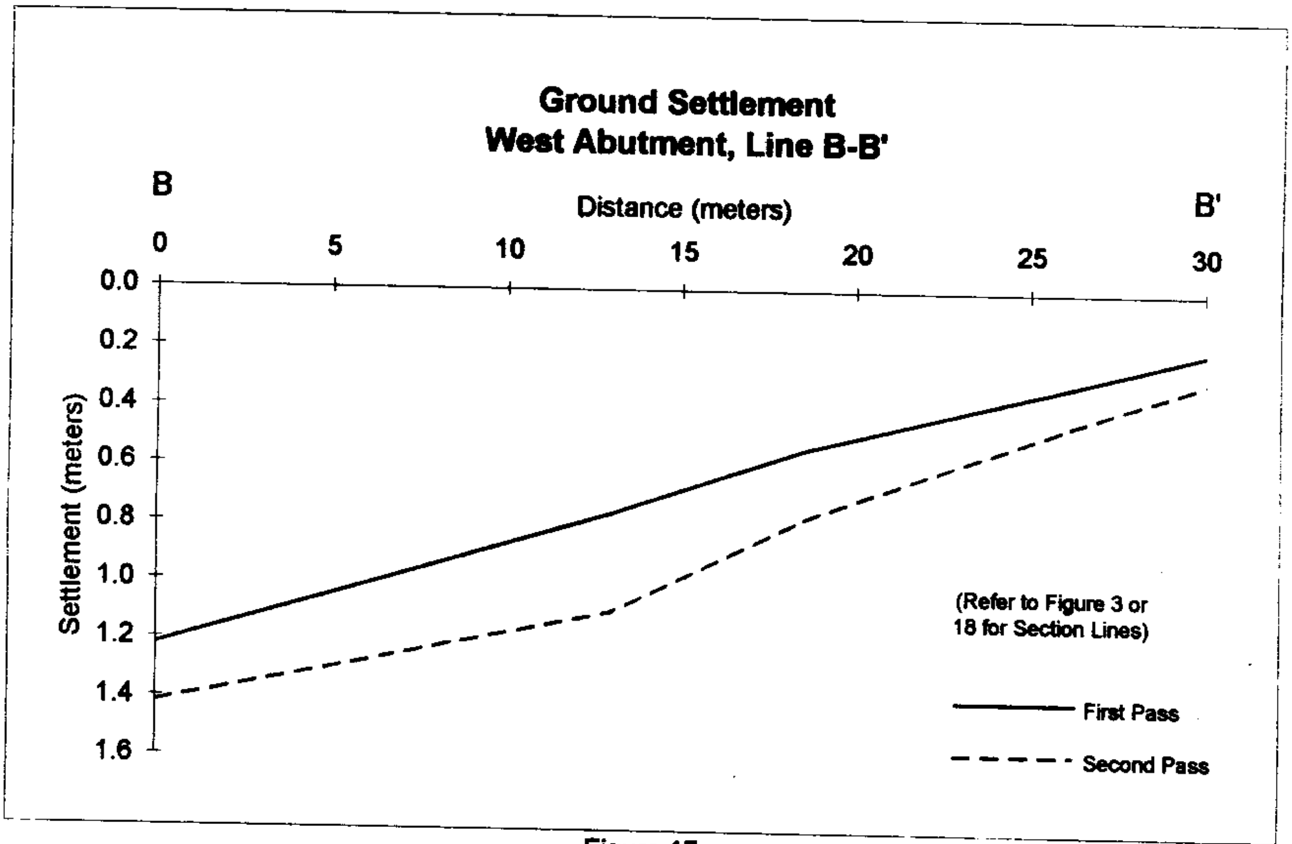


Figure 17

SETTLEMENT CONTOUR MAP
West Abutment

Contour Interval: 0.2 meters

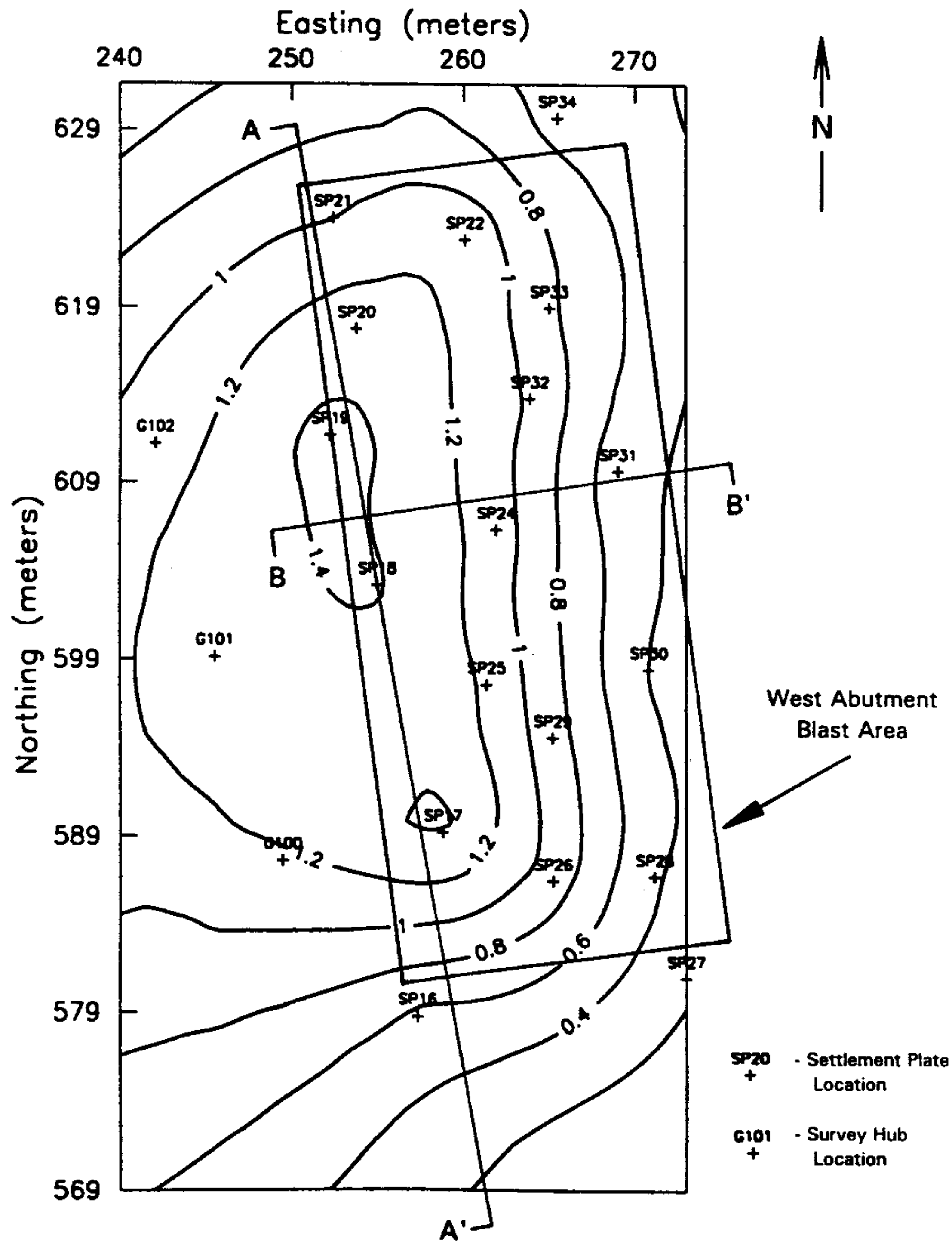


Figure 18

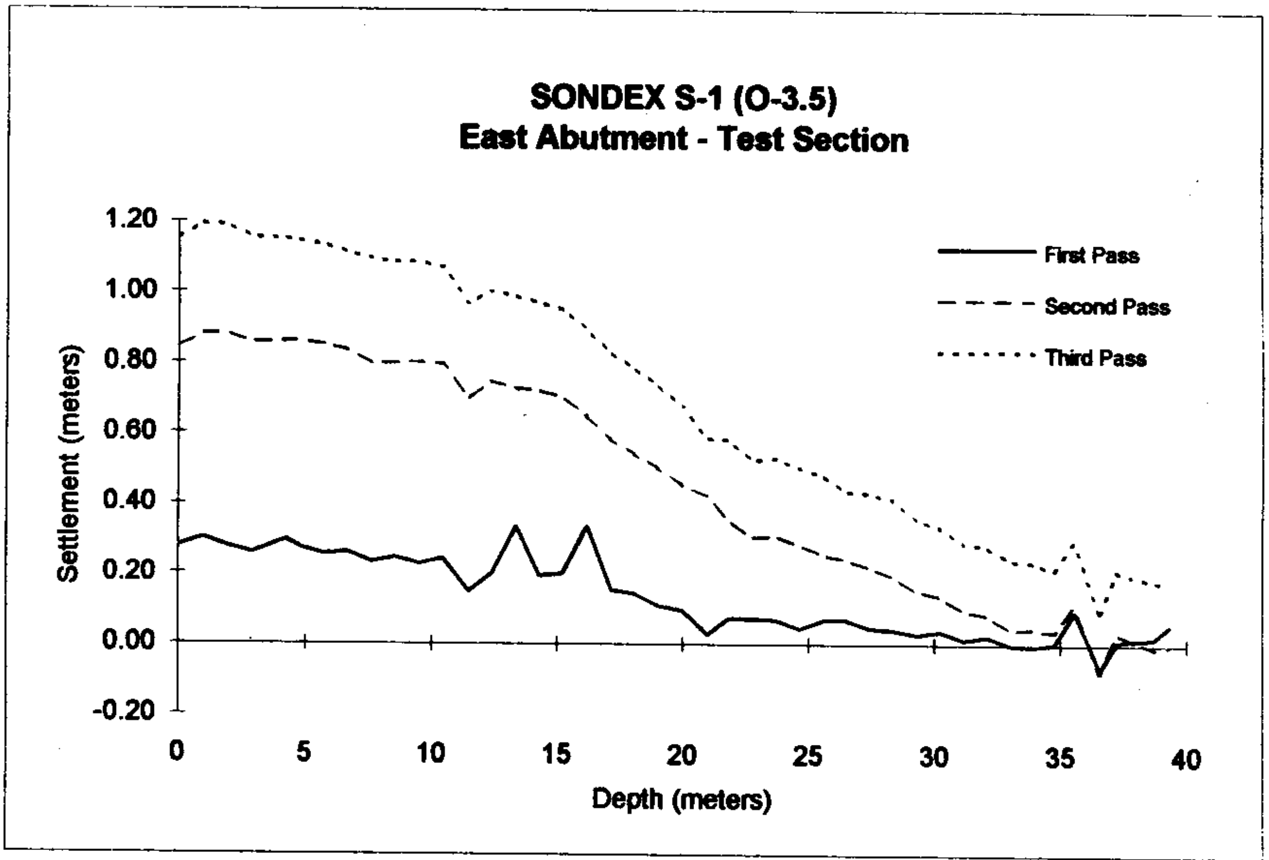


Figure 19

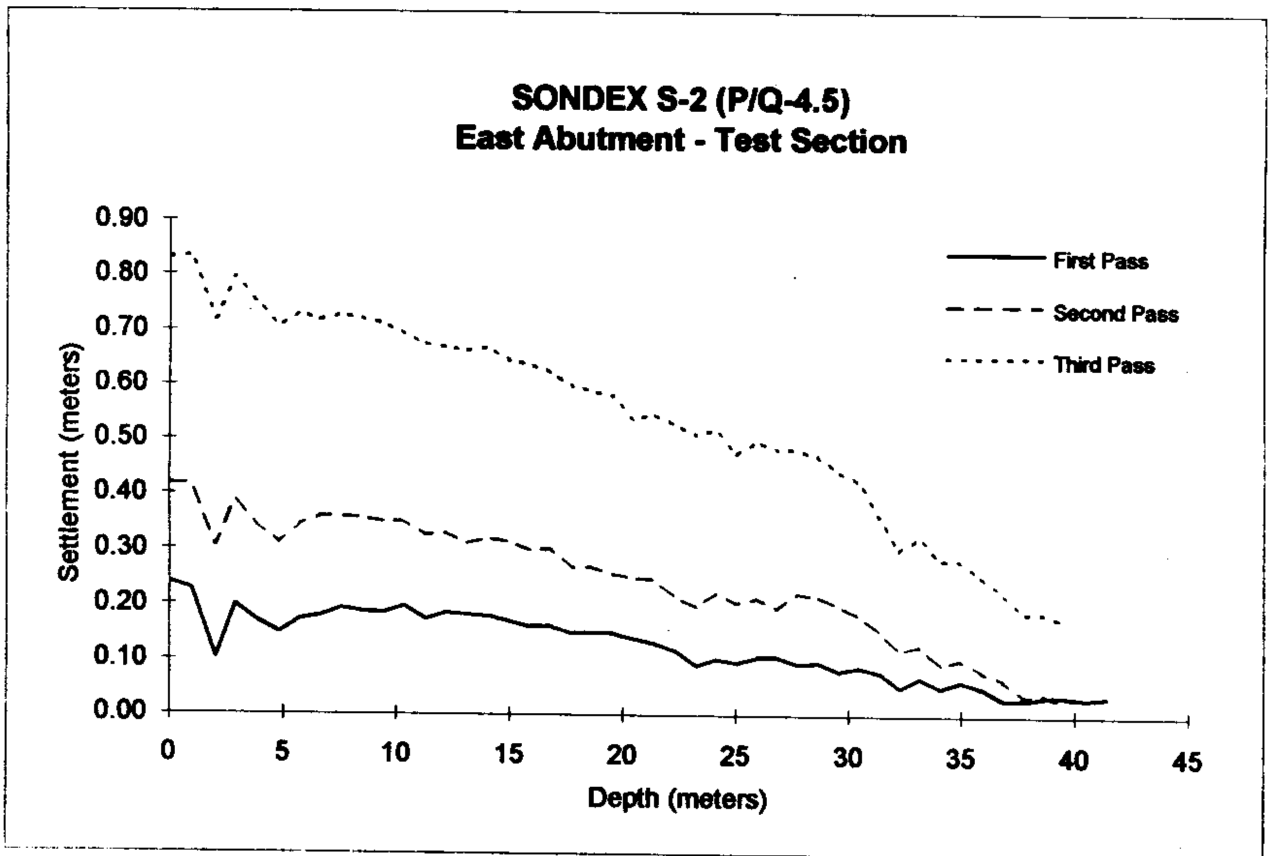


Figure 20

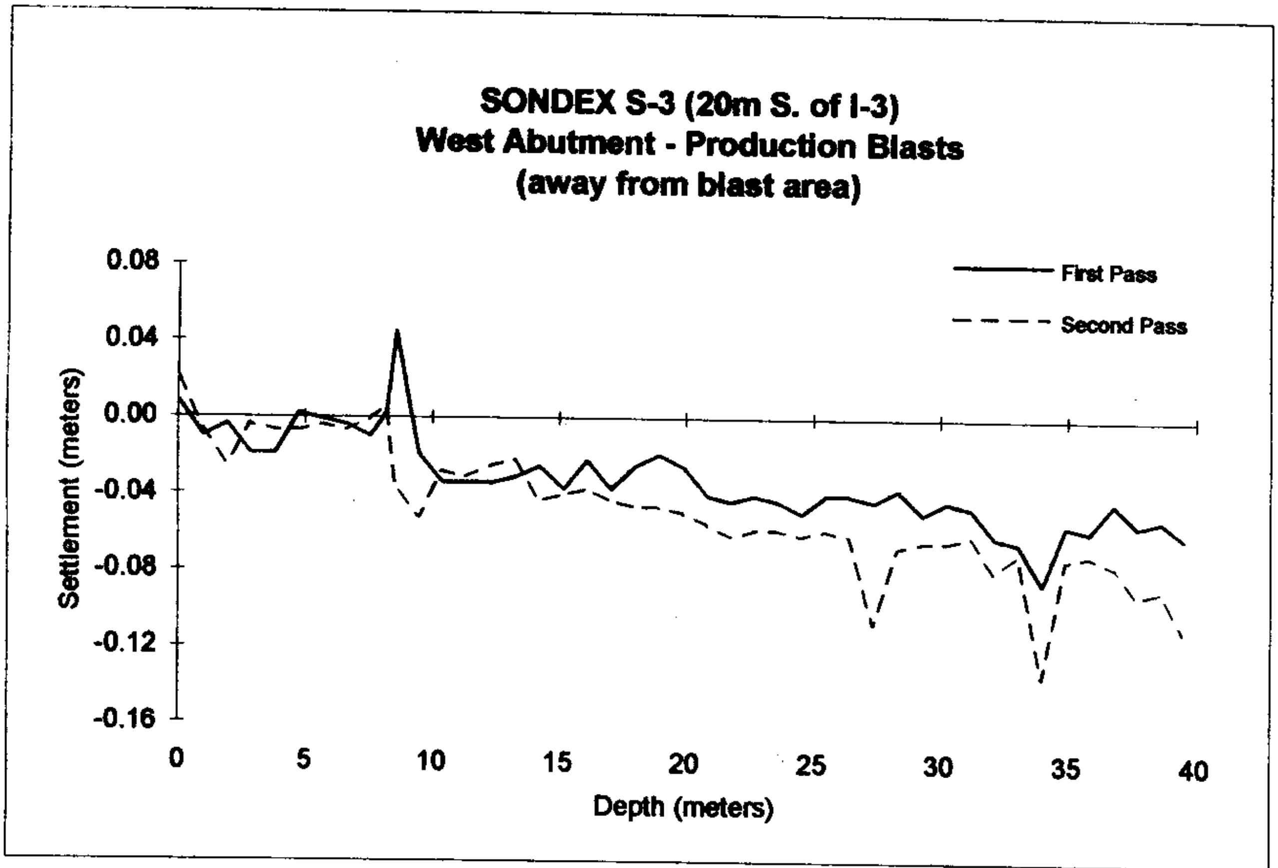


Figure 21

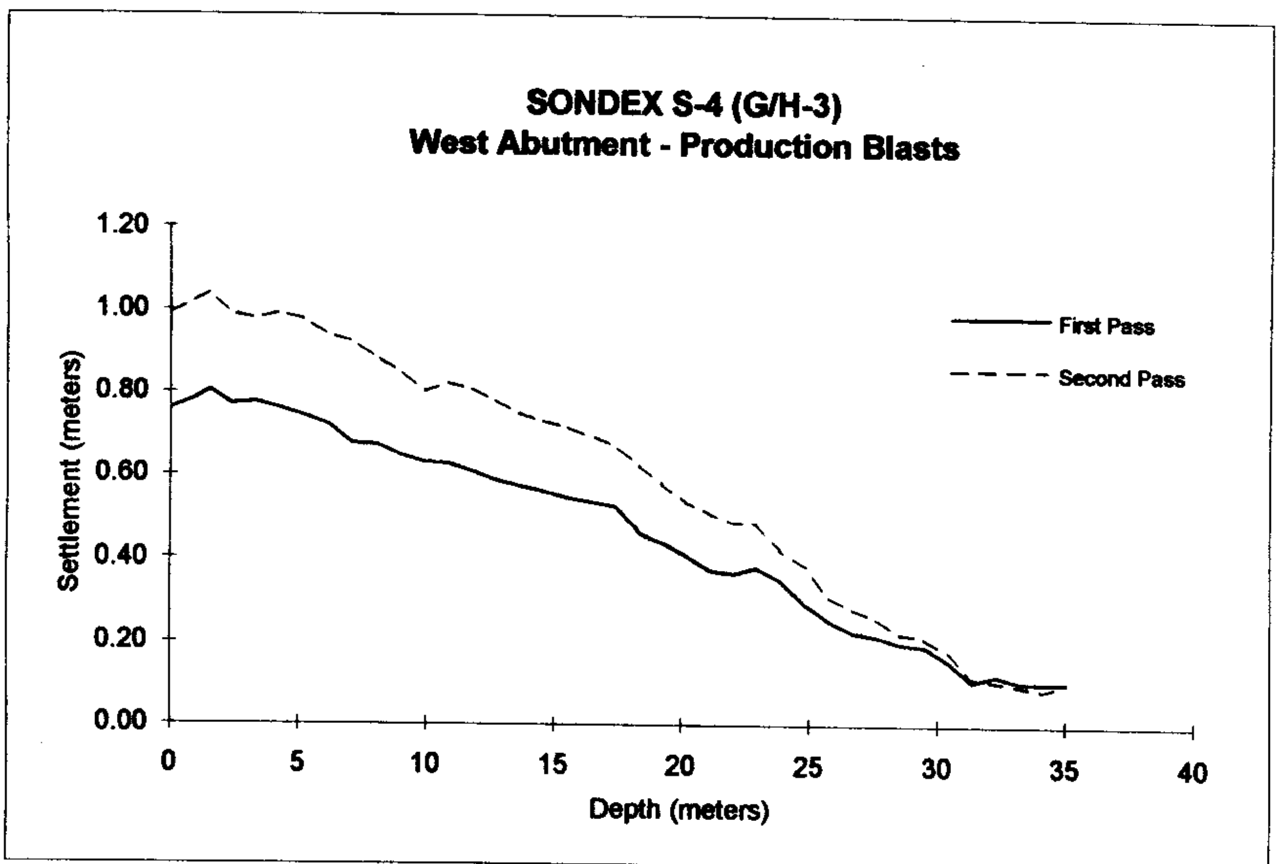


Figure 22

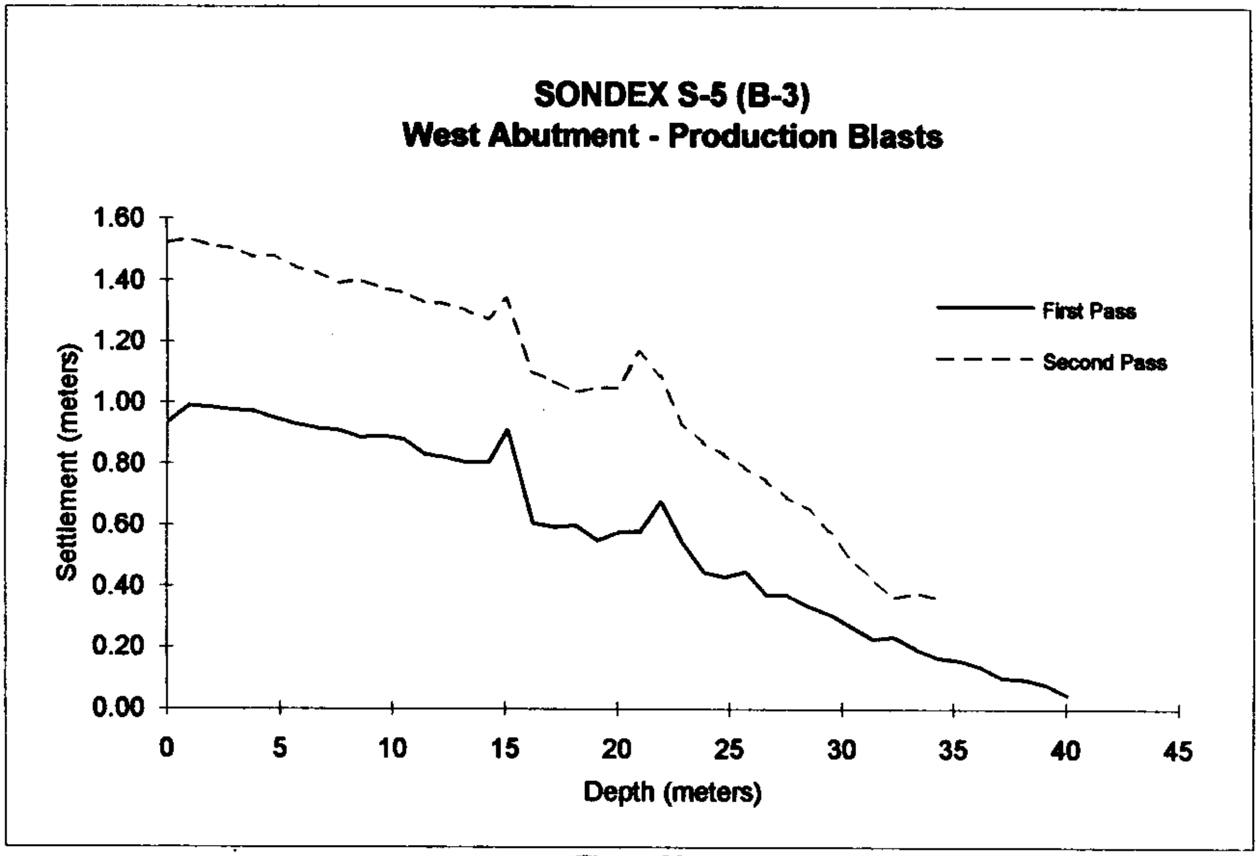


Figure 23

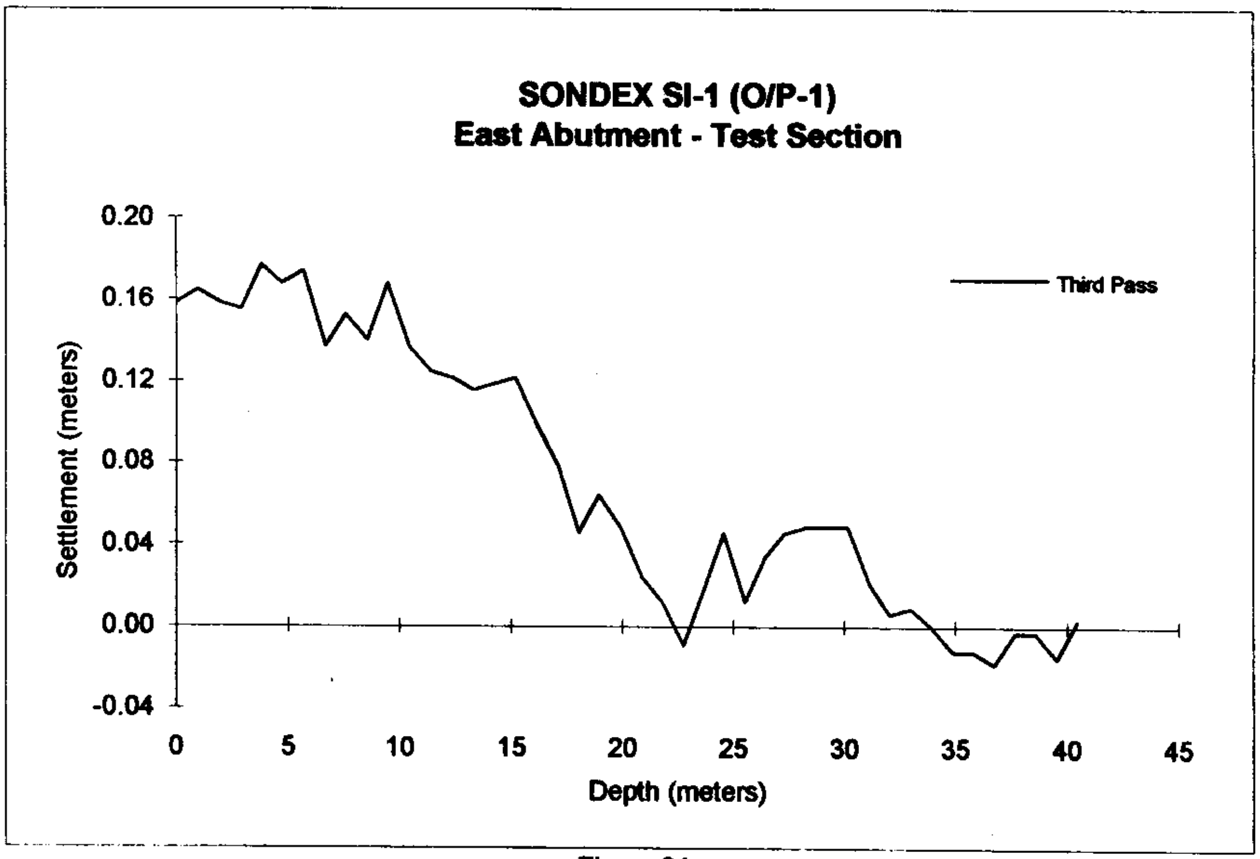


Figure 24

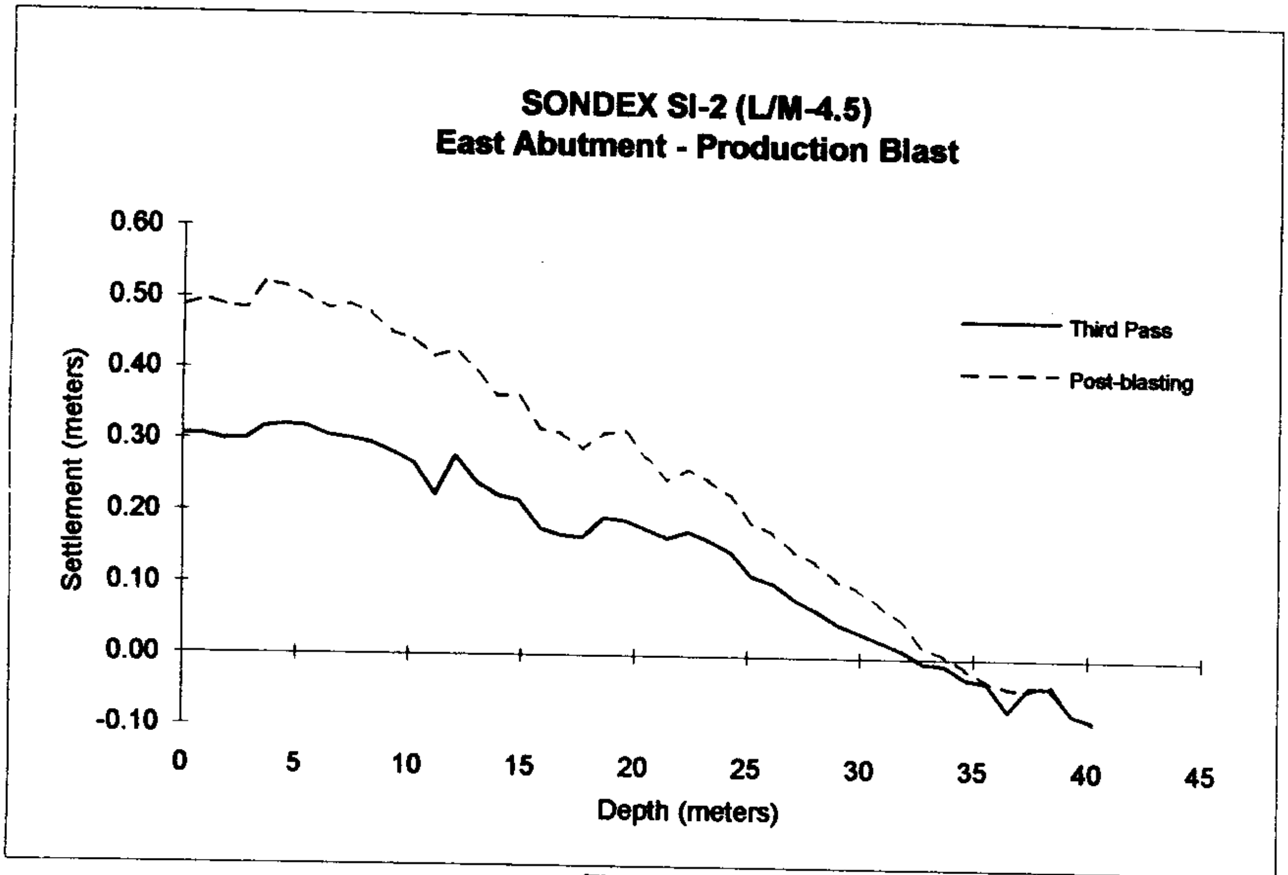


Figure 25

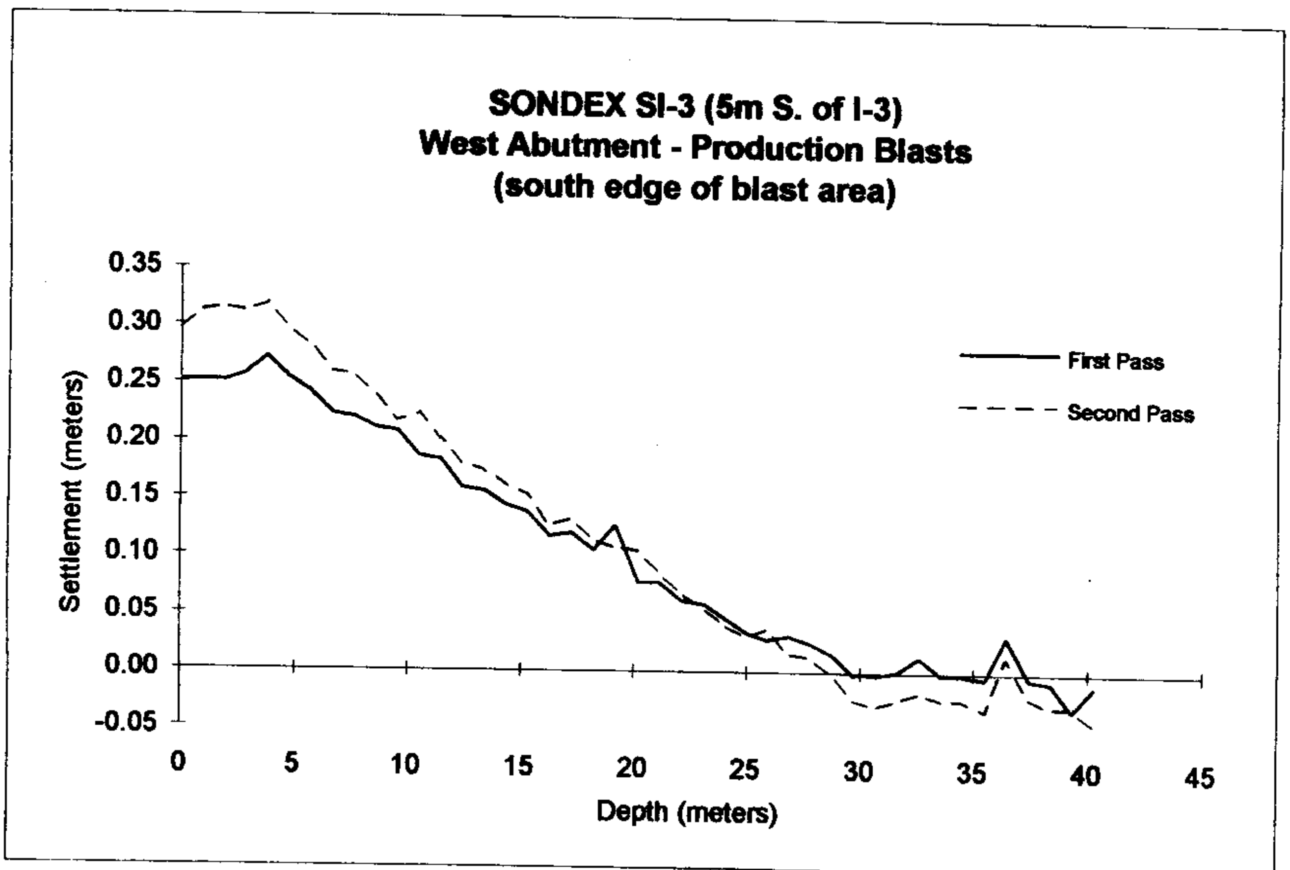


Figure 26

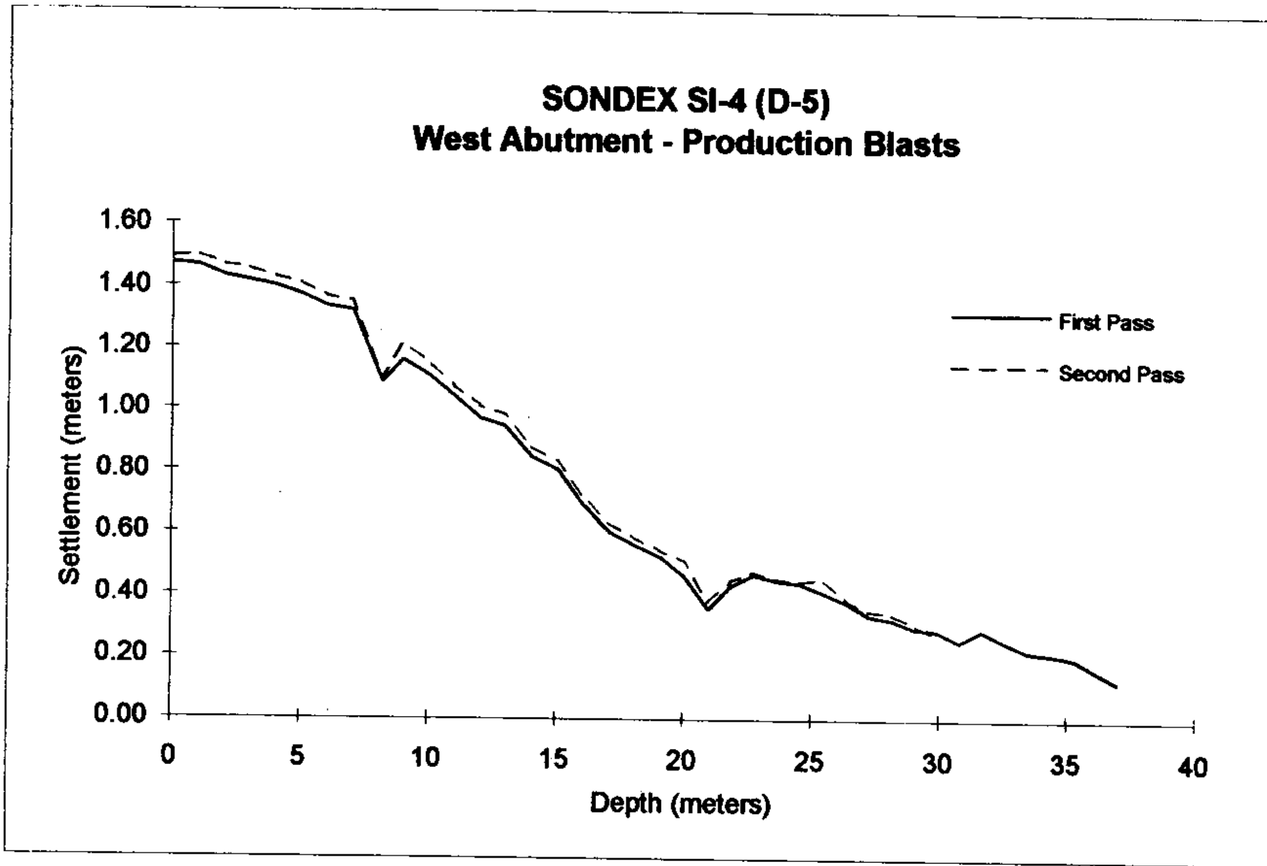


Figure 27

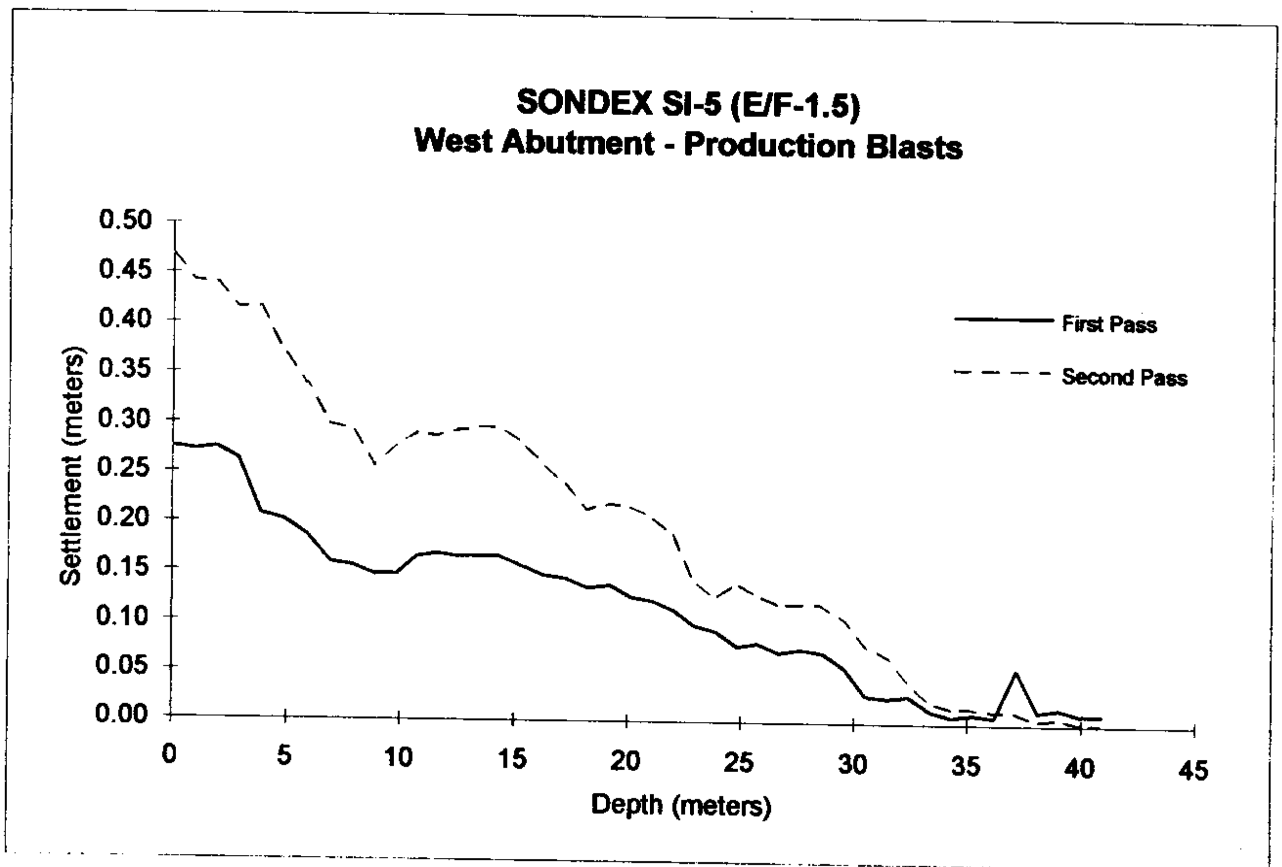


Figure 28

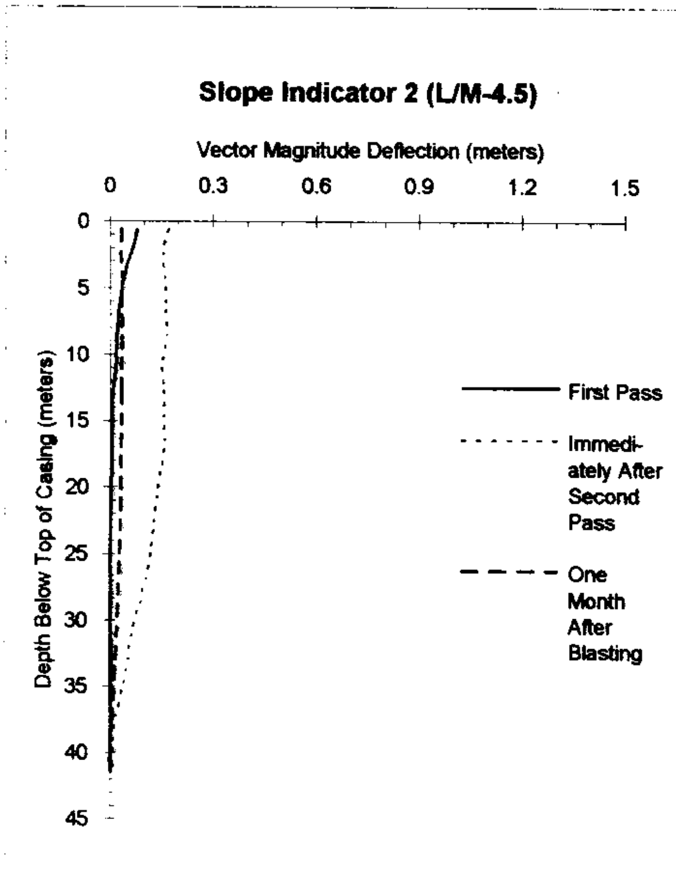


Figure 29

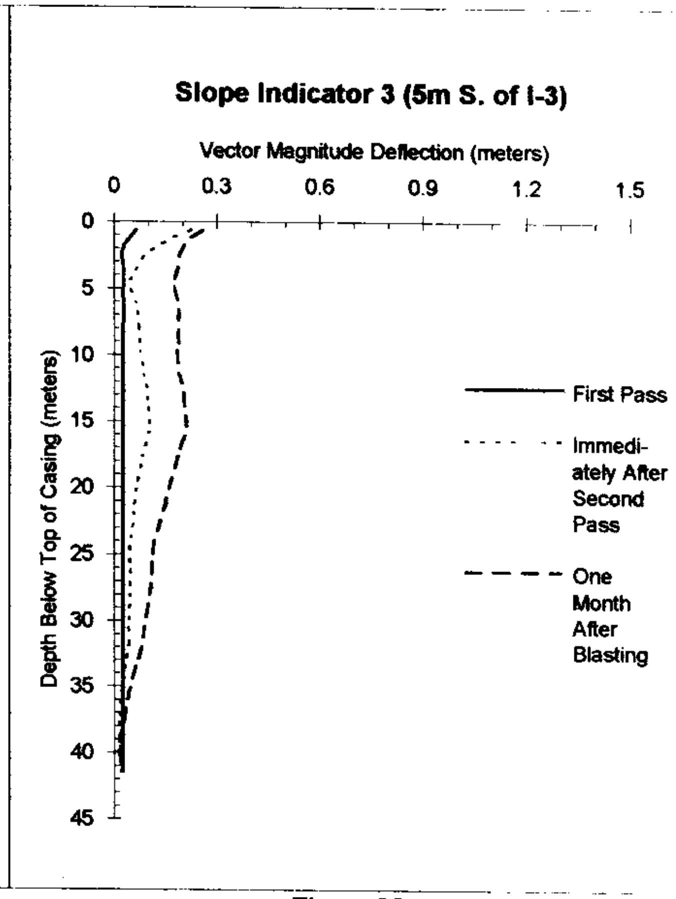
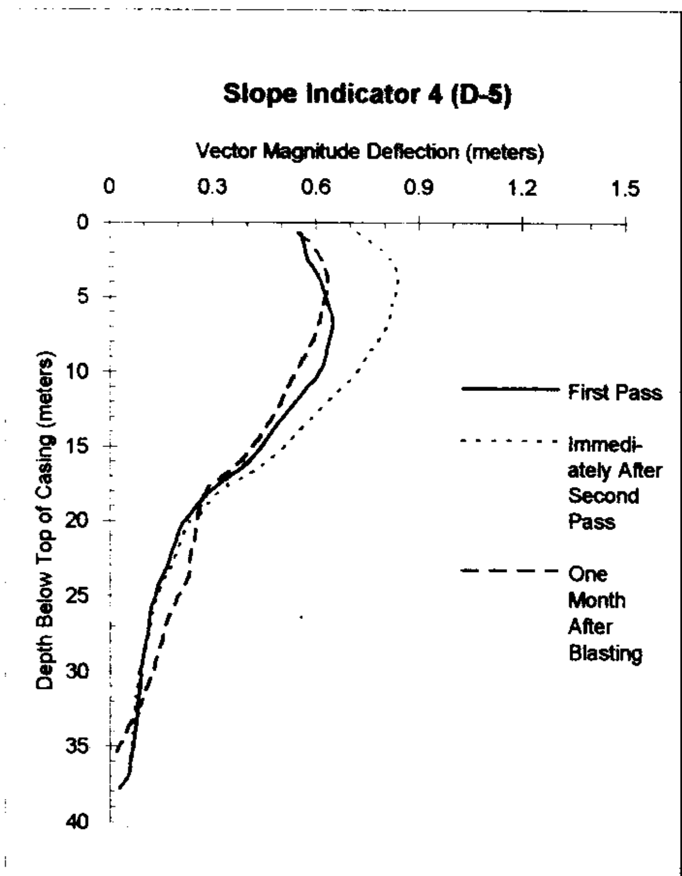


Figure 30



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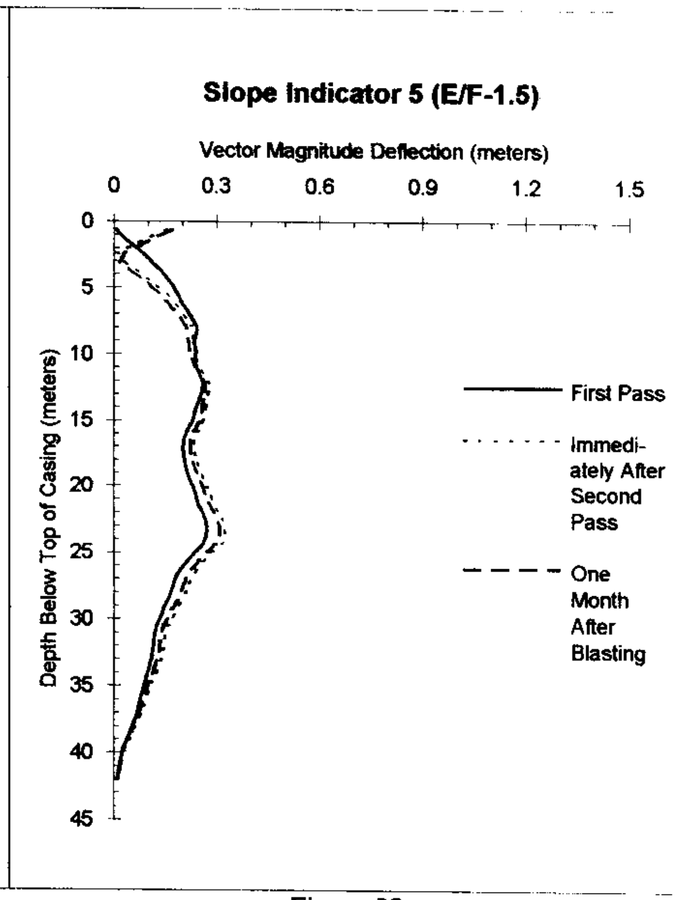


Figure 32

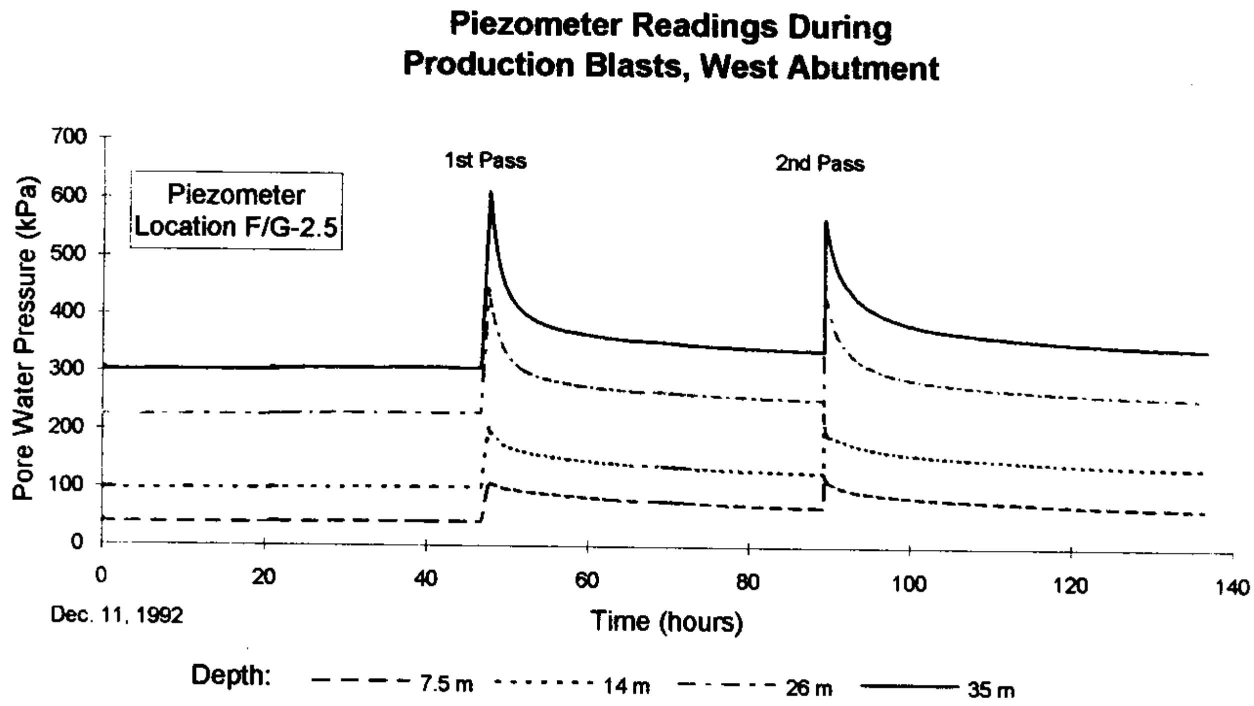


Figure 33a

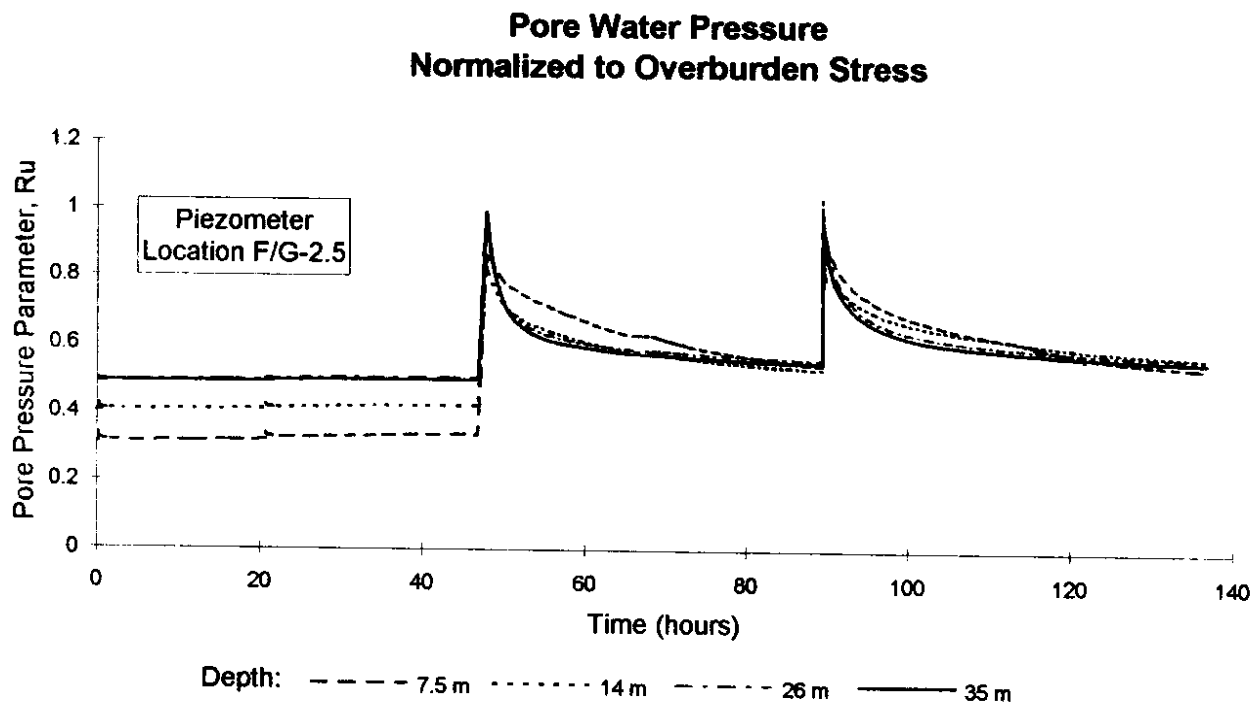
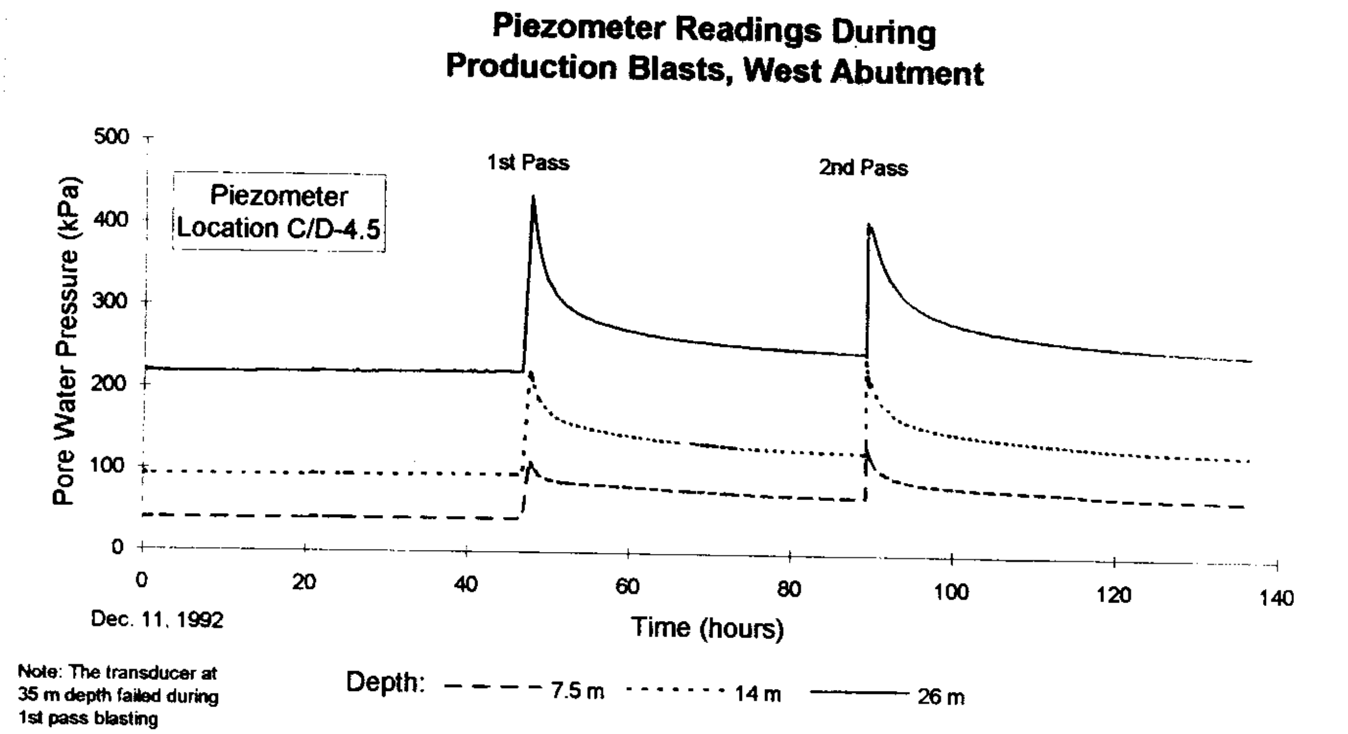


Figure 33b



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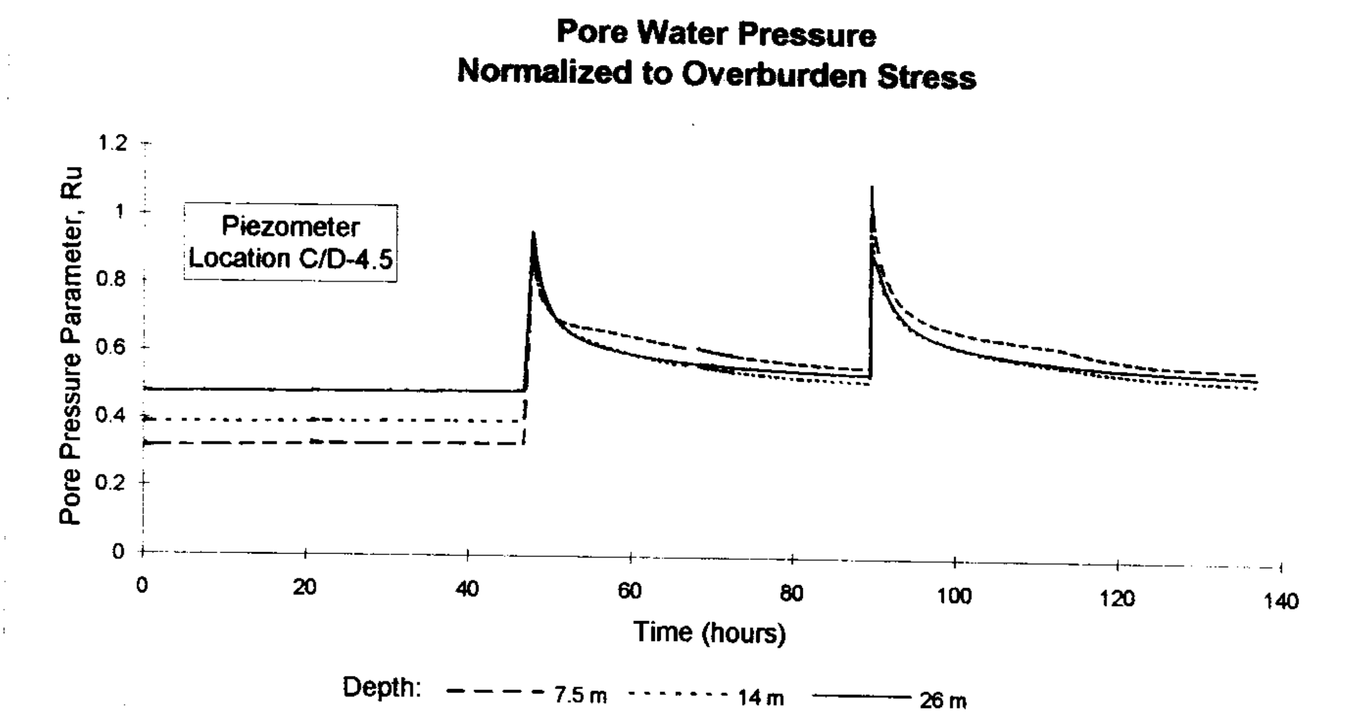


Figure 34b

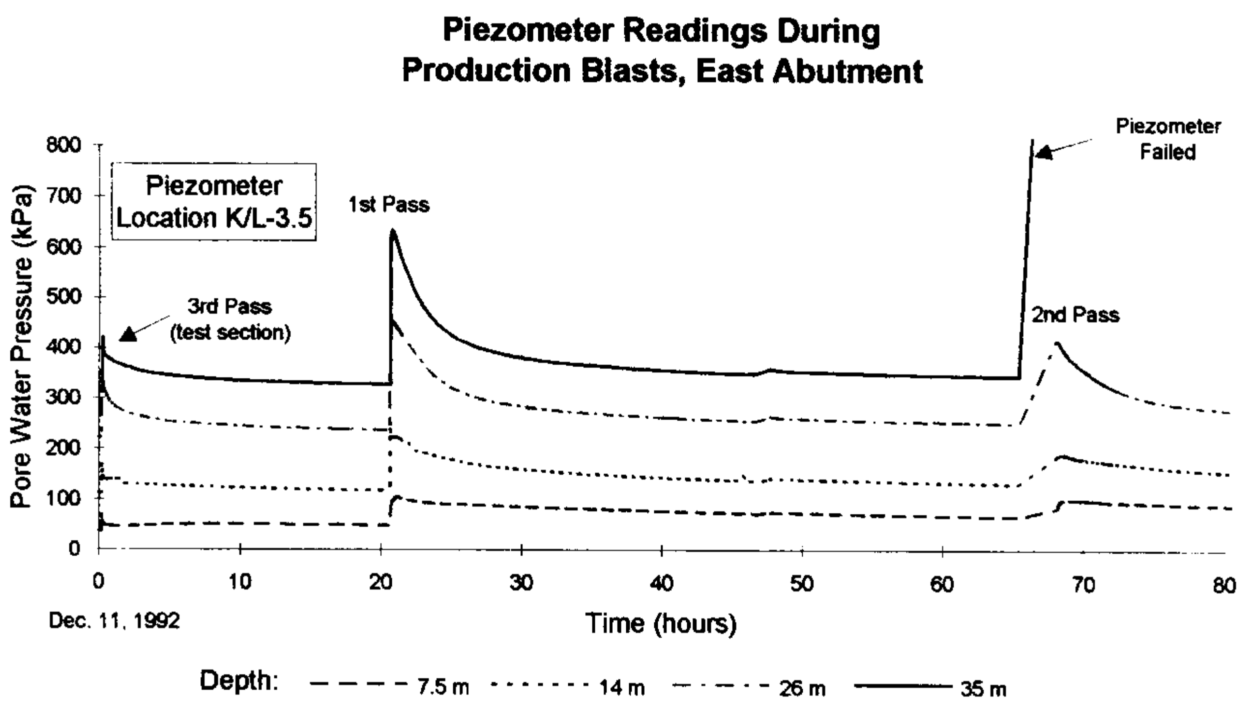


Figure 35a

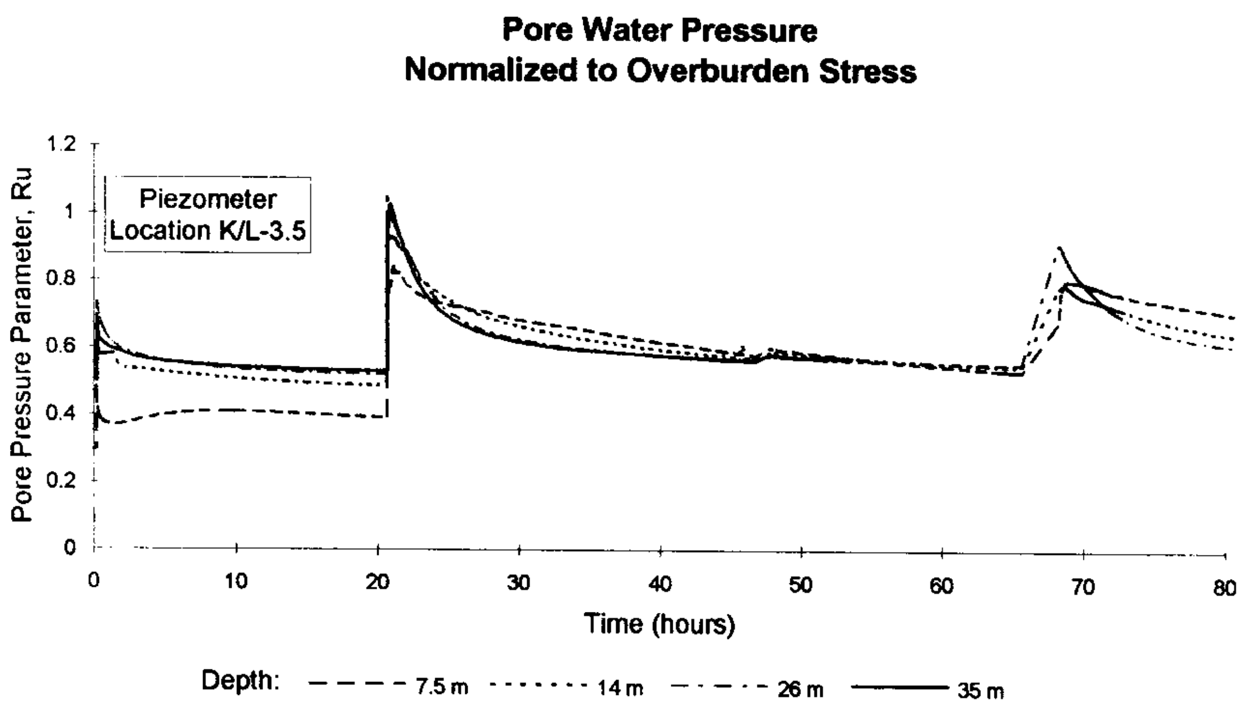


Figure 35b

PHOTOGRAPHS

BRIDGE 12 SITE

Photo 1 — Site location. Dashed line shows alignment of the Coldwater Lake Outlet to Johnston Ridge segment of the Spirit Lake Memorial Highway (SR 504).

Photo 2 — Becker Hammer Drilling Rig.

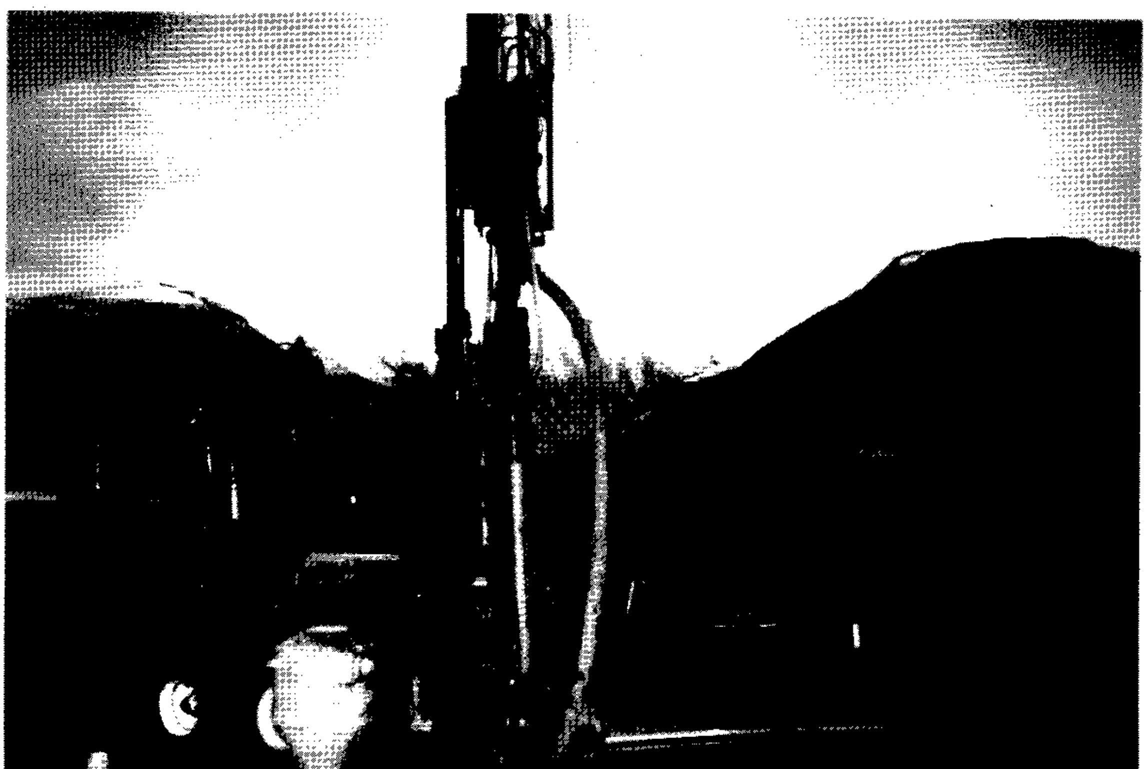


Photo 3 — Primary blast of Test Section, East Abutment, November 9, 1992. The two spouts are the initial flow of silty water from two of the vertical drains. West Abutment is in background. Note the "hummocks" which dominate the natural ground surface topography.

Photo 4 — Further eruption of silty water shortly after blast sequence detonation.

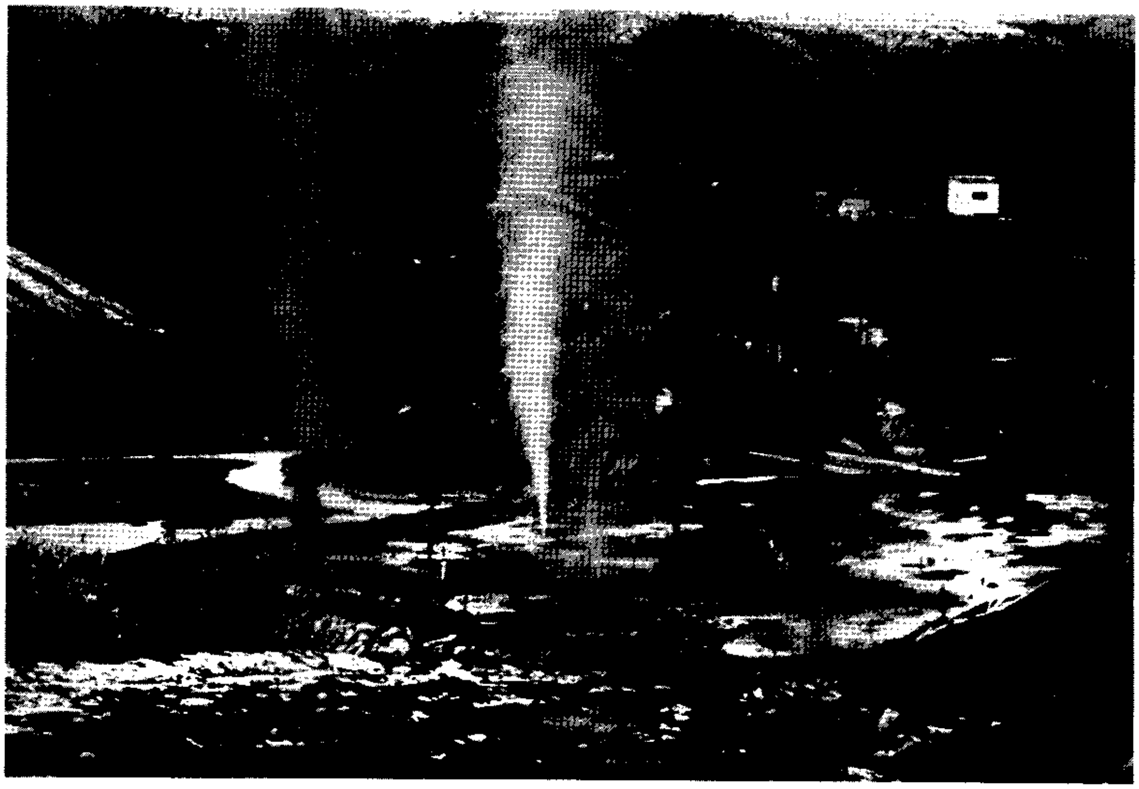


Photo 5 — Further eruption of silty water and exhausting of explosive gases from blast holes (white plume).

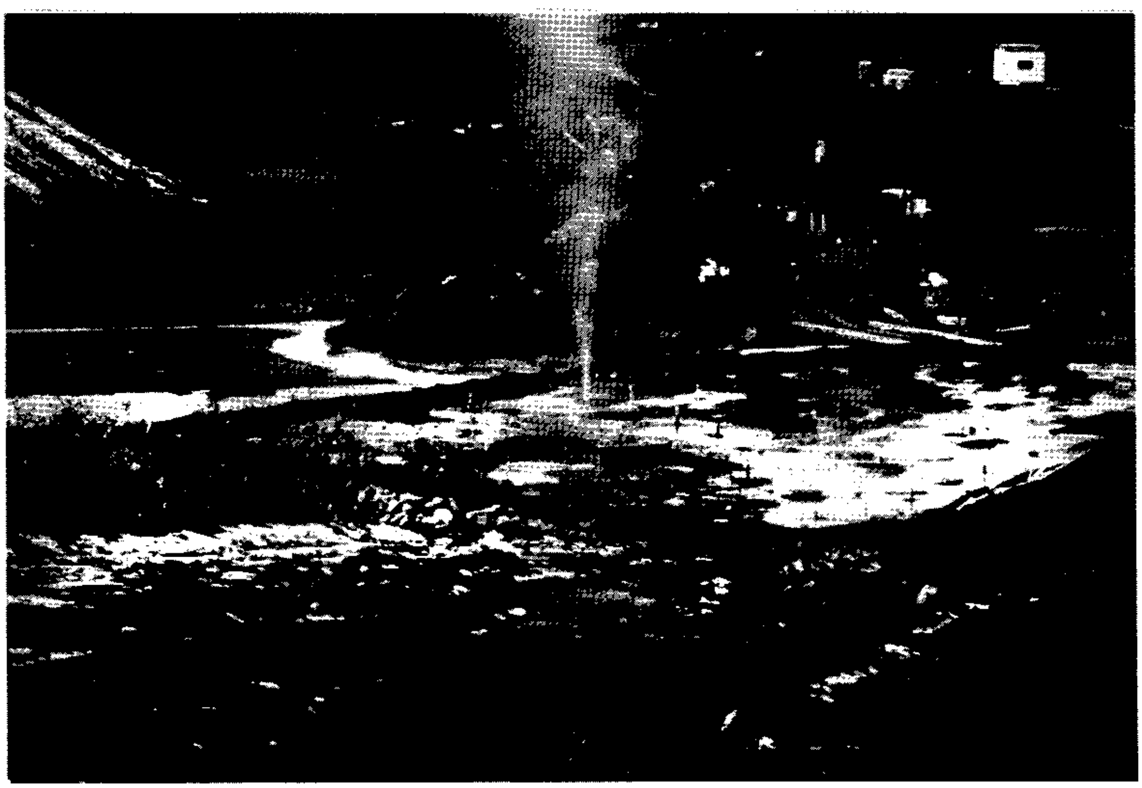


Photo 6 — End of primary blast sequence detonation in Test Section. Groundwater flow to surface via hydrofractures is not yet occurring.

Photo 7 — Settlement cracks atop hummock (staging area) on the west side of the west abutment.

Photo 8 — Pooled water from groundwater flow to surface via hydrofractures on West Abutment following primary production blast, December 13, 1992.

Photo 9 — Cratering around blast hole, second pass on the remainder of the East Abutment.

APPENDIX A

Excerpt: Contract Plans and Provisions

XE 3073

Contract Provisions And Plans

For Construction of:

SR 504, MP 45.91 TO MP 46.04

SO. COLDWATER CR. BRIDGE 504/12 FOUNDATION DENSIFICATION

DISTRICT 4

COWLITZ COUNTY

DISTRICT PROJECT

Washington State
Department of Transportation

BLASTING DENSIFICATION

Description

This work shall consist of performing a ground modification program using deep blasting within drill holes to densify a thick deposit of loose silty sand, gravel, cobbles, and boulders. The depth of the deposit ranges from 120 to 140 feet. This shall be done by installing a series of blast holes, vertical drains and instrumentation as shown in the Plans and described in these Special Provisions.

The Contractor shall provide all labor, tools, equipment, transportation, supplies and personnel to complete all contract work. The main work items include obtaining Becker hammer resistance data (both prior to blasting and post-blasting): obtain SPT data (immediately after blasting); installation of vertical drains; drilling of blast drill holes; placement of casing and blast decks in blast drill holes; complete blasting at designated locations and sequencing; installation of instrumentation as described elsewhere in these Plans and Special Provisions; and any other incidental work. The State will supply the survey work and the reading of the instruments except obtaining the analyzer information during the Becker Hammer Testing.

Submittals

All bidders must demonstrate experience and capability to successfully carry out the bid package. Bidders must provide a list of all drilling and support equipment; key personnel experience list including lead drillers; blaster, blasting consultant, site supervisor, method statement of installation, drilling through boulders; blasting plan including types of explosives, caps, delays, timing devices and safety program.

The Contractor shall retain the services of a blasting consultant to review the blasting plan. The consultant shall be an expert in the field of drilling and blasting who derives his primary source of income from providing specialized blasting and/or blasting consulting services. The consultant shall not be an employee of the Contractor, explosive manufacturer, or explosive distributor.

Five days prior to the pre-construction meeting, the Contractor shall provide specific details of his proposed procedures, equipment, personnel (with resumes), and schedule. The Contractor shall also discuss the details of the "test section" phase of the work. Prior to start of construction, the Contractor must receive in writing authorization to proceed from the Engineer.

Qualifications

All prospective bidders shall meet the minimum requirements to be prequalified specifically in Class 14, Drilling and Blasting-Blast Densification, regardless of the current status of their prequalification with the Washington State Department of Transportation. In order to receive consideration for the issuance of a proposal form, a person, firm or corporation shall submit data to supplement that which is required in the Standard Questionnaire and Financial Statement form. Such data is necessary to ascertain that the prospective bidders have the necessary experience, organization and technical skills to perform a controlled blast densification project. The following specific information must accompany the prospective bidder's Standard Questionnaire and Financial Statement.

1. List all controlled blasting and related projects which have been completed by your organization. A minimum of 3 years blasting experience is required. Include the following information for each project:

   A. Title, year completed, and description of work including:

      (1) Method of drilling through boulders.
      (2) Depth of explosive charges and method of installation of charges
      (3) Experience in performing field

      installation of monitoring equipment

   B. Bidders responsibility on the project (such as prime contractor or subcontractor).

   C. Percent of work performed by the bidder's organization.

   D. Original and final contract value of work.

   E. Name, address, and telephone number of project owner and owner's representative.

2. List all key field and home office personnel and the position to which they may be assigned on a drilling and blasting project. Personnel should include key people such as Project Manager, Instrumentation Supervisor, Safety Officer, lead drillers, blaster, Site Superintendent for Field Management, and Principal-In-Charge for home office Management. The Bidder shall submit a Table of Organization with resumes which include the following information concerning each listed individual:

   A. Present position or capacity and length of employment.

   B. Years of construction experience and prior employer(s).

   C. Years of drilling and blasting experience.

   D. Type of position and capacity held for blast densification and related drill and blasting projects.

E. Education and Professional Registration.

3. In addition, the Contractor must provide evidence to show that the following full-time personnel are on the staff and available to support such a project:

   A. Lead ~~drilling~~ driller with at least two years experience to include boring vertical holes to depths greater than 100 ft.

   B. Instrumentation Supervisor with at least three years experience with geotechnical instrumentation including deep settlement devices and piezometers.

   C. Licensed Blast Supervisor with at least five years experience in controlled blasting including loading and shooting vertical borings greater than 100 feet.

In the event a prospective bidder is currently prequalified, the State may accept submission of only the aforementioned supplemental experience information, thereby avoiding the submission of a new Standard Questionnaire and Financial Statement form.

Materials
Blast holes must be suitably cased to allow installation of blast charges. Casing shall consist of 3.5 inch O.D. schedule 40 PVC, or an approved equivalent.

Vertical drains will be cased with 3.0 inch I.D. slotted schedule 40 PVC~~, or~~ approved equivalent. The casing will be slotted over its full length. A #100 slot size with a minimum 6 rows per foot is to be used.

Charges are based on equivalent weight of TNT.

General Construction Requirements
The site is located in an area which has geological significance. The intent of this contract is to densify the existing ground and minimize disturbance to the area. The area of allowable disturbance is shown in the Plans. This area includes, densification zone, nominal zone outside the densification limits, and designated areas for access and staging. Access and staging outside the designated areas are not allowed without written permission from the Engineer.

Order of Work

Test Phase

1. Drill primary blast holes (9) and vertical drains(8).
2. Drill instrumentation holes (4).
3. Install settlement plates.
4. Load and detonate primary blast holes.
5. Drill, load and detonate secondary blast holes(4).
6. Evaluation period 1 week maximum, Engineer will assess need for third round of blasting, if needed the Contractor shall repeat step 5. If a third round of blasting is required the Contract time will be adjusted accordingly. The Engineer will finalize the blast design for the production phase.

Production Phase

1. Drill primary blast holes (20) and vertical drains (32), 4 of the BPT test holes shall be used as blast holes.
2. Drill instrumentation holes (14).
   a. 8 sondex inclinometer
   b. 6 piezometric
3. Install settlement plates (25).
4. Install geophones (5)
5. Load and detonate primary blast holes.
6. Drill, load and detonate secondary blast holes(12).
7. Drill 4 BPT and 4 SPT test holes.
8. 4 month aging period.
9. Remobilize and drill 4 BPT test holes.

"Test Section"

Initial Drill Holes: The Contractor shall drill a total of 25 drill holes at the locations shown in the test area shown in the Plans or as directed by the Engineer. The holes will be drilled to depths ranging from 100 to 150 feet as directed by the Engineer. The holes shall be completed as either vertical drains, blast holes, or instrumentation holes, as specified in the Plans or as designated by the Engineer.

"Test Section" Phase: The "test section" phase will consist of implementing the program specified in the Plans, or as designated by the Engineer. The initial blasting sequence will involve a coarse grid as indicated in the Plans. The vertical spacing of the decks shall be at depths of about 15, 35, 55, 75, 95, and 120 feet. The energy (equivalent pounds of TNT) in each deck shall be 5, 10, 15, 20, 24, and 30 pounds at the 15 foot through 120 foot depth decks respectively. After the initial blast, settlement and pore pressure will be measured.

At the direction of the Engineer the charges for the second round of blasts may be modified. The Engineer will assess the results of two rounds of blasting. If the results are satisfactory to the Engineer, the Engineer will finalize production blast design and direct work to proceed. Otherwise, assess potential design modifications and the need for a third round of charges.

The information in the above paragraphs pertaining to timing, layout and quantity of explosives used, shall be incorporated into the Blasting Plan to be reviewed by the blasting consultant and approved by the Engineer.

Sequencing details are subject to change and the Contractor should be prepared for delays between 0 and 1 second both between holes and within holes. The data from the test section will be reviewed by the Engineer to determine the blasting details for the remaining areas. As the program continues, additional modifications may be required.

Blast Timing: It is anticipated a single pass of blasting (either test phase or production phase, first pass or second pass) may include detonation of up to 9 blast holes. A row of two or three holes will be detonated simultaneously, with additional rows being detonated at a 0.75 millisecond delay. Within a single hole, there will be delays of approximately 0.5 seconds between detonation of each of the decks, with the bottom deck to be detonated first, and the top deck to be detonated last

"Production Section"

The Engineer shall develop a specific program for the production Phase of this work. Changes to the initial plan could include changes to spacing of decks, unit weight of blasting agent per deck and possibly adding a third round of charges. The Contractor shall not initiate this work without written approval from the Engineer. The Engineer will specify the program in writing within one week of completion of the "test section". The State reserves the right to modify the program as the production phase progresses.

Any changes to the initial Blasting Plan shall be reviewed by the blasting consultant and approved by the Engineer.

Becker Penetration Testing:

Before ground modification and immediately after completion of the ground modification, the Contractor shall drill a total of 8 Becker Hammer Penetration Tests at the locations designated by the Engineer. Four tests before modification and four tests after the modification. The first four test holes will be used to place blast casing.

The post ground modification work will be done no less than 2 days after the "production section" phase of work is completed and no more than 5 days after this work has been completed.

Dynamic testing will be performed during Becker Hammer Testing. This will be performed at the eight BPT test holes. The dynamic testing equipment (Pile Driving Analyzer) and personnel needed to operate the equipment shall be provided by the Contractor. The Contractor may obtain a Pile Analyzer from one of the following firms:

Bert Miner
GRL and Associates, Inc.
P. O. Box 340
Manchester, WA 98353-0340
Phone: (206) 624-0220

Art O'Brien
CH2M Hill, Inc.
1500 114th S.E.
Bellevue, WA 98004
Phone: (206) 453-5000

MiKe Holloway
InSitu Tech, Inc.
262 Grand Ave. Suite 200

Oakland, CA 94610
Phone: (415) 839-6567

The Contractor shall provide the following information, as obtained from the Pile Annalyzer, at 5 foot intervals for full depth of test hole or at the direction of the Engineer:

Blows per foot
Transferred Energy
Driving System Transferred Efficiency

This information shall be made available to the Engineer during the BPT test.

Standard Penetration Testing:
After completion of the ground modification, the Contractor shall drill 4 test holes using Standard Penetration Testing (SPT). The SPT holes will be drilled to the depth and location specified by the Engineer. The holes will be drilled not less than 2 days after completion of the ground modification and not more than 5 days after the ground modification.

* Then 4 months after completion, the Contractor shall drill 4 Beck Hammer Penetration Test holes at the locations directed by the Engineer. This will require the Contractor to mobilize a drill rig to the site and complete this work. The State must give the Contractor 30 days notice prior to this work.

Items of Work

Drill Holes without penetration testing: These are drill holes to be advanced using a suitable drill rig for purpose of installing instrumentation, blast casing, and vertical drains.

Drill Holes with Becker Penetration Testing (BPT): These are drill holes to be advanced using a 6-5/8" casing I.D. Becker Hammer drill rig in conjunction with a Becker Penetration Testing. BPT will be performed with a pile analyzer in conjunction with an oversize 8-5/8" casing shoe and drilling mud to reduce shaft friction during testing. These holes may be used for installation of blast casing.

Drill Holes with Standard Penetration Testing (SPT): These are drill holes to be advanced using a suitable drill rig equipped with an automatic hammer that allows performance of a Standard Penetration Test. SPT's will be conducted at 5 foot intervals.

Blast Holes: Blast holes are drill holes to be advanced by suitable equipment. Where required, the drilling of blast holes may be accompanied by penetration testing (e.g. BPT). Blast holes must be suitably cased to allow installation of blast charges.

Vertical Drains: Vertical drain holes are to advanced by suitable equipment to allow installation of 3.0 inch I.D. slotted PVC pipe.

Decks: Each blast hole will have several charges, each installed at specific elevations/depths within the casing. Each individual charge is referred to as a deck.

Evaluation Period: The Contractor shall expect as much as one week evaluation period after the test section detonation. No work shall be done in this period. Work shall not resume until the Contractor recieves notification from the Engineer. This evaluation period time will not be charged against the contract time.

Measurement

Drill holes with BPT, drill holes without BPT/SPT, drill holes with SPT, and drill holes for instrumentation will be measured by the lineal foot installed. These holes are associated with the "test section" and "production section" of blast densification.

Decks will be measured per pound of TNT used per charge including all costs associated with installing and detonating the charge, not including costs associated with drilling and installing the blast hole casing (those items being bid separately). The weights refer to equivalent weights of TNT.

Blast casing and vertical drains will be measured per lineal foot installed.

Payment

The unit contract price per lineal foot for "Drill Holes With BPT", "Drill Holes Without BPT/SPT", "Drill Holes With SPT", "Drill Holes For Instrumentation", "Blast Casing", and "Vertical Drains" shall be full pay to perform the work as specified.

The unit contract price per pound for " TNT" shall be full pay to perform the work as specified.

TEST HOLE DATA

For test hole information and location see results of the geotechnical investigation Bridge No. 12 SR 504 Coldwater Lake to Johnson Ridge, figures 2-6 Plan and Profile Bridge 12 by Golder and Associates. This information is available at the project engineer's office.

Doug Ficco, P.E.
2400 Talley Way
Kelso, WA 98626
Telephone (206) 577-2230

GROUND DENSIFICATION INSTRUMENTATION

Description

This work shall consist of furnishing all materials and labor, and performing all tests necessary to install instruments in accordance with the Plans and these Special Provisions. Two phases of instrumentation will be performed. Phase 1 will include the installation and testing of instruments located within the "test section" zone. Phase 2 instruments will be installed and tested within the "production section" zone of blast densification.

The Contractor shall install the instruments under the supervision of a qualified geotechnical instrumentation specialist having a minimum three years of experience installing similar instrumentation.

Phase 1 instruments will be installed and tested prior to blasting densification of the Test Section. Phase 2 instrumentation shall not be installed until after completion of the test section and as directed by the Engineer. The Engineer will survey the instrument locations take all readings and interpret the data. Readout devices (except for Sondex and Inclinometer) will be supplied by the Contractor on a rental basis. The Contractor will accommodate the Engineer during the reading of the instrumentation, providing access and time to take readings.

Submittals

No later than the preconstruction conference, the Contractor shall submit in writing, a list of the instruments including instrument specifications, installation procedures, and readout devices. Also, at this time, the Contractor shall submit resumes of those individuals responsible for instrument installation and testing. The list shall include references, including current telephone number, that can verify the experience requirements. Work shall not begin until the Engineer has approved instruments, installation procedures, and personnel.

Instruments

Inclinometer Casing

Inclinometers shall consist of 2.75 inch O.D., internally grooved plastic casing, in 10-foot lengths, provided with all necessary end plugs, caps and couplings. The spiral twist of casing grooves in one 10-foot section of casing shall not exceed one degree. If requested by the Engineer, the manufacturer shall supply reports verifying the twist. The top of each casing shall be provided with plastic cap. The casing shall be manufactured by either Carlson/RST Instruments Inc. of Yakima, Washington, Slope Indicator Company (SISINCO) of Seattle Washington, or an approved equal.

Piezometric Transducer

The piezometric transducer shall be of a strain gage type made of corrosion resistant material such as stainless steel or an approved equivalent, with a sintered metal filter. Transducer pressure range shall be to 120 psi with a minimum 200 percent overrange capability and capable of a frequency response of 100 Hz minimum. The transducer shall be capable of withstanding a short term pressure pulse equal to 500% of the rated pressure without damage or change to the calibration. The transducer will be vented to the atmosphere and will be have a minimum of 300 feet of signal cable. The cable must be capable of withstanding direct burial and settlement from densification. The transducer shall be calibrated by the manufacturer and supported with calibration test data and certificates.

Data Logger

The data logger readout device for the piezometric transducer shall have a minimum 24 channel capacity. Scan rate shall be programmable, with a minimum rate of reading one channel per second. Channels shall be individually programmable with respect to input type, range, on/off. Data shall be stored in a format suitable for downloading to an IBM PC compatible computer. Real-time display of selected channels shall be available. The data logger shall be calibrated by the supplier and must be supplied with a power source and connectors to allow continuous operation. The system accuracy of the Transducer and Data Logger shall be at least +/- 0.25 P.S.I., the system resolution shal be at least +/- 0.06 P.S.I..

Borros Anchors

Borros anchors shall consist of a three-pronged anchor attached to a 0.25 inch diameter inner steel pipe placed within a 1 inch diameter outer steel pipe. The inner pipe shall be free to move inside the larger diameter outer pipe.

Settlement Plates

Settlement plates shall consist of 1-foot width square plates attached to a 5-foot long steel post, as shown in the Plans.

Deep Settlement Device

Deep settlement devices shall consist of a probe extensometer system installed in vertical holes. The system shall be a SONDEX settlement system manufactured by SINCO or an approved equivalent. Settlement casing shall be 3 inch nominal diameter corrugated plastic pipe without perforations. Casing shall be provided in a single continuous length and the bottom of the casing shall be fitted with a cap. The casing shall be capable of accommodating a minimum of 12 feet of settlement over a length of 100 feet. Measurement rings shall be installed on the casing at 3-foot intervals along its entire length. Rings shall be compatible with a SONDEX extensometer probe model 50819. The internal support casing shall consist of either 2.75" O.D. inclinometer casing or 2.5-inch nominal diameter rigid schedule 40 PVC, flush coupled or as recommended by the manufacturer.

Ground Surface Vibration Measurements

The instrumentation shall consist of three-component geophones and associated equipment. The recording hardware and software shall be capable of direct measurements of ground velocities of up to 5 in/sec and with the capability to produce a hardcopy waveform velocity print-out. The equipment shall be capable of having ground velocity trigger levels set to automatically start the recording, with trigger levels as low as 0.02 in/sec. Minimum recording times of 5 seconds (from triggering of the record) will be required and the velocity resolution levels shall be greater than 0.01 in/sec.

Installation

Inclinometers

A total of five inclinometers shall be installed within the "production section" zone (Phase 2) of densification at locations specified by the Engineer. The casings shall be installed to a depth of 140 feet and one set of grooves in the casing shall be aligned in the north compass direction. Corrugated "settlement casing" will be placed over inclinometer casing. The installation of settlement casing is described in the following section "Deep Settlement Devices". The zone between the drill hole and casing shall be backfilled with sand or an approved equivalent.

Piezometric Transducers

A total of 12 transducers shall be installed within the zone identified as the "production section" (Phase 2). The transducers shall be placed in either a Ottawa sand filled slotted PVC pipe or a Ottawa sand filled bag which is constructed from a 7 oz (minimum). non-woven, needle-punch geotextile. No more than two transducers shall be installed in a single hole. The holes shall be located by the Engineer. The transducers will be placed at the following elevations or at the designation of the Engineer:

Three Holes with transducers @ 115' & 85'
Three Holes with transducers @ 45' & 25'

The transducers shall be placed in the middle of a 9 foot long completion zone. The completion zone consist of a 3 foot column of Ottawa sand sandwiched between 3 feet of Bentonite. Bentonite for backfilling and sealing piezometer installation shall be in the form of chips or pellets. Backfill between completion zones may consist of drill cuttings or other approved equivalent. Slack must be allowed in the cable to accommodate differential movements during densification. The transducers will be checked after initial placement and before backfill with the bentonite seal and immediately after the installation has been completed. Transducers that fail during this testing period will be replaced at the Contractor's expense.

Data Logger

The Contractor shall provide protection to the data logger as recommended by the supplier. The data logger shall be located a safe distance from the zone of blast densification. This distance is defined as an area where an operator may work safely. In case of adverse weather conditions an enclosure with temperature control may be required.

Borros Anchors

Two Borros anchors will be installed (one per hole) in the "test section" zone (Phase 1) of blast densification". The anchors shall be located at the following depths below the ground surface: 60 and 90 feet. The anchors shall be installed in accordance with the manufacture's specifications. The State will be responsible for survey of the anchors. The Contractor shall allow time for survey between primary and secondary blasting.

Settlement Plates

Fifteen settlement plates will be placed in the "test section" zone (Phase 1) and 25 plates will be placed in the "production section" (Phase 2) of blast densification. The plate will be buried 2 feet below the ground surface, with the steel post to extend above the ground surface to serve as a survey stake. The settlement plates located outside of the Densification limits, as shown in the Plans, shall be hand placed. The State will be responsible for survey of the settlement plates. The Contractor shall allow time for survey between primary and secondary blasting.

Deep Settlement Devices

A total of 10 deep settlement devices will be installed. Two will be located in the "test section" (Phase 1) and the remaining eight will be placed within the "production section" (Phase 2). The bottom of the casing shall be placed to a depth of 140 feet. The corrugated casing will be capped to prevent ingress of water during densification. Install the rigid PVC casing in the corrugated casing so that the outer casing can move freely in the vertical direction with respect to the inner pipe at all elevations. The zone between the drill hole and corrugated casing shall be backfilled with sand or an approved equivalent. The corrugated casing and support casing will be filled with water to groundwater table prior to densification.

Ground Surface Vibration Monitoring

The Contractor will install 5 geophones at locations designated by the Engineer. The instruments shall be installed according to the recommendations provided by the manufacturer. The installation of the instruments will be done within the "production section" (Phase 2). The geophones will be placed along a single axis at distances of 0 (center of blast), edge of blast zone, 25' 50' and 100' outside the blast zone.

Measurement

All materials including installation.

Payment

The lump sum contract price for instruments in the "Instrumentation Test Section" (Phase 1) and lump sum contract price for instruments in the "Instrumentation Production Section" (Phase 2) shall be full pay to perform the work as specified.

CONSTRUCTION GEOTEXTILE

October 23, 1989

Description

The Contractor shall furnish and place construction geotextile in accordance with the details shown in the Plans.

Materials

Geotextile and Thread for Sewing

The material shall be a woven or non-woven geotextile consisting only of long chain polymeric filaments or yarns formed into a stable network such that the filaments or yarns retain their position relative to each other during handling, placement, and design service life. At least 95 percent by weight of the long chain polymers shall be polyolephins, polyesters, or polyamides. The material shall be free from defects or tears. The geotextile shall conform to the properties as indicated in Tables 1, 2, and 3 for each specified use. The geotextile shall be free of any treatment or coating which might adversely alter its physical properties after installation.

Thread used shall be high strength polypropylene, polyester, or Kevlar thread. Nylon threads will not be allowed. The thread used to sew permanent erosion control geotextiles must also be resistant to ultraviolet radiation.

Geotextile Properties

Table 1: Geotextile for underground drainage.

Geotextile Property	Test Method	Geotextile Property Requirements: Low Survivability	Geotextile Property Requirements: High Survivability
AOS	WSDOT Test Method 922	.21 mm max. (#70 sieve)	.21 mm max. (#70 sieve)
Water Permeability	WSDOT Test Method 924	.08 cm sec min.	.08 cm sec min.
Tensile Strength, min. in machine and x-machine direction	WSDOT Test Method 916	90 lbs min.	180 lbs min.
Seam Breaking Strength	WSDOT Test Method 918 and WSDOT Test Method 916 (Grab Test)	80 lbs min.	160 lbs min.
Burst Strength	WSDOT Test Method 920	140 psi min.	290 psi min.
Puncture Resistance	WSDOT Test Method 921	40 lbs min.	80 lbs min.
Tear Strength, min. in machine and x-machine direction	WSDOT Test Method 919	30 lbs min.	60 lbs min.

Table 2: Geotextile for soil stabilization.

Geotextile Property	Test Method	Geotextile Property Requirements
AOS	WSDOT Test Method 922	.42 mm max. (#40 sieve)

SUMMAR[Y]

PROJECT LIMITS

NO LENGTH.

SR 504

MP 45.91 TO MP 46.04

STA. 2421+00 TO 2428+00

FOR EQUATIONS/EXCEPTIONS SEE VIC. MAP

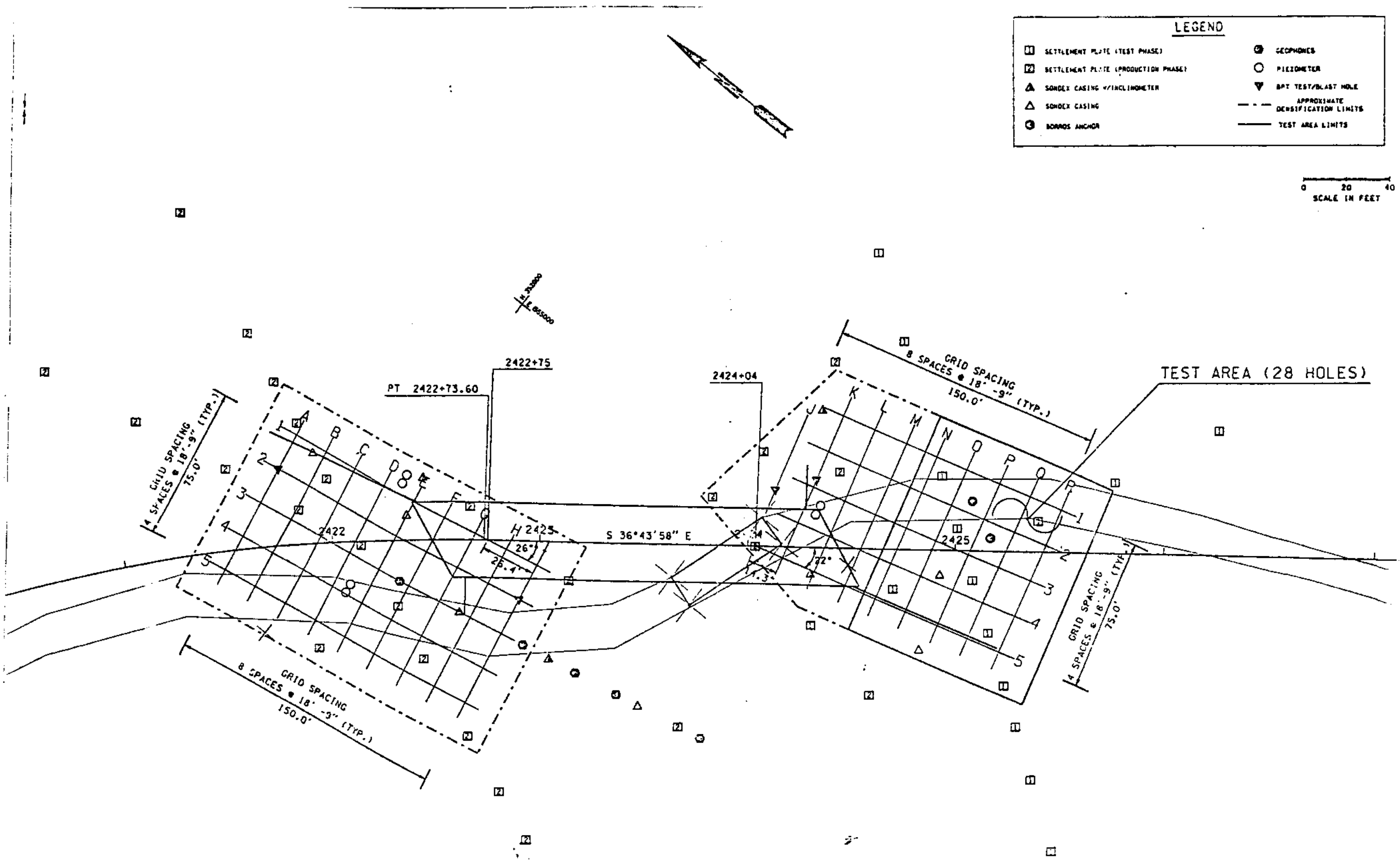
FEDERAL AID	
SR 5	
GROUP 1	

ITEM NO.	TOTAL QUANTITY	STD. ITEM NO.	UNIT	ITEM	2421+00.00 TO 2428+00.00 CS 0800NT	Bid Price
				PREPARATION		
1	LUMP SUM	0001	L.S.	MOBILIZATION	L. S.	59,370
				GRADING		
2	6,100	0310	C.Y.	ROADWAY EXCAVATION INCL. HAUL	6,100	4.80
				DRAINAGE		
3	770	1074	C.Y.	LIGHT LOOSE RIPRAP	770	47.00
				TRAFFIC		
4	74	6982	S.F.	CONSTRUCTION SIGNS CLASS A	74	18.00
				OTHER ITEMS		
5	EST. 15,000	7480	DOLLAR	ROADSIDE CLEANUP	EST. 15,000	15,000
6	720	7556	L.F.	CONSTRUCTION GEOTEXTILE FOR SILT FENCE	720	4.20
7	LUMP SUM		L.S.	INSTRUMENTATION TEST SECTION (PHASE 1)	L. S.	3,740
8	LUMP SUM		L.S.	INSTRUMENTATION PRODUCTION SECTION (PHASE 2)	L. S.	37,660
9	1,680		L.F.	DRILL HOLES WITH BPT	1,680	40.00
10	11,340		L.F.	DRILL HOLES WITHOUT BPT/SPT	11,340	16.00
11	560		L.F.	DRILL HOLES WITH SPT	560	61.00
12	2,230		L.F.	DRILL HOLES FOR INSTRUMENTATION	2,230	25.00
13	6,300		L.F.	BLAST CASING	6,300	2.50
14	5,600		L.F.	VERTICAL DRAINS	5,600	4.00
15	4,725		LB.	TNT	4,725	6.80
16	40		EACH	SETTLEMENT PLATES	40	100.

NOTE: FOR SPECIAL FEATURES SEE SPECIAL PROVISIONS

DESIGNED BY	REGION NO.	STATE	FED. AID PROJ. NO.
ENTERED BY	10	WASH	
CHECKED BY	JOB NUMBER		FEDERAL AID
PROJ. ENGR.			

DIST4



PROGRAM DEVELOPMENT DIVISION

Washington State Department of Transportation

SO. COLDWATER CK. BRIDGE 504/12
FOUNDATION DENSIFICATION

M. MCBERLY
P. HARRISON

STATE FED.AID PROJ.NO.

WASH.

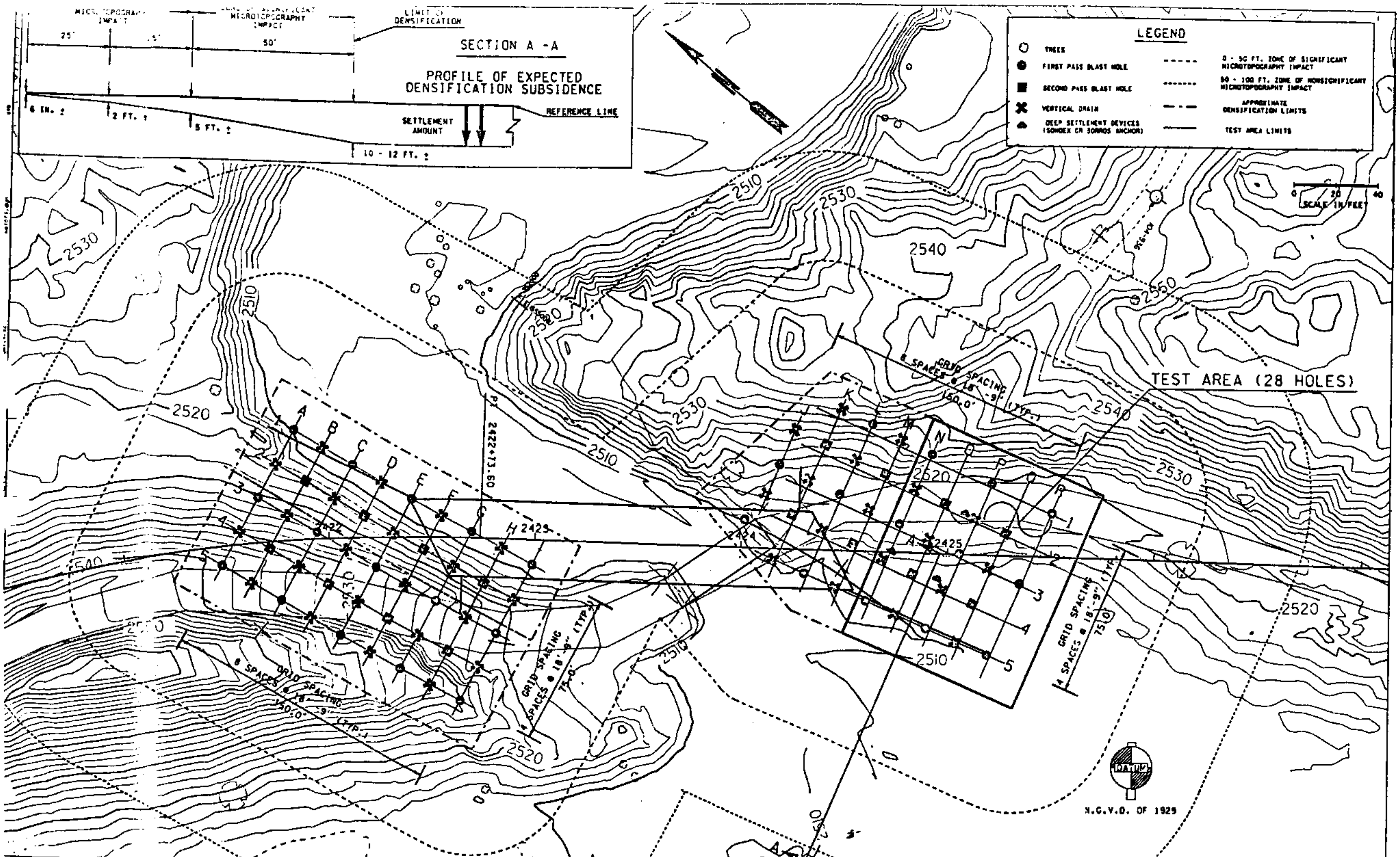


FIGURE 1 OF FIGURES 1, 2A & 2B OF GOLDER REPORT

TEAM LEADER M. MODERLY DESIGNED BY P. HARRISON ENTERED BY PJK CHECKED BY P. HARRISON PROJ. ENGR. J. ALLINGER	REGION NO. 10 STATE WASH FED. AID PROJ. NO. ER-0805(142)	PROGRAM DEVELOPMENT DIVISION	Washington State Department of Transportation	SO. COLDWATER CK. BRIDGE 504/12 FOUNDATION DENSIFICATION

APPENDIX B

Excerpt: "A Testing Technique for Earthquake Liquefaction Prediction in Gravelly Soils" by Klohn Leonoff

NATIONAL RESEARCH COUNCIL OF CANADA

INDUSTRIAL RESEARCH ASSISTANCE PROGRAM

A TESTING TECHNIQUE FOR EARTHQUAKE LIQUEFACTION PREDICTION IN GRAVELLY SOILS

Improvements to the Becker Penetration Test for Estimation of SPT Resistance
IRAP-M 40401W

PB 5532 01

SEPTEMBER 1992

2. BACKGROUND OF THE BECKER METHOD

The Becker drill has been used in subsoil and aggregate exploration work since its development by Becker Drills Ltd. in Canada, in the late 1950's. The essential features of the drilling system are illustrated on Figure 2.1 which shows how a string of double walled pipe is driven into the ground using a small diesel pile driving hammer, while compressed air, introduced into the annulus between the two pipes, flushes soil cuttings up to the ground surface through the centre hole.

Two sizes of Becker drill casing are in common use:

- 5½ inch OD x 3¼ inch ID; and
- 6⅝ inch OD x 5 inch ID.

The drill is especially useful for obtaining disturbed samples of coarse grained gravelly soils that cannot be easily sampled using conventional drive-tube sampling techniques. The resistance to penetration of the drill pipe, recorded as blows/foot, has been used as a qualitative indicator of soil consistency for many years.

Beginning in the early 1970's, the Becker Penetration Test (BPT), initially named the "Becker Denseness Test", came into geotechnical engineering practice, primarily due to development by Becker Drills Ltd. in Vancouver, British Columbia. Used in this mode the conventional drill bit was replaced by a closed-end shoe or plugged bit. No compressed air supply is needed since no sample can be recovered, and the pipe is simply driven into the ground, much like a small pile driving test. The penetration resistance, blows/foot, of the plugged bit soundings was found to be much less affected by groundwater in sandy soils when compared to open bit soundings, and more confidence was gained in using it to estimate equivalent Standard Penetration Test (SPT) values and other parameters such as pile drivability and end bearing refusal elevations. Several correlations were developed between the Becker blows/foot and the Standard Penetration Resistance (SPT) (see Figure 2.2). The SPT test is used worldwide as an indicator of granular soil density, and many engineering soil properties are correlated with the SPT blows/ft, or N value.

The SPT test in sandy soils is used as an indicator of the resistance to earthquake shaking. For many years, research was undertaken at the University of California at Berkeley, under the direction of Professor H.B. Seed, which resulted in a set of design charts whereby the resistance to the triggering of subsoil liquefaction in loose sandy soils by earthquake forces could be estimated knowing the value of SPT $(N_1)_{60}$ blows/ft. An example of these charts is reproduced on Figure 2.3. The values of $(N_1)_{60}$ are derived from the field measured N values by correcting to a standard reference level of effective overburden pressure, 1 tsf, and correcting for the energy efficiency of the SPT hammer/ anvil system to a standard reference energy of 60% of the theoretical maximum value. For many years, the research on liquefaction concentrated on sandy soils for which the SPT test is well suited and for which there is widespread field evidence. However, liquefaction was also noted in gravelly soils in the Alaska earthquake of 1964 and the Tangshan earthquake of 1976. The 1983 Borah Peak earthquake in Idaho, however, gave the first well documented examples of level ground liquefaction and liquefaction induced lateral spreading and deformation in sloping ground in gravelly soils. A problem arises with trying to use the Seed design charts in gravelly soils. The standard split spoon used for the SPT has an outside diameter of 2 inches and inside diameter of only 1 3/8 inches. Thus, gravel particles of, say, ¼ inch or larger can easily jam in the mouth of the sample tube and result in little or no sample recovery and an excessively high blowcount/ft. This combination of a need to be able to evaluate the liquefaction potential of gravelly soils, and the unreliability of the conventional SPT technique, led to the first widely published effort to correlate the SPT $(N_1)_{60}$ with the Becker Penetration Test, published by Harder and Seed (1986) and Harder (1988) at the University of California, Berkeley.

The work by Harder and Seed included testing at three sites having sand and silt subsoils using the Becker drill and also conventional rotary drilling with SPT tests. The main purpose of this was to develop a correlation between Becker penetration test and

SPT in sandy soil where the SPT is known to be reliable and free of the interference problem caused by gravelly soils as described above. Additional testing was performed at two sand and gravel sites. The testing at the three sandy sites was done using the 6⅝-inch-diameter Becker drill casing. The two gravelly sites were tested using a mix of 6⅝-inch-diameter and 5½-inch-diameter drill casing. The other equipment variable is the drill rig itself. Two types of Becker drills are currently in use, the AP-1000 and the HAV-180. Both drill types use exactly the same International Construction Equipment (ICE) Model 180 diesel pile driving hammer. The principal difference is the way in which the hammer is suspended in the leads on the drill mast.

In developing a correlation between SPT and Becker Penetration resistance, Harder made a correction to the measured Becker blowcounts to account for variable combustion conditions in the operation of the diesel hammer. He used the peak value of the pressure measured in the bounce chamber of the diesel hammer after each blow, as an indicator of the potential energy of the hammer available for the next blow. This was then used to develop a reference line or "constant combustion condition rating curve" to which all field data could be corrected to account for any variations in hammer combustion efficiency from blow to blow. The principle of this correction technique is illustrated on Figure 2.4.

This correction process is not to be confused with the energy correction used to account for the effect of different efficiencies of hammer-anvil-rod systems on the blowcount measured in an SPT test. The purpose of SPT energy correction is to correct the measured blowcount to a standard value of driving energy transmitted into the drill rod string. Sixty percent of the theoretical maximum of 350 ft-lb is the chosen standard for the SPT since most north American manual rope and cathead drill systems deliver about this much energy on average. The Harder and Seed correction procedure described above for the Becker hammer differs from the SPT energy correction concept because on the one hand it only attempts to address the energy of the hammer and not the energy transmitted in the drill pipe, and on the other hand is not a complete energy measurement since it does not attempt to include the explosive energy of the injected fuel.

Having corrected the Becker data for combustion efficiency variations Harder then correlated the Becker penetration resistance blowcount with the corrected SPT blowcount (N_{60}) using a series of side-by-side tests. An example of the correlation developed is shown on Figure 2.5, for the AP-1000 Becker drill using 6⅝-inch-diameter plugged bit casing at the three sandy sites in California, Salinas, Thermalito, and San Diego. A similar correlation developed for a HAV180 drill rig using 5½-inch-diameter, plugged bit casing at a sandy site near Squamish, British Columbia, is shown on Figure 2.6.

4. THE FOUNDEX BECKER PENETRATION TEST

The approach adopted was to substantially eliminate Becker casing friction using the patented[1] Foundex Explorations Ltd. CPTU rod friction reduction method. To achieve this it is necessary to drive a string of Becker casing into the ground with an oversized plugged bit.

Two systems incorporating an oversized bit were tested:

- A dry system - consisting of Becker casing with an oversized plugged bit with a long sleeve considered sufficient to ream an oversized hole and minimize subsequent soil-casing contact along the shaft. The proposed combination is a 5½-inch-diameter casing with a 6⅝-inch-diameter plugged bit and sleeve 15 inches long;
- A wet system - consisting of Becker casing with a similar oversized sleeved bit, but in this case, hydraulically pressured bentonite drilling mud is flushed out from behind the oversized sleeve to both maintain the oversized hole and lubricate the casing. The proposed combinations are 5½-inch-diameter casing with a 6⅝-inch-diameter plugged sleeved bit, and a 6⅝-inch-diameter casing with a 8 5/8-inch-diameter plugged sleeved bit.

Figure 4.1 shows a schematic of these two proposed systems.

A third BPT test system, consisting of pulling back and redriving at the end of each 8-ft long casing was also evaluated.

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[1] U.S. Patent No. 4499954

5. TEST PROGRAM

The field test program was carried out in the right-of-way of B.C. Hydro's transmission line at the site of Tower 5-4, River Road, Delta, British Columbia. The tests were laid out in a grid pattern over a plan area of about 40 ft by 40 ft. The site was selected because of its deeper than average deposit of Fraser River sand. The field tests consisted of two phases; site characterization tests, and a series of Becker penetration tests.

5.1 Site Characterization Tests

Site characterization tests consisted of three piezoelectric cone penetration tests (CPTU) and two mud rotary borings with standard penetration tests. The three CPTUs were first put down about 14 ft apart at the locations shown in Figure 5.1. The terminal depths of the CPTU soundings range from 160 ft to 190 ft. Cone bearing, sleeve friction and pore pressure response behind the cone tip were measured using Foundex Explorations' Geotech cone equipment. The two mud rotary holes were then drilled adjacent to the CPT test holes to depths ranging from 177 ft to 220 ft. Their locations are also shown in Figure 5.1. Standard penetration tests were carried out using an automatic trip hammer, and standard split spoon soil samples were taken within the different soil layers identified by the cone penetration profiles. The SPT trip hammer driving energy transmitted in the SPT sampling rods was measured at test hole SPT91-1, at depths between 9 ft and 120 ft, by the In situ Testing Group of the Civil Engineering Department, at the University of British Columbia (Appendix II).

5.2 Becker Penetration Tests

The Becker penetration tests consisted of four series of tests:

- regular (i.e., conventional flush casing with no special oversized bit) plugged bit Becker penetration tests with both 5½-inch-diameter and 6⅝-inch-diameter casings;
- the proposed Foundex-Becker penetration test with 5½-inch-diameter casing and 6⅝-inch-diameter oversized shoe. In

this series, both dry sleeve and wet mud flush systems were tested;

- the proposed Foundex-Becker penetration test with 6⅝-inch-diameter casing and 8⅝-inch-diameter oversized shoe. In this series, only the wet mud flush system was tested; and

- a regular plugged bit Becker penetration test using 6⅝-inch-diameter casings was tested with a pull-back and redrive of 3 ft at the end of each 8-ft casing length.

A total of 10 Becker penetration test holes were put down, and their locations in relation to CPTU and SPT holes are shown in Figure 5.1. The numbering system used to designate the Becker test holes is also explained on Figure 5.1.

The peak bounce chamber pressure values were measured and recorded for each Becker penetration test using Klohn Leonoff's portable micro-computer based data acquisition system. In addition, for the regular (flush casing, unlubricated) tests and the mudded Foundex-Becker tests the dynamics of the Becker diesel hammer/drill casing system was monitored using a Pile Driving Analyzer supplied by the B.C. Ministry of Transportation and Highways and operated by the In situ Testing Group of the Civil Engineering Department, UBC (Appendix II).

Table 5.1 gives a listing of all the field Becker test holes carried out for this research project.

8. SUMMARY AND CONCLUSIONS

A research program was carried out, partially funded by the National Research Council's Industrial Research Assistance Program, to develop and evaluate the Foundex-Becker penetration test technique. This technique employs an oversize plugged bit with or without mud flush to eliminate the shaft friction in the regular Becker penetration test. The research program consisted of equipment development, field test, and data evaluation phases.

In the equipment development, two configurations of oversize plugged bit and Becker casings were developed for the mud flush system, and one configuration was developed for the dry sleeve system.

In the field test program, two mud rotary drill holes with SPT and three CPTU holes were put down for characterizing the test site and for developing SPT-BPT correlations. Ten Becker penetration tests were carried out with different Becker casing and plugged bit configurations. Regular Becker penetration tests with both 5½ inch and 6⅝ inch casings were also performed to evaluate the Foundex-Becker penetration tests.

This study shows that shaft friction resistance does exist in the regular BPT tests, especially at depths below 70 ft. Such friction resistance makes identification of soft soil layers difficult, and also affects the SPT-BPT correlations.

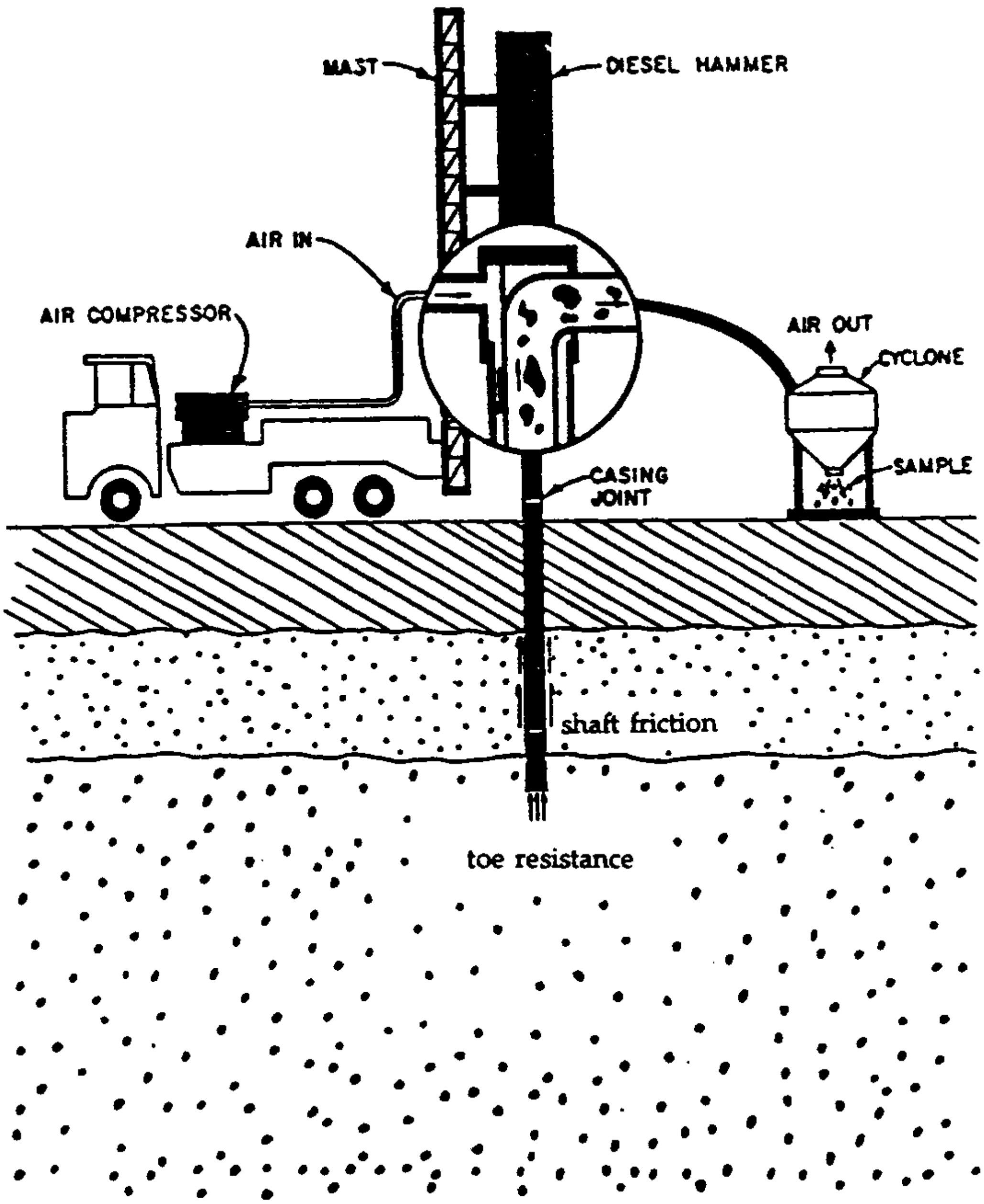
This study shows that the Foundex-Becker mud lubricated penetration test system works. Both PDA measurements and comparison of Becker blowcounts with SPT blowcounts indicate that Foundex mud flush drive system is an effective way to eliminate the shaft friction resistance to Becker casing penetration.

It is found that the constant combustion condition rating curves are different for each BPT test condition even with the same drill rig and at the same test site. Constant combustion condition rating curves for each category of test configuration were developed, and BPT tests of each category were normalized to their own constant combustion condition rating curves. The Becker blow counts so corrected to the bounce chamber pressure variation were then used to develop various SPT-BPT correlations.

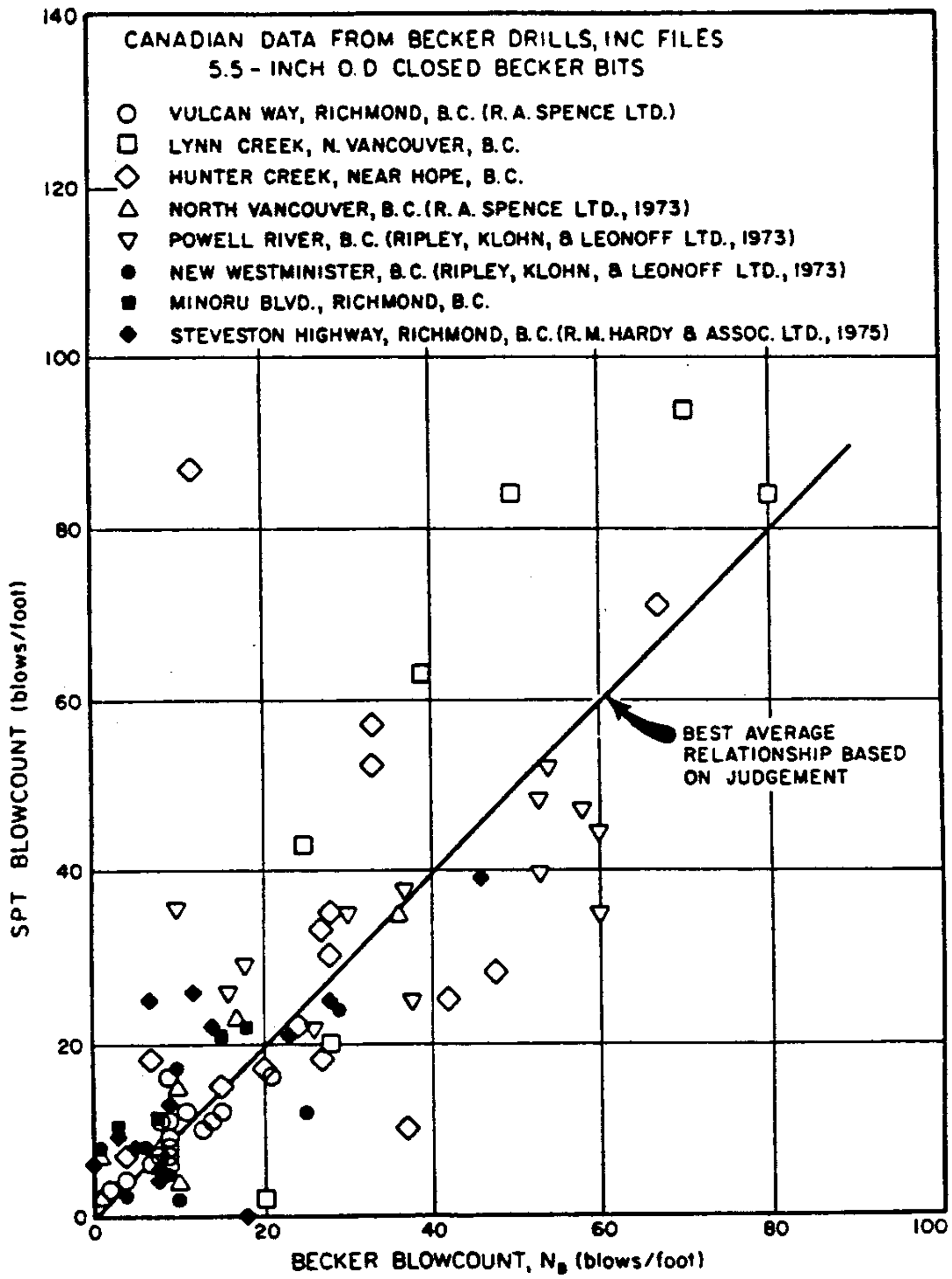
It is seen that due to the shaft friction the correlation between SPT and the regular BPT is of a nonlinear shape. Thus, unless new sites have the same friction characteristics as embedded in the correlation, the application of such correlation is limited, and will result in some uncertainty equivalent SPT blow counts.

On the other hand, the Foundex mud flush system with both plugged bit sizes gives SPT-BPT correlations that are of a linear shape. Thus, the SPT-BPT correlations developed from Foundex-Becker penetration tests have excluded the site specific shaft friction effects.

Wave equation (WEAP) analysis was also carried out to help to understand the fundamentals of both the regular Becker penetration tests and Foundex BPT tests. The analysis suggests that the constant combustion condition rating curve represents a narrow range of ENTHRU values achieved in the tests. We speculate that the different constant combustion rating curves may be explained by the amount of hammer energy, ENTHRU, transmitted into the Becker casing during each test, which depends upon many factors, such as hammer-casing-soil interaction. The normalization process proposed by Harder on the BPT blowcount can be interpreted as normalizing the Becker blowcount of different ENTHRU values to a narrow range of ENTHRU. The WEAP analyses indicate that the constant combustion rating curves are basically similar for both regular and mud flush BPT tests but each would have a different family of blowcount correction curves. The mud flush seems to have flatter correction curves than the regular one. However, the above results need to be verified by field test data which is discussed in the following section.



Becker hammer drill system (from Harder and Seed, 1986)

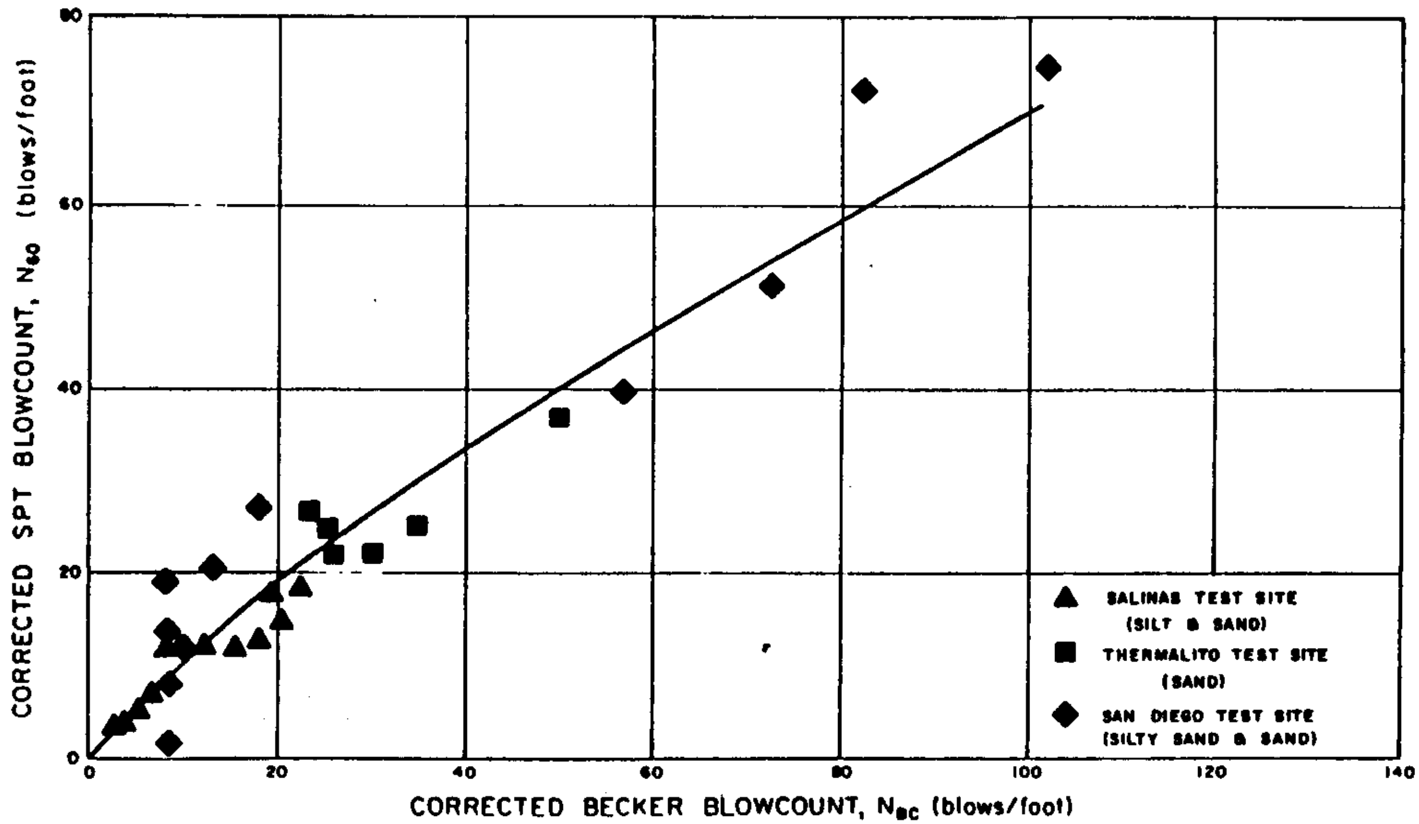


CORRELATION BETWEEN BECKER AND SPT BLOWCOUNTS DEVELOPED FROM CANADIAN DATA OBTAINED FROM BECKER DRILLS, INC. FILES

from Harder (1988)

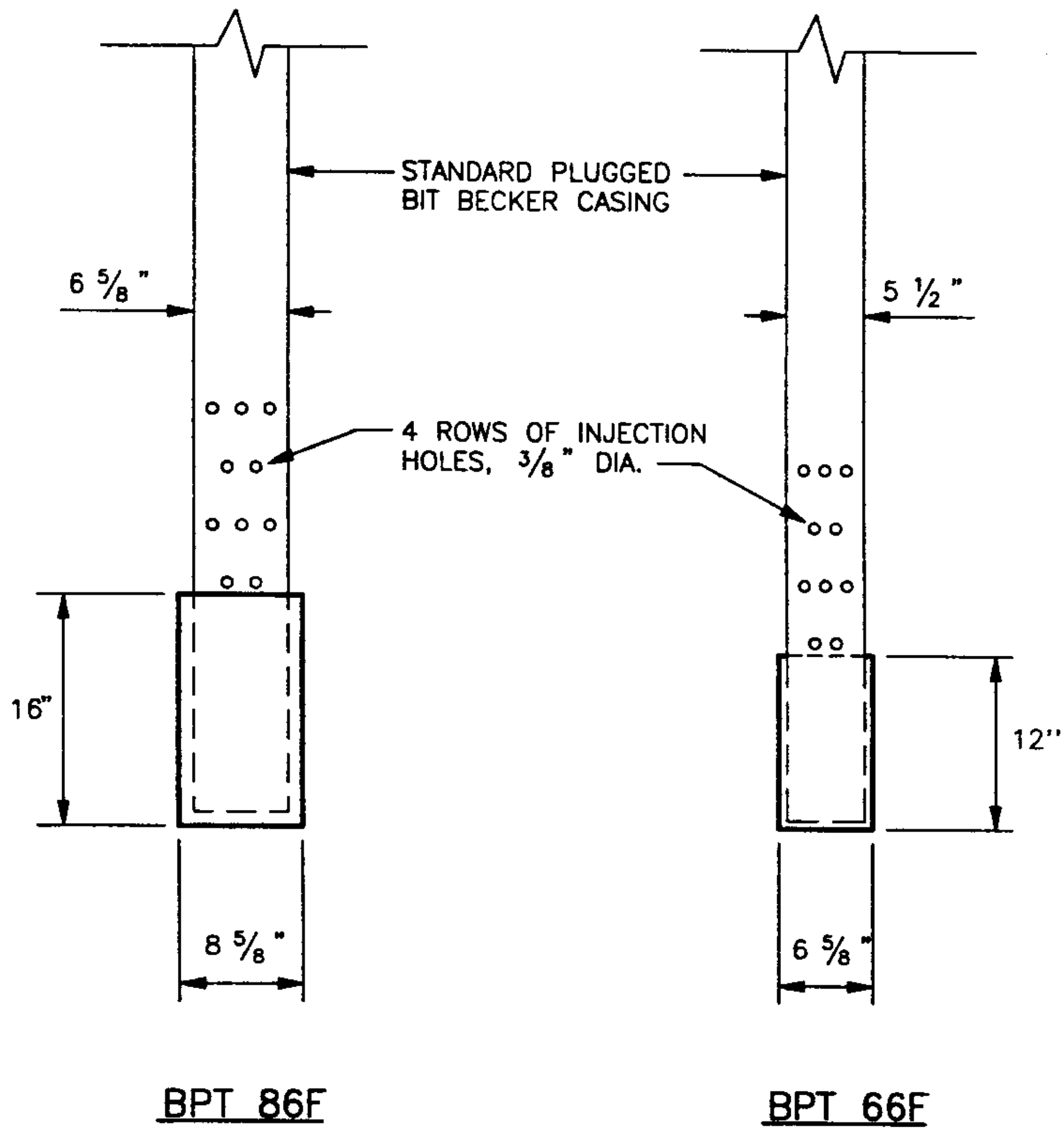
KLOHN LEONOFF FOUNDEX

FIGURE 2.2

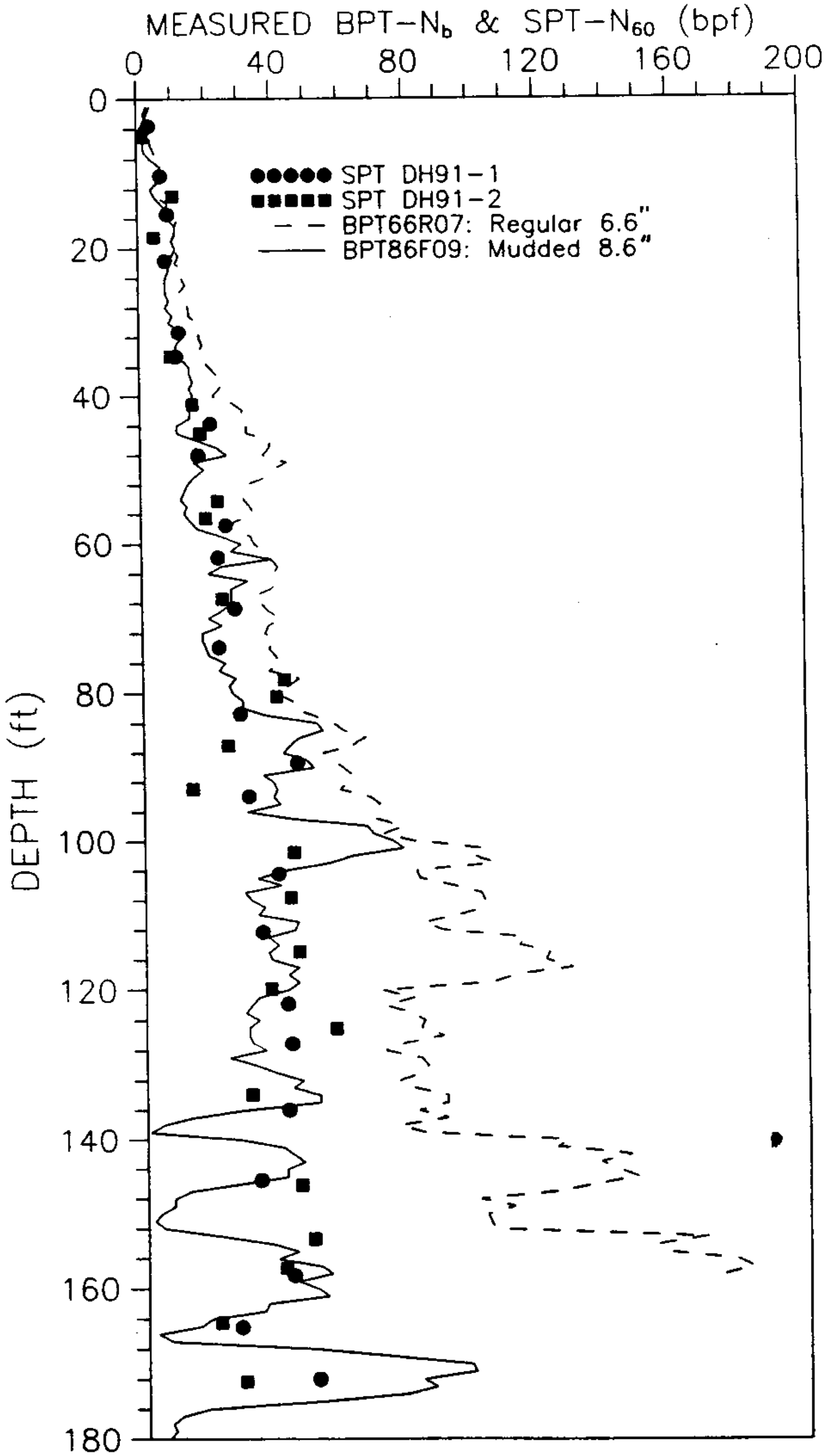


CORRELATION BETWEEN CORRECTED BECKER AND SPT BLOWCOUNTS

from Harder (1988)

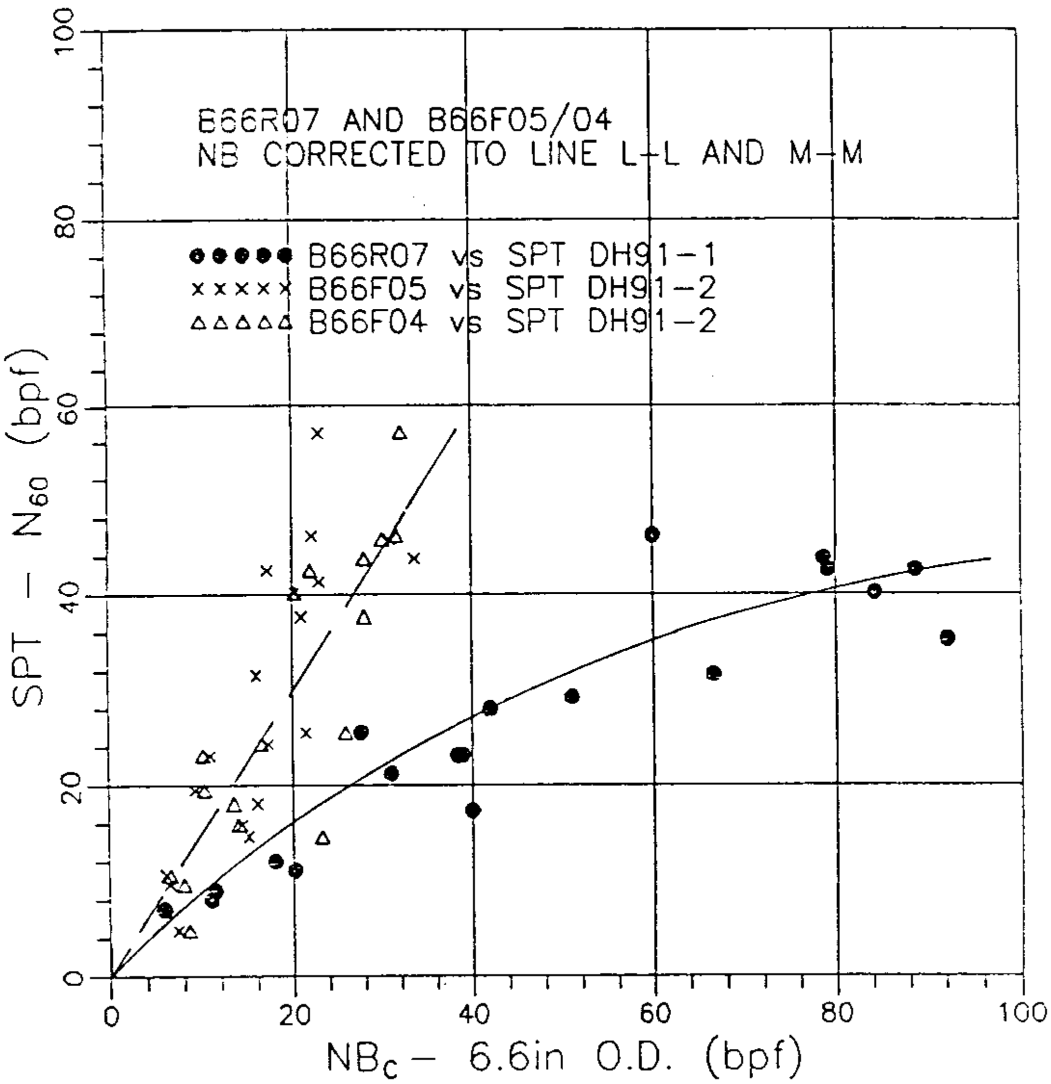


SKETCH OF FOUNDEX OVERSIZED BITS WITH MUD INJECTION



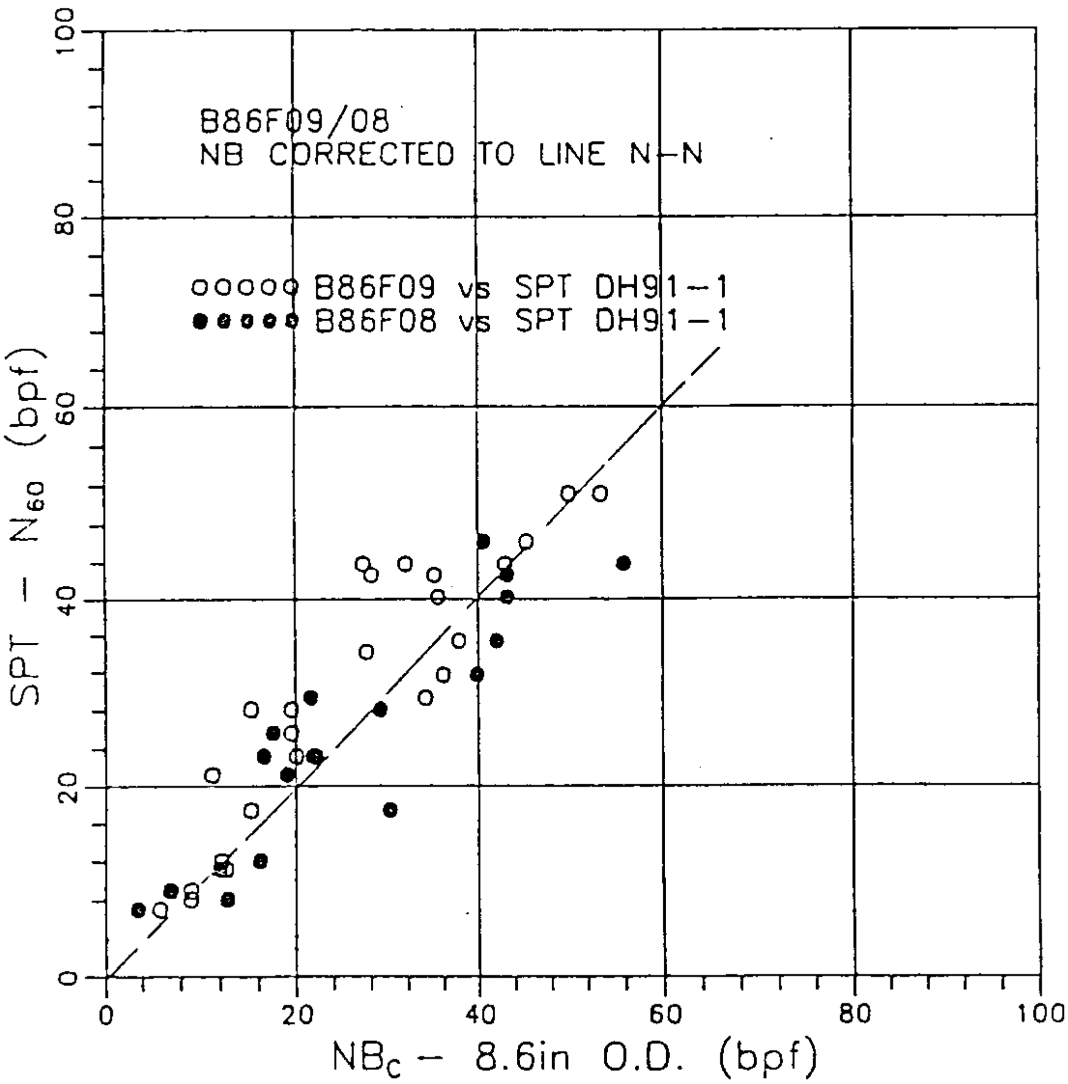
KLOHN LEONOFF FOUNDEX

FIGURE 6.5



KLOHN LEONOFF FOUNDEX

FIGURE 6.17



KLOHN LEONOFF FOUNDEX

FIGURE 6.18

APPENDIX C

Excerpt: Dynamic Measurements of
Becker Hammer Drill Penetration Tests
by Goble Rausche Likins and Associates, Inc.

GRL

Goble Rausche Likins and Associates, Inc.

March 3, 1993

Mr. David Otto
Foundex Inc.
1333 Lincoln Street, Suite 221
Bellingham, Washington 98226

Re: Dynamic Measurements of Becker Hammer Drill Penetration Tests
South Coldwater Creek Bridge, Foundation Densification
Mount St. Helens National Monument, Washington GRL Job No. 926017

Dear Mr. Otto:

This report presents results obtained from dynamic measurements of Becker Penetration Tests on November 27-30, 1992 and January 8-12, 1993. The primary goal of the dynamic measurements was the evaluation of energy transferred from the driving system to the Becker drill rod. This energy transfer measurement and other results from the dynamic measurements were desired as an aid to interpretation of the recorded Becker Hammer Drill blow count.

Field measurements were made with strain transducers, accelerometers, and a Pile Driving Analyzer^{TM} (PDA) manufactured by Pile Dynamics, Inc. The dynamic measurements were displayed on the PDA screen, and computed results were displayed, printed and stored. The recorded dynamic measurements were also stored digitally to allow later replay. Appendix A contains further description of our testing equipment and analyses.

Test Details

Becker Hammer Drill System

The truck mounted HAV 180 Becker Hammer Drill used for all testing was owned and operated by Foundex, Inc. of Bellingham Washington. The system included a double acting diesel hammer mounted in a mast, drill rod, and appurtenances to mix and pump drilling fluid. The drill rod consisted of an outer casing containing a smaller diameter, centralized steel tube. Driving stresses were transmitted from the hammer to the shoe by the casing. The casing string was assembled with threaded joints at 8 ft intervals (typical). The inner tube was mated with O-rings at each casing joint, and moved within the casing such that axial stress transfer from the hammer or casing to the tube was minimal.

MAIN OFFICE: 4535 Emery Industrial Parkway • Cleveland, OH 44128 • (216) 831-6131 • Fax (216) 831-0916 • Telex 985662

Branch Offices	BOULDER, CO	CHICAGO, IL	LOS ANGELES, CA	ORLANDO, FL	PHILADELPHIA, PA	SEATTLE, WA
Phone:	(303) 494-0702	(708) 776-9890	(714) 548-1174	(407) 826-9539	(215) 459-0278	(206) 624-0220
Fax:	(303) 494-5027	(708) 776-9932	(303) 494-5027	(407) 859-8121	(215) 459-0279	(206) 871-5483

The annulus between the casing and the tube contained drilling fluid pumped from the top. Fluid flowing down the annulus exited the casing through perforations near the bottom of the rod, and was intended to surround the entire length of the casing. The bottom of the rod rested inside an oversize driving shoe. The outside diameters of the shoe and casing were 8.625 and 6.625 inches, respectively. Foundex developed this system with the oversize shoe and drilling fluid to minimize skin friction during penetration tests.

Hammer and Driving System

The hammer was an ICE 180 double acting diesel hammer with a nominal ram weight of 1.73 kips and a manufacturer's maximum energy rating of 8.1 kip-ft. Impact forces were transmitted from the hammer, to a specially fabricated ball and socket, into a short section of pipe called the "spout", and then into the casing. It was possible to alter the alignment and position of the hammer by tilting or translating the mast and by hydraulic leveling of the truck. These means were used to maintain reasonable alignment between the hammer and casing if the position or inclination of the casing varied during driving.

Soils

Detailed information on subsurface materials and conditions is beyond the scope of this report. Each penetration test was in the right of way of proposed SR-504 at the South Coldwater Creek crossing. General soil descriptions provided to GRL indicated volcanic debris for a thickness of approximately 140 ft. This debris resulted from the 1980 eruption of Mount St. Helens and was thought to consist primarily of ash, silts, sands, gravels and boulders.

Soil in the vicinity of the test holes had been subject to two rounds of blast densification. After an initial densification, two test holes were made on each side of the creek; these first four instrumented tests were designated with an FBPT (example; FBPT D2). Following a second round of blasting, the instrumented tests were repeated and an "S" was added to the hole labels (example; FBPT D2-S).

Becker Drill Rods

The nominal cross sectional area of the 6.625 inch O.D. Becker casing was computed as 11.8 square inches. The inner floating in the rod sections did not appear to transmit axial driving stresses, thus we did not include the tube in our dynamic analyses. GRL's instrumentation was attached to a sub of approximately 2 ft length that was continually placed on the top of the casing string as the depth of penetration increased. Detailed measurements of this sub made at the University of British Columbia indicated a cross sectional area of 12.11 square inches where the gages were attached to this sub.

Test Sequence

The first four instrumented Becker tests were made from November 27-30, 1992. The second set of four holes began January 8, 1993 and ended January 11, 1993. The sequence and depth of these holes is summarized below.

TEST	DATE	FINAL DEPTH
FBPT D2	11/27/92	140 ft
FBPT I3	11/28/92	142 ft
FBPT P2	11/29/92	143 ft
FBPT L4	11/30/92	143 ft
FBPT P2-S	01/08/93	130 ft
FBPT L4-S	01/09/93	142 ft
FBPT I3-S	01/10/93	142 ft
FBPT D2-S	01/11/93	142 ft

On January 12, 1993, after the second round of tests, Foundex and GRL conducted an instrumented closed end 6.625 inch O.D. Becker test. This test did not employ drilling fluid or an oversize driving shoe. GRL also made dynamic measurements on a Standard Penetration Test (SPT) hole in the vicinity of FBPT P2-S. The final Becker test, and the SPT instrumentation are beyond the original scope of this investigation. However, individual results from these tests appear in Appendix B of this report.

RESULTS

Tables 1 to 4 summarize the blow count and energy transfer data obtained from the instrumented Becker tests. Each result is given as an average over a 5 ft interval. However, at the start and end of each test, the interval may include data for less than a 5 ft increment. The measured transfer energy was divided by the hammer's 8.1 kip-ft energy rating to yield an average transfer efficiency for each interval. Figures 1-8 provide a graphical summary of our measurements as a function of depth for each test.

The level of energy transfer measured on this site is consistent with measurements made on Becker Penetration Tests at other sites, including those reported by G. G. Goble (1192) and Sy and Campanella (1992). Figure 9 contains a statistical summary of data collected by GRL on a large number of sites were double acting diesel hammers were used to drive steel piles. These data suggest that transfer efficiency on Becker tests is somewhat lower than levels common to pile driving.

Near the start of each hole, the measured energy transfer was relatively variable. This pattern was partially attributed to variables in alignment between the hammer and casing which may have effected transfer efficiency. Significant alignment variations were common during the start of test holes. Additional causes for energy variation include the hammer and air temperature, hammer lubrication, and the tightness of the threaded drill rod connections. The hammer fuel supply setting is variable, and is controlled by the driller. Generally, full fuel is used unless hard driving causes excessive hammer rebound (racking). Although such hard driving and racking seldom occurred on this project, it is likely that variations in the fuel setting and in the amount fuel injected caused some variation of energy transfer.

The observed variation in energy transfer assists comparison between blow counts at different penetrations and locations. For example, at Location D2 during the initial test, the average blow count and transferred energy were 4.9 blows per foot (BPF) and 2.4 kip-ft, respectively. Test D2-S (after blasting) yielded an average blow count of 26.6 BPF with an average of 2.2 kip-ft of energy transfer. Compared with the Location D2, the average transfer energy for D2-S was approximately 8 percent lower and the blow count was approximately 540 percent higher. This comparison confirms the significance of the large increase in apparent penetration resistance after blasting. A similar, but less marked, trend was observed at all four test locations.

Becker Penetration tests may include the effects of significant skin friction resistance that can increase the overall penetration resistance. Unless the magnitude of this friction is known, proper strata specific interpretation of the field penetration resistance is not possible. The penetration tests conducted by Foundex, with an oversized driving shoe and a mudded hole was intended to eliminate friction. The effectiveness of this system in reducing friction can be confirmed by evaluation of the dynamic measurements. Dynamic measurements taken on D2-S are shown in Figure 10 and show the measured force very nearly equal to the product of the measured velocity and the rod impedance (V*Z) from the time of impact until the time of the end-reflection. The measured proportionality between force and velocity indicates that friction forces are very small.

As a further on friction, GRL analyzed the measurements of D2-S at 140 ft shown in Figure 10 using the CAse Pile Wave Analysis Program (CAPWAP®). A description of the CAPWAP method is given in Appendix A, and detailed CAPWAP results are given in Appendix C. The total static resistance computed with CAPWAP was 30 kips, with 2.2 kips of friction and 27.8 kips of end bearing. Visual analysis of data for other depths and other tests at this site indicate records with proportionality similar to that of Figure 10. Thus, we conclude that friction resistance above the shoe was a very small portion of the driving resistance and that friction had little effect on the observed blow counts for the eight mudded tests.

Because skin friction was very low, the observed blow count is primarily a function of the soil resistance on the driving shoe and the average transfer energy over a given interval. To assist in making relative comparisons of soil resistance at the shoe, the observed blow count can be normalized to a set energy level. Sy and Campanella (1992) have proposed normalizing to a transfer energy of approximately 2.44 kip-ft. This energy level is 30 percent of the rated energy of the ICE 180. The normalized blow count, N_{b30}, is computed as the product of the measured blow count and energy divided by 2.44 kip-ft. Tables 6-9 present a summary of the observed and normalized blow counts for each test.

March 3, 1993

It was a pleasure working with you, and we appreciate your interest and cooperation in collecting extra measurements of SPT and regular BPT holes at this site. Please contact us if you have any questions about this report.

Very truly yours,

GOBLE RAUSCHE LIKINS
AND ASSOCIATES, INC.

Robert Miner

Robert Miner

C. Michael Morgano

C. Michael Morgano

RM:bs
Re:wa3/93

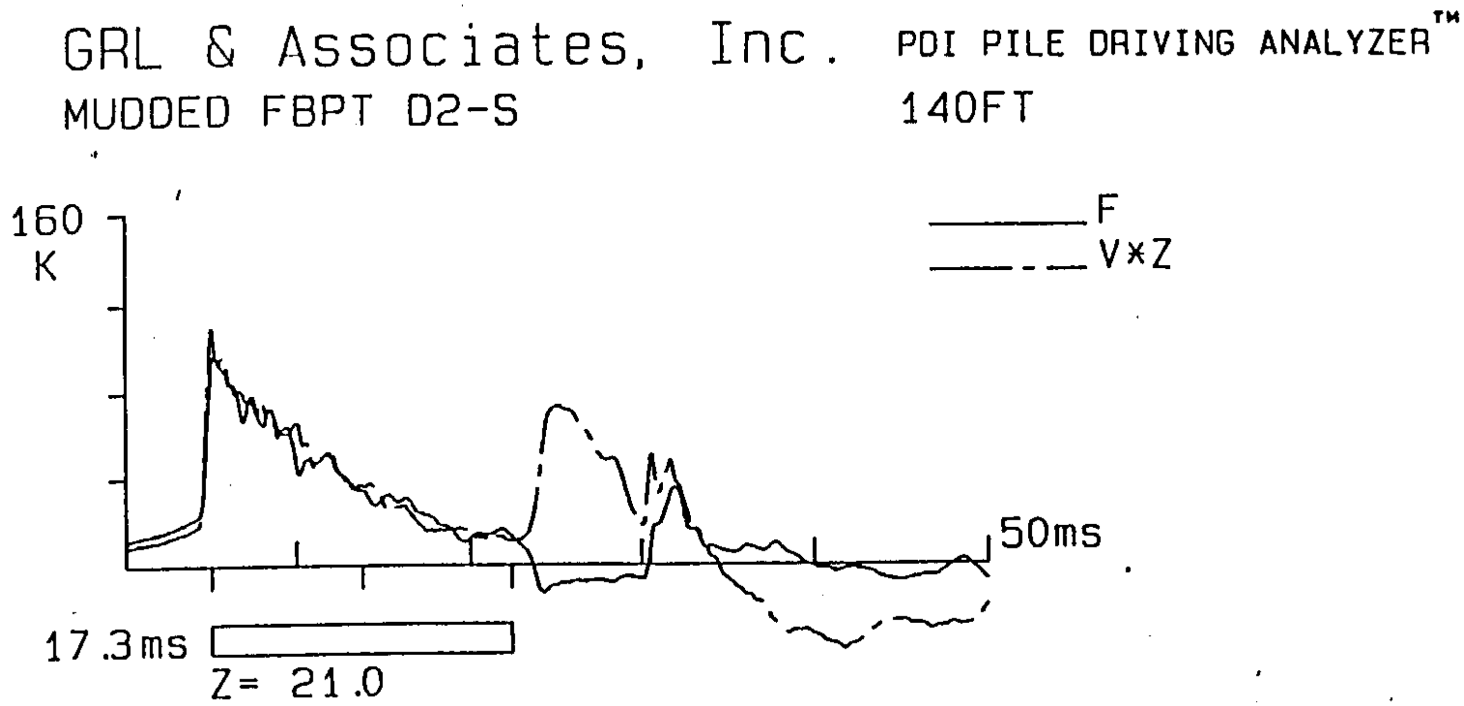


FIGURE 10: Dynamic Measurements of Force and Velocity, D2-S, 140 ft.

Summary of Dynamic Measurements of SPT, Jan. 11, 1993
South Coldwater Creek Bridge, Foundation Densification
Hammer H1: 140 lb Safety Hammer, rope and cathead
Hammer H2: 140 lb Donut type, rope and cathead
Rod: AW with Mayhew upset thread

Approx. Depth ft	Approx. Blow Count blow/set	Average Maximum Force kips	Average Transfer Energy (a) lb-ft	Averge Transfer Efficiency (b) percent	Remarks
65.0	33/6"	19	150	0.43	H1
65.5	32/6"	19	150	0.43	H1
66.5	43/6"	18	150	0.43	H1
67.0	>50/6"	18	110	0.31	H2, overdrive sampler
85.0	37/6"	21	160	0.46	H1
80.5	32/6"	20	140	0.40	H1
81.0	32/6"	22	150	0.43	H1
90.0	30/6"	20	130	0.37	H1
90.5	36/6"	20	140	0.40	H1
91.0	40/6"	20	140	0.40	H1
125.5	>50/6" ?	21	160	0.46	H1
126.0	>50/6" ?	17	120	0.34	H1
126.5	>50/6" ?	19	150	0.43	H1
126.5	>50/6" ?	21	170	0.49	H1, WD-40 on guide ro
126.7	>50/6" ?	16	100	0.29	H2

GRL & ASSOC, INC. 2/26/93

REGULAR BPT, D2-S, JAN. 11, 1993

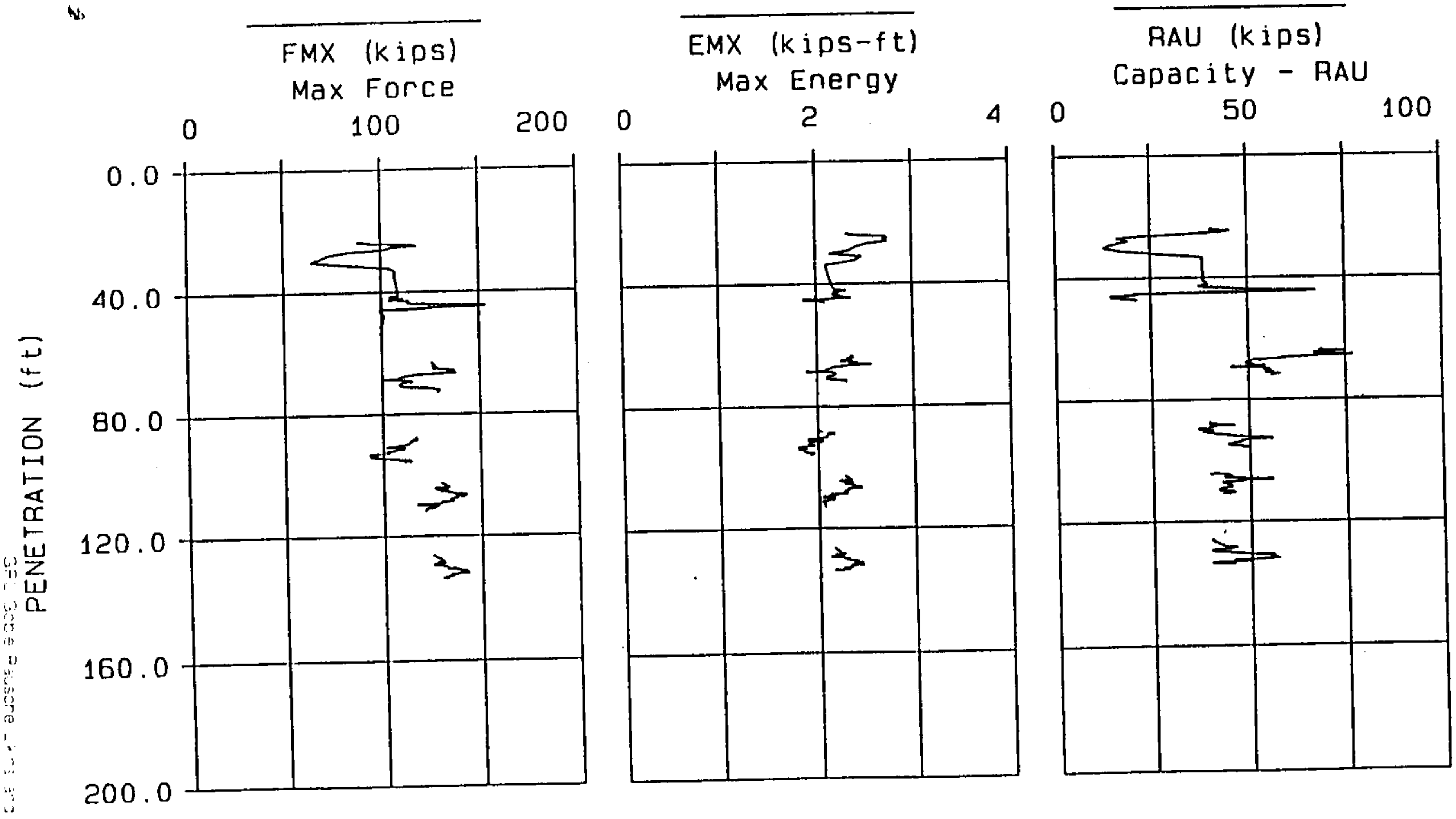


FIGURE B3: Summary of Case Method Results, Closed End BPT Test Without Shoe or Drilling Mud, Location D2-S.

APPENDIX D

Excerpt: Vibration Monitoring Data,
by Geo Recon International

GEO RECON INTERNATIONAL

applied geophysics

January 11, 1993
J92-560

FOUNDEX
14613 64th Avenue
Surrey, British Columbia
Canada V35 1X6

Attn: Dennis Diggle

RE: Vibration Monitoring Data, Coldwater Creek Bridge

Enclosed are the computer outputs of the S-6 Sinco generated blast monitoring data for the two blasting events on the west bank of Coldwater Creek that occurred on Sunday, December 13 and Tuesday, December 15,1992. The Sunday event, consisting of 15 shot holes (Event #00) produced a maximum peak particle velocity vector of 9.16 inches per second (ips) and the Tuesday event consisting of 8 shot holes (Event #1) produced a maximum peak particle velocity vector of 7.28 ips. The shot hole plan and geophone sensor locations are shown in Figure 1.

Shot hole explosives parameters were different from those previously provided and called for changing the maximum particle velocity expected to be well above the 5 ips stated in the initial RFP. Revised explosives information is in the client's records. All geophone sensor packages remained buried in place for both shots. For Event #00 the geophone packages were buried in dry pea gravel backfill. As a result of the densification and dewatering action from this shot the geophone sensors at locations 1 and 2 were under water during Event #1. The geophone sensor packages for instrument No.1, SN:3225 did sustain damage and required repair.

Three SINCO, S-6 blast monitoring systems were used for this project, with each system monitoring two locations. Instrument No.1 with serial number 3225 monitored the shot pattern center location (geophone sensors package shown at station #1) on channel A, and the shot pattern edge location (geophone station #2) on channel B. Instrument No.2, SN:4420 monitored station #3 on channel A and station #4 on channel B. For the above 4 stations the S-6 monitors were set to record to 30 ips and to trigger at 1% which is 0.3 ips. Instrument No.3, SN:3083 monitored station #5 with the 30 ips range setting on channel A and station #6 at the same location but with a 3 ips range setting and .03 ips trigger threshold on channel B. The geophone sensor locations (figure 1) were selected by client personnel. All longitudinal geophone axes are aligned in-line toward the center shot hole.

P.O. Box 55189 Seattle, Wa. 98155 USA (206) 362-9484

The blast monitoring data for the six locations are presented in the six computer output sections submitted with this letter: three sections for event #00 and three section for event #1. The event times for the three instruments vary by about 45 seconds because these times were set independently from one another and were not calibrated to a single time. Under the serial number and operator initials listed in the heading are the numbers 1, 2 or 3 for instrument identification. There is a peak particle velocity in inches per second for channels A and B listed for the longitudinal, vertical and transverse directions or axes followed by the vector sum (resultant vector). The remaining time domain, graph and frequency data relate to the vector sum of the peak event.

Note that although the event window is 5 seconds, the recorded data presented relate only to the peak event within that window and for 200 milliseconds before and 400 milliseconds after that peak event. These data were obtained in the Graph and Peaks mode and those sections of the S-6 manual relating to interpreting data are attached to this letter.

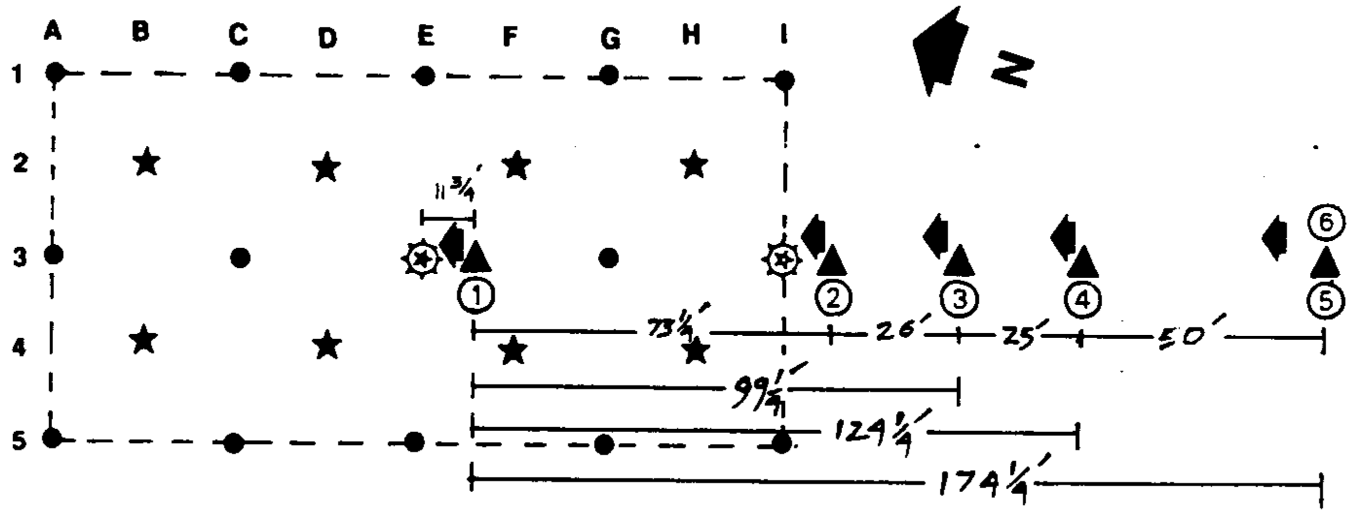
Although GRI was not retained for interpretive consultation of the blast data, the following general observations may be of interest. Event #00 being greater than event #1 may have resulted more from the geophone stations being closer to the shot holes than from the increase in the number of shot holes and total amount of explosives. The longitudinal and transverse vector data of channel A of instrument #1 (center shot data) may be questionable because multiple blast wave fronts hit the sensors from all directions. The variations in the longitudinal and transverse horizontal components suggest a shear movement in the blast area. A shearing motion may be evident from the crack and joint pattern caused by event #00 and may also be evident on the Foundex video record. A series of photos are enclosed which were taken of the cracks caused by event #00. Photos of event #1 are also enclosed.

We trust the above will be sufficient for your requirements. If you require further assistance or have any questions please let us know.

For: Geo Recon ~~International Ltd.~~

Clyde A. R

Clyde A. Ringstad
Principal Geophysicist



15 SHOTHOLE PATTERN (13 DEC 92)

8 SHOTHOLE PATTERN (15 DEC 92)

CENTER SHOTHOLE

INLINE EDGE OF PATTERN SHOTHOLE

PATTERN BOUNDARY

GEOPHONE SENSORS PACKAGE LOCATION

DIRECTION OF LONGITUDINAL AXIS

③ GEOPHONE SENSORS PACKAGE NUMBER

D1 SHOTHOLE GRID DESIGNATION

WEST BANK

1" = 40'

COLDWATER CREEK BRIDGE	SITE PLAN

J92-560 GEO-RECON INTERNATIONAL FIG 1

EVENT #00 15:39:40 13 DEC 92

EVENT WINDOW = 5 SECONDS

SN:S6=3225 A=0619 B=0640 SL=0000

OPERATOR = MIE

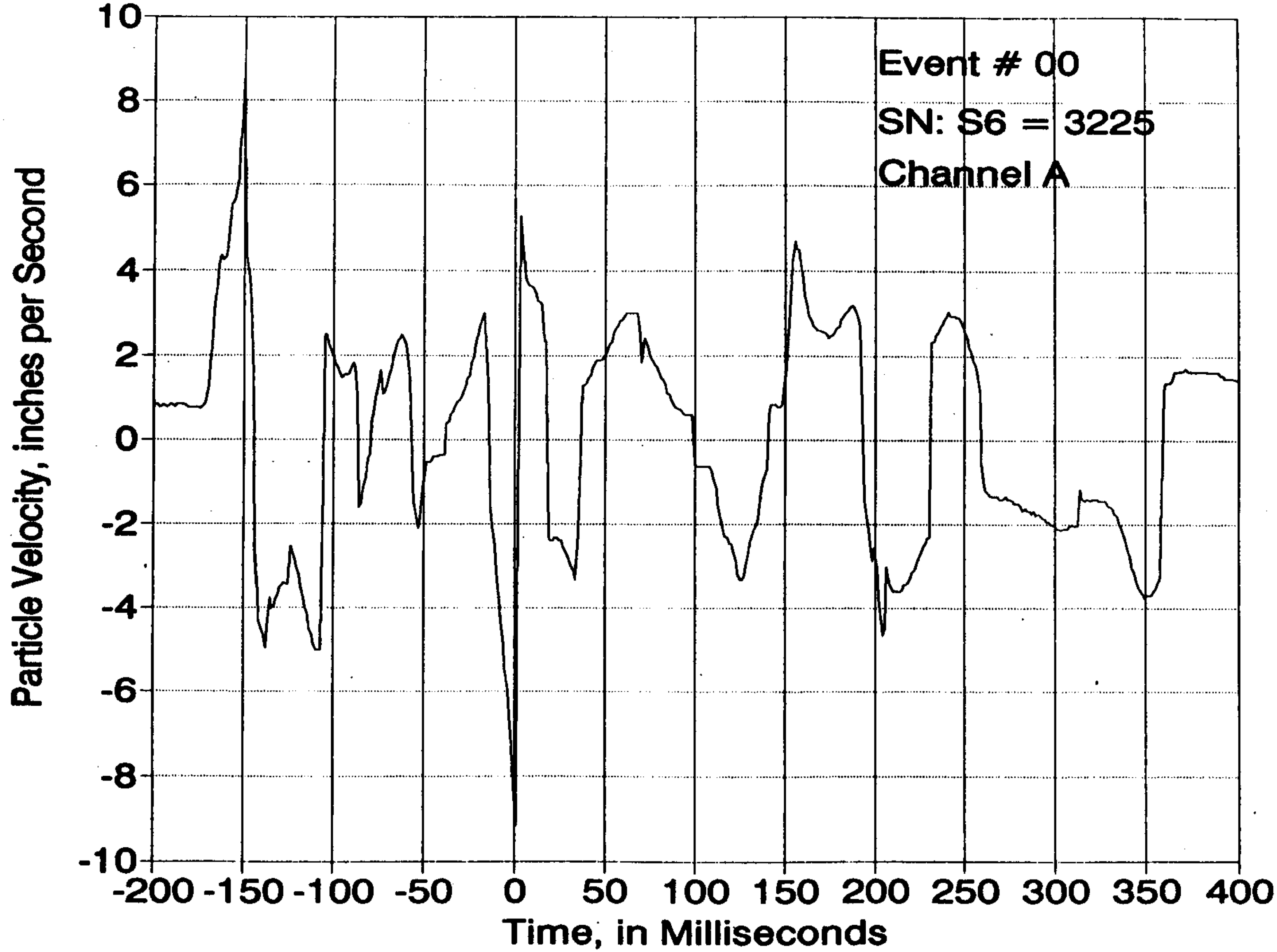
INSTRUMENT 1

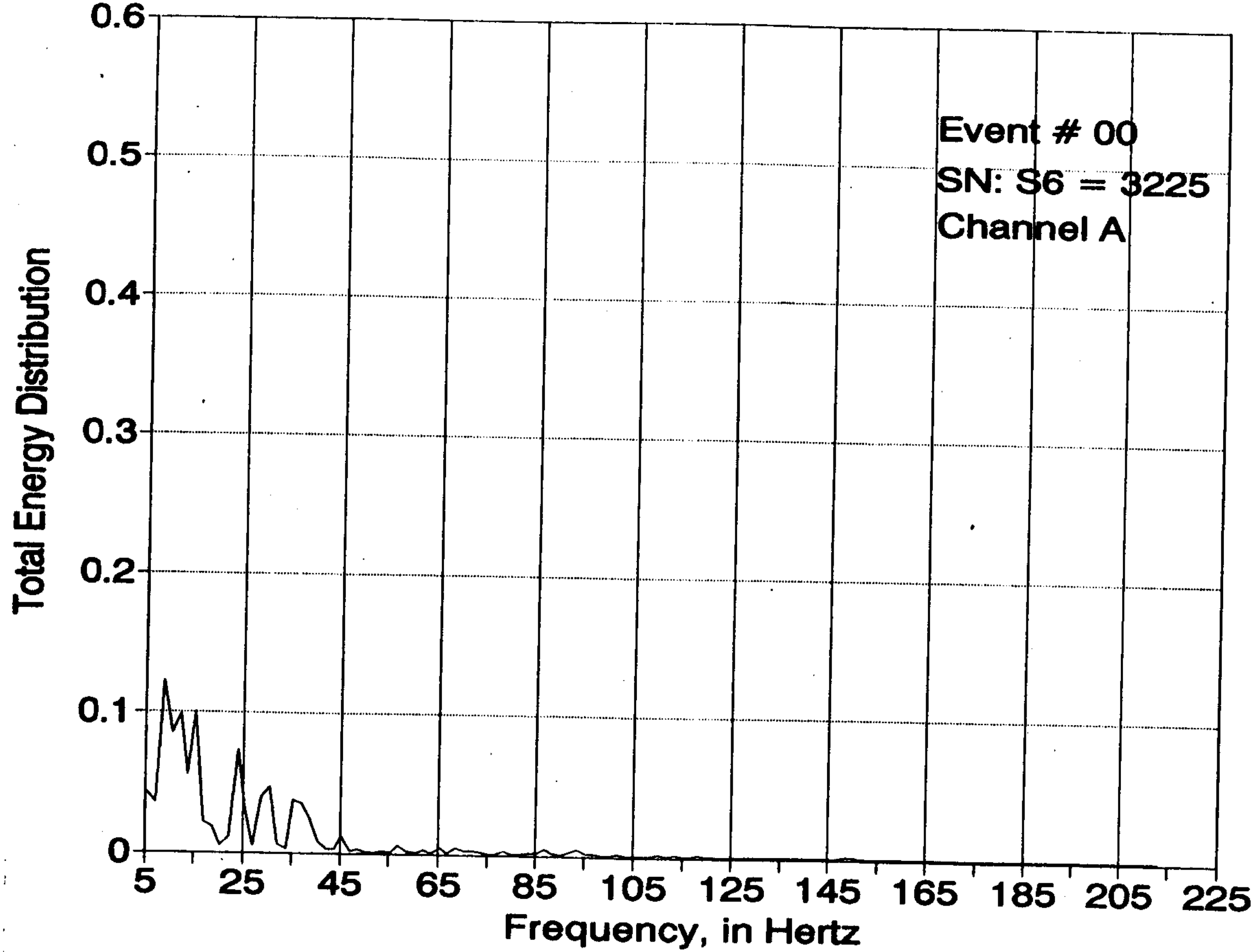
Location Notes:

Transducer Package A at center of shot pattern 11.75 feet South of center hole.

Transducer Package B at edge of pattern shot but 10 feet South of pattern line. A to B distance is 73.25 feet.

CHANNEL	PEAK		
A LONG.	-4.23 IPS		
A VERT.	-8.63 IPS		
A TRAN.	-4.81 IPS		
A VECTOR	9.16 IPS		
THLD	0.30 IPS	RANGE	30
B LONG.	-4.46 IPS		
B VERT.	-7.28 IPS		
B TRAN.	-6.63 IPS		
B VECTOR	7.28 IPS		
THLD	0.30 IPS	RANGE	30


Event # 00
SN: S6 = 3225
Channel A
Particle Velocity, inches per Second
10
8
6
4
2
0
-2
-4
-6
-8
-10
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
Time, in Milliseconds



Event # 00
SN: S6 = 3225
Channel A
Total Energy Distribution
0.6
0.5
0.4
0.3
0.2
0.1
0
5
25
45
65
85
105
125
145
165
185
205
225
Frequency, in Hertz

EVENT #00 15:40:25 13 DEC 92

EVENT WINDOW = 5 SECONDS

SN:S6=4420 A=0687 B=0692 SL=0000

OPERATOR = MIE

INSTRUMENT 2

Location Notes:

Transducer Package A is 26 feet south of Transducer Package B of Instrument 1 or 99.25 feet from the first geophone sensor package location.

Transducer Package B is 25 feet south of Transducer Package A or 124.25 feet from the first geophone sensor package location.

CHANNEL	PEAK		
A LONG.	-3.46 IPS		
A VERT.	-5.28 IPS		
A TRAN.	-5.58 IPS		
A VECTOR	6.34 IPS		
THLD	0.30 IPS	RANGE	30
B LONG.	-2.23 IPS		
B VERT.	-4.05 IPS		
B TRAN.	-1.76 IPS		
B VECTOR	4.11 IPS		
THLD	0.30 IPS	RANGE	30

EVENT #00 15:40:10 13 DEC 92

EVENT WINDOW = 5 SECONDS

SN:S6=3083 A=0484 B=0485 SL=0000

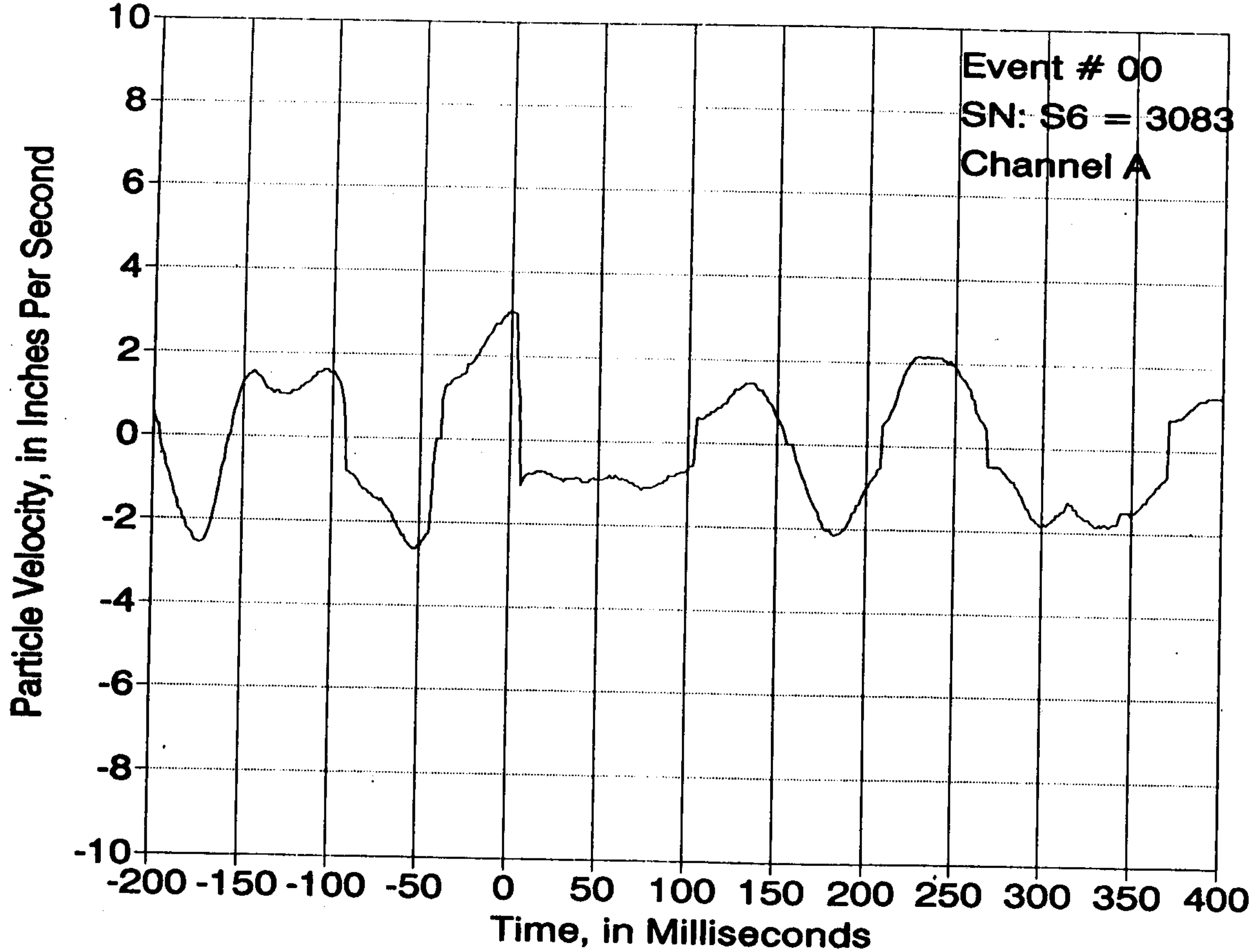
OPERATOR = MIE

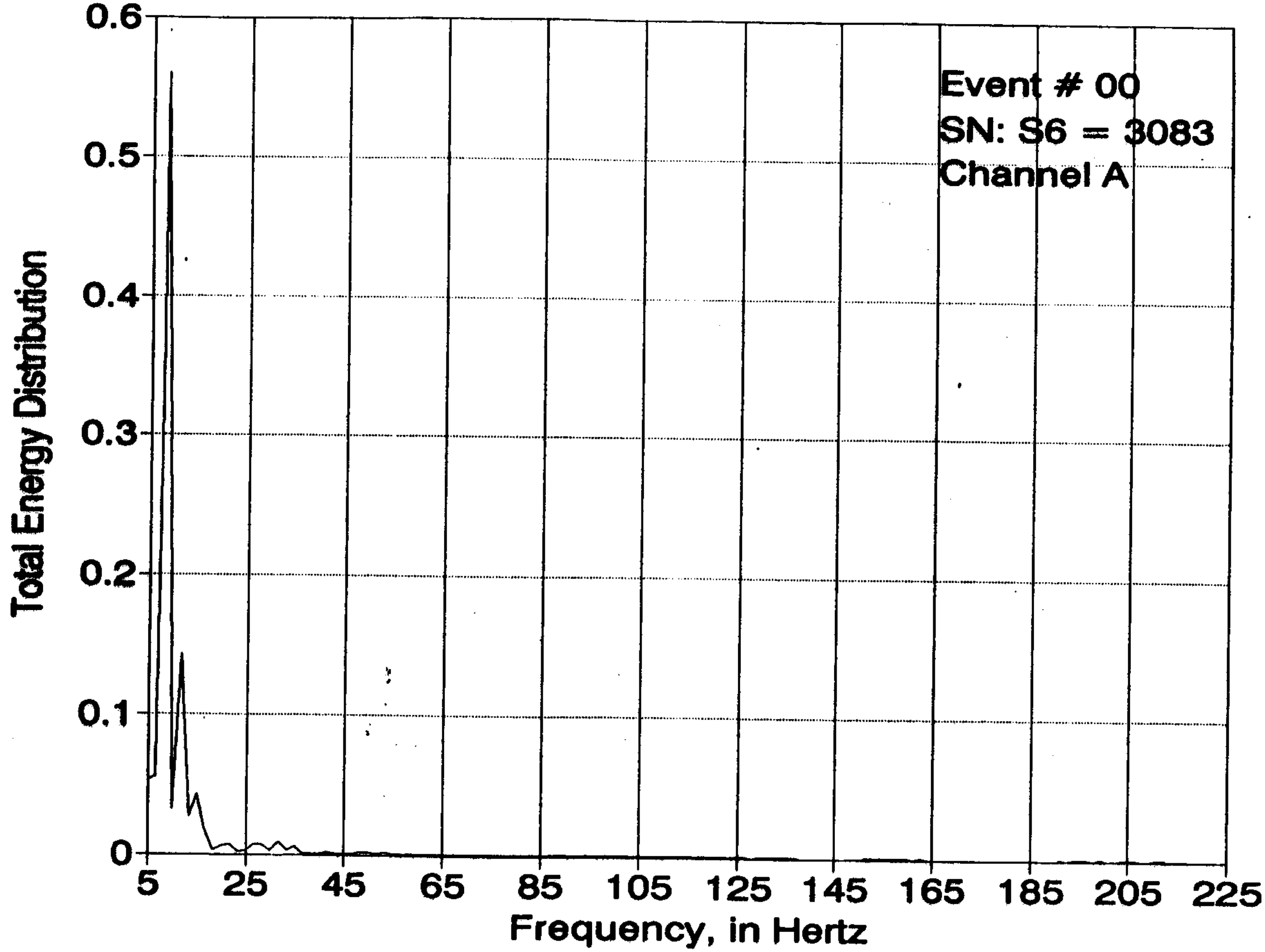
INSTRUMENT 3

Location Notes:

Transducer Package A and B are 50 feet South of Transducer Package B of Instrument 2 or 174.25 feet from the first geophone sensor package location.

CHANNEL	PEAK		
A LONG.	-1.64 IPS		
A VERT.	-2.99 IPS		
A TRAN.	+2.00 IPS		
A VECTOR	3.05 IPS		
THLD	0.30 IPS	RANGE	30
B LONG.	-1.521 IPS		
B VERT.	-2.947 IPS		
B TRAN.	+1.961 IPS		
B VECTOR	3.018 IPS		
THLD	0.030 IPS	RANGE	3


Event # 00
SN: S6 = 3083
Channel A
Particle Velocity, in Inches Per Second
10
8
6
4
2
0
-2
-4
-6
-8
-10
-200 -150 -100 -50 0 50 100 150 200 250 300 350 400
Time, in Milliseconds


Event # 00
SN: S6 = 3083
Channel A
Total Energy Distribution
0.6
0.5
0.4
0.3
0.2
0.1
0
5
25
45
65
85
105
125
145
165
185
205
225
Frequency, in Hertz

EVENT #1 09:57:26 15 DEC 92

EVENT WINDOW = 5 SECONDS

SN:S6=3225 A=0619 B=0640 SL=0000

OPERATOR = MIE

INSTRUMENT 1

Locations Notes:

Transducer Package A at center of shot pattern 11.75 feet South of center hole.

Transducer Package B at edge of pattern shot but 10 feet South of pattern line. A to B distance is 73.25 feet.

CHANNEL	PEAK		
A LONG.	-3.17 IPS		
A VERT.	-7.46 IPS		
A TRAN.	+4.58 IPS		
A VECTOR	7.63 IPS		
THLD 0.30 IPS		RANGE	30
B LONG.	-1.64 IPS		
B VERT.	-4.40 IPS		
B TRAN.	-2.64 IPS		
B VECTOR	4.64 IPS		
THLD 0.30 IPS		RANGE	30

EVENT #1 09:58:12 15 DEC 92
EVENT WINDOW = 5 SECONDS
SN:S6=4420 A=0687 B=0692 SL=0000
OPERATOR = MIE

INSTRUMENT 2

Location Notes:

Transducer Package A is 26 feet South of Transducer Package B of Instrument 1 or 99.25 feet from the first geophone sensor package location.

Transducer Package B is 25 feet South of Transducer Package A or 124.25 feet from the first geophone sensor package location.

CHANNEL		PEAK		
A LONG.		-1.82 IPS		
A VERT.		+4.23 IPS		
A TRAN.		-2.47 IPS		
A VECTOR		4.46 IPS		
THLD	0.30 IPS		RANGE	30
B LONG.		+1.23 IPS		
B VERT.		-3.76 IPS		
B TRAN.		-1.12 IPS		
B VECTOR		3.82 IPS		
THLD	0.30 IPS		RANGE	30

EVENT #1 09:57:56 15 DEC 92

EVENT WINDOW = 5 SECONDS

SN:S6=3083 A=0484 B=0485 SL=0000

OPERATOR = MIE

INSTRUMENT 3

Location Notes:

Transducer Package A and B are 50 feet South of Transducer Package B of Instrument 2 or 174.25 feet from the first geophone sensor package location.

CHANNEL	PEAK		
A LONG.	+1.41 IPS		
A VERT.	-2.41 IPS		
A TRAN.	+1.35 IPS		
A VECTOR	2.47 IPS		
THLD	0.30 IPS	RANGE	30
B LONG.	+1.544 IPS		
B VERT.	-2.419 IPS		
B TRAN.	+1.339 IPS		
B VECTOR	2.489 IPS		
THLD	0.030 IPS	RANGE	3

APPENDIX E

Excerpt: Surface-to-Downhole
Shear Wave Velocity Surveying

FINAL REPORT
SR 504 BLAST DENSIFICATION PROJECT
SURFACE-TO-DOWNHOLE SHEAR WAVE VELOCITY SURVEYING

Stephen P. Palmer
Washington Department of Natural Resources
Division of Geology and Earth Resources
P.O. Box 47007
Olympia, WA 98405-7007
(206) 902-1437

July 6, 1993

CONCLUSIONS

Borings WB-1 and WB-3 were drilled within 10 ft of each other on the west abutment pad of Bridge 12 on SR 504. A comparison of pre-blast and post-blast shear wave velocities (V_s) in these borings leads to these observations:

- V_s in the upper 20 ft did not significantly change after blasting from its nominal value of 500 ft/s;
- V_s increased by 50% in the depth interval 20 to 40 ft, from a pre-blast value of 530 ft/s to a post-blast value of 810 ft/s;
- V_s increase by nearly 20% in the depth interval 40 to 80 ft from a pre-blast value of 700 ft/s to a post-blast value of 830 ft/s;
- there was no significant change in V_s from 80 to 100 ft after blasting.

These observations lead to the following interpretation:

- there was no significant densification above the upper deck of explosives at a depth of 20 ft;
- significant densification occurred in the depth interval from 20 to 80 ft, and was greatest in the interval from 20 to 40 ft;
- no densification measurable as an increase in V_s occurred below 80 ft.

The following observations can be made from the V_s surveys performed in borings EB-1 and EB-2 drilled on the east abutment pad:

- o the low velocity surface layer (approximately 475 ft/s), similar to that observed on the west pad, is present to depths of 20-25 ft in both EB-1 and EB-2;
- o the velocity profile of EB-1 is very similar to that of the west pad borings, where the low velocity surface layer overlies soils with shear velocity of approximately 825 ft/s;
- o in boring EB-2 there is a thick layer of intermediate V_s (665 ft/s) underlying the low velocity surface layer, and extending to a depth of 55 ft;
- o between 55 and 70 ft in EB-2 the shear velocity increases to 840 ft/s, and below 70 ft V_s increases to 960 ft/s.

No conclusions on pre- and post-blast changes can be made for the east pad, as EB-1 and EB-2 were not drilled until after the blasting was completed. It is interesting to note that similarity of the velocity profiles in EB-1 and the west pad borings. In contrast, EB-2 has a thick layer of intermediate shear velocity not observed in any other post-blast survey.

Detailed analysis of samples collected during drilling of EB-2 and WB-3 suggests that a distinct stratigraphic sequence is present in both borings. This sequence includes an upper unit of brown-grey sandy gravel composed of lithic grains and Castle Creek andesite and basalt clasts, and a lower unit of light grey gravelly sand composed of glass shards and Kalama and Goat Rocks dacite fragments. Photographic reconstructions performed by Harry Glicken (Glicken, 1986) concluded that the May 18, 1980, rockslide and debris-avalanche on Mount St. Helens involved three distinct, large slide blocks, and that the Bridge 12 site is underlain by material derived from Glicken's slide block I. The toe of slide block I incorporated the Goat Rocks dacite dome, and this would presumably be the first material to arrive at the Bridge 12 site. The headscarp of slide block I would include older dacite and Castle Creek andesites and basalts, which would presumably follow in the path of material moving at the toe of the slide block. This model of slide transport would result in the stratigraphic sequence observed at the Bridge 12 site.

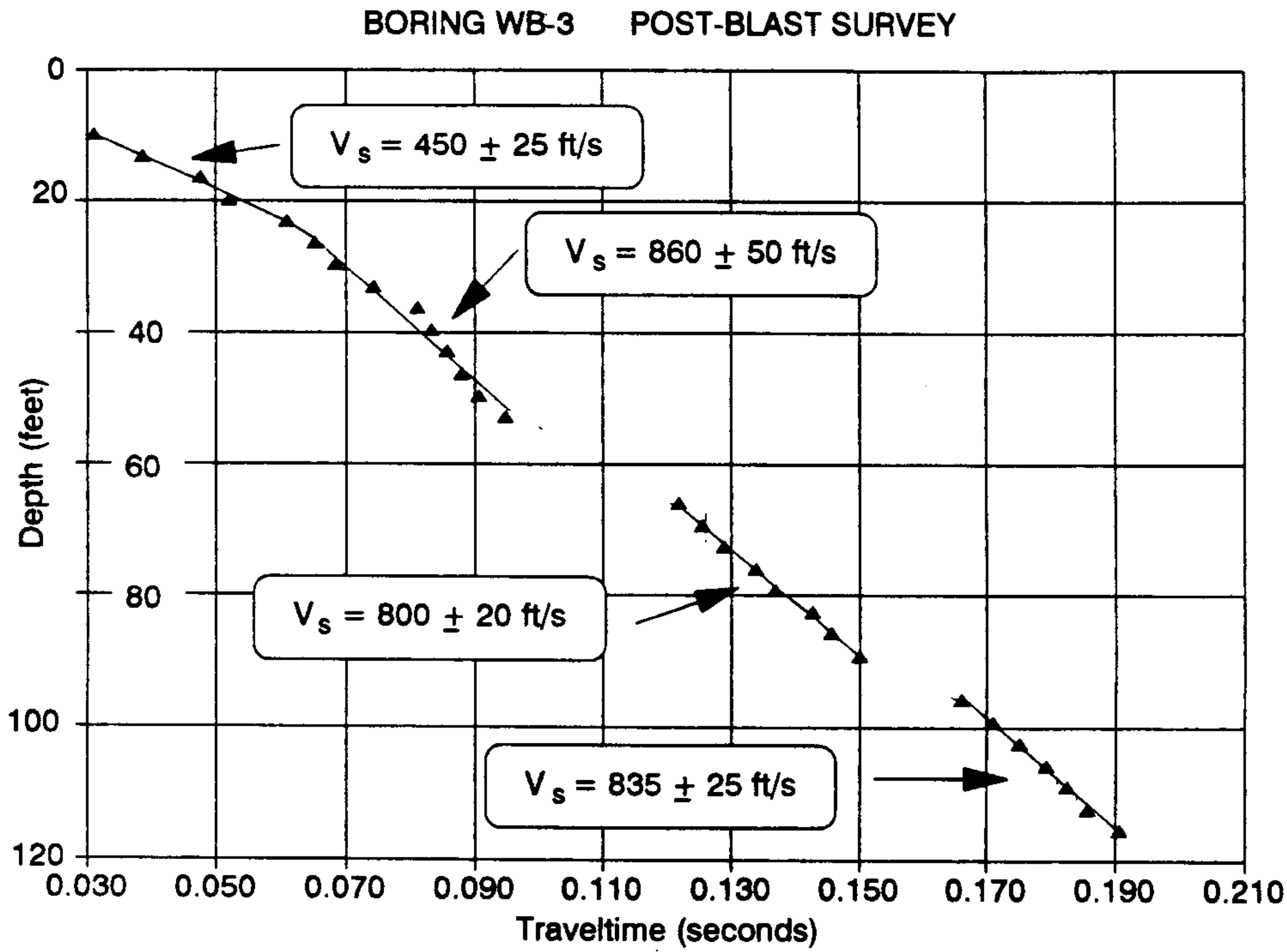


FIGURE 7 -- Traveltime data acquired in boring WB-3 on April 29, 1993.

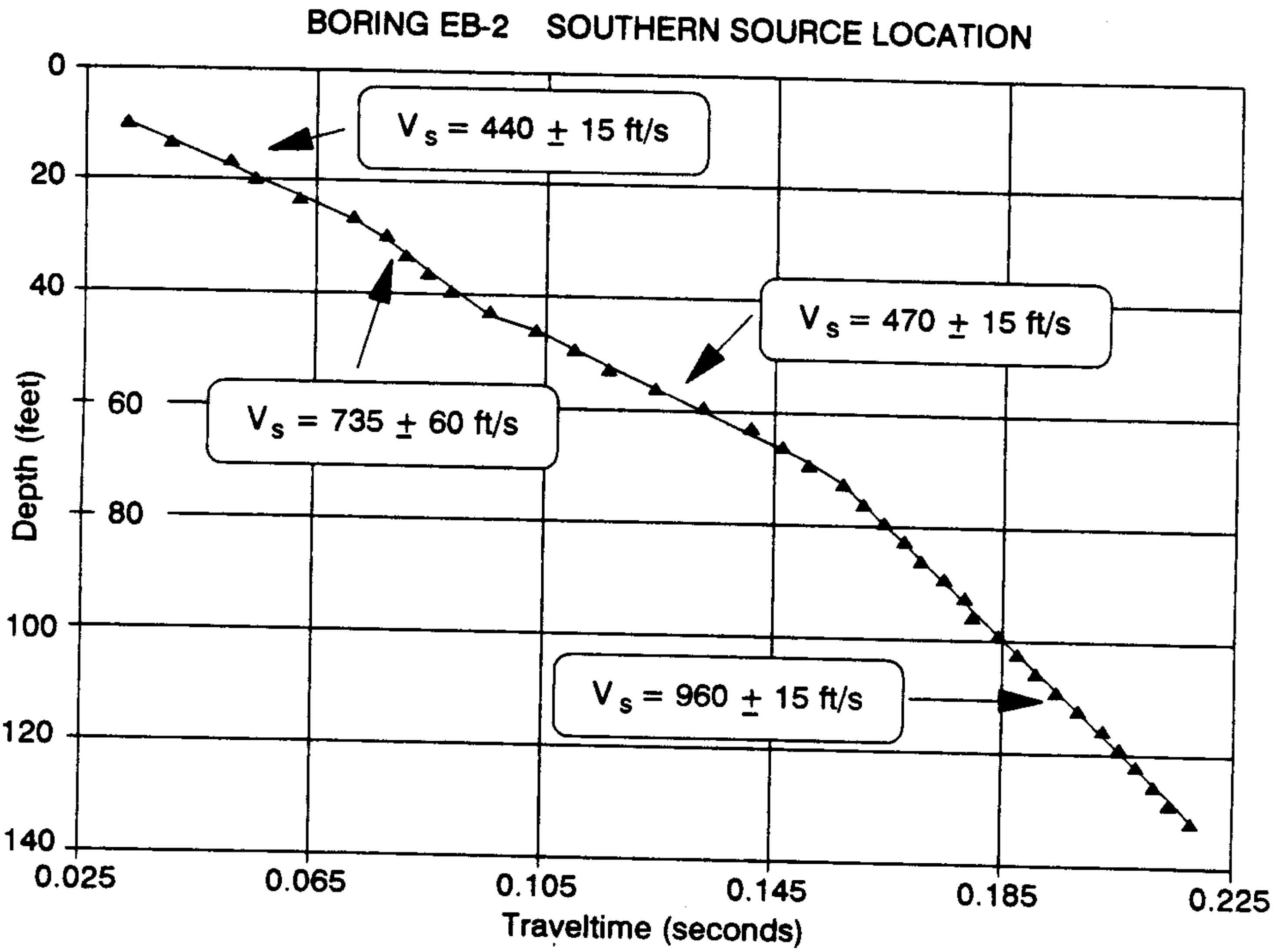


FIGURE 8 -- Traveltime data acquired in boring EB-2 on May 1, 1993.

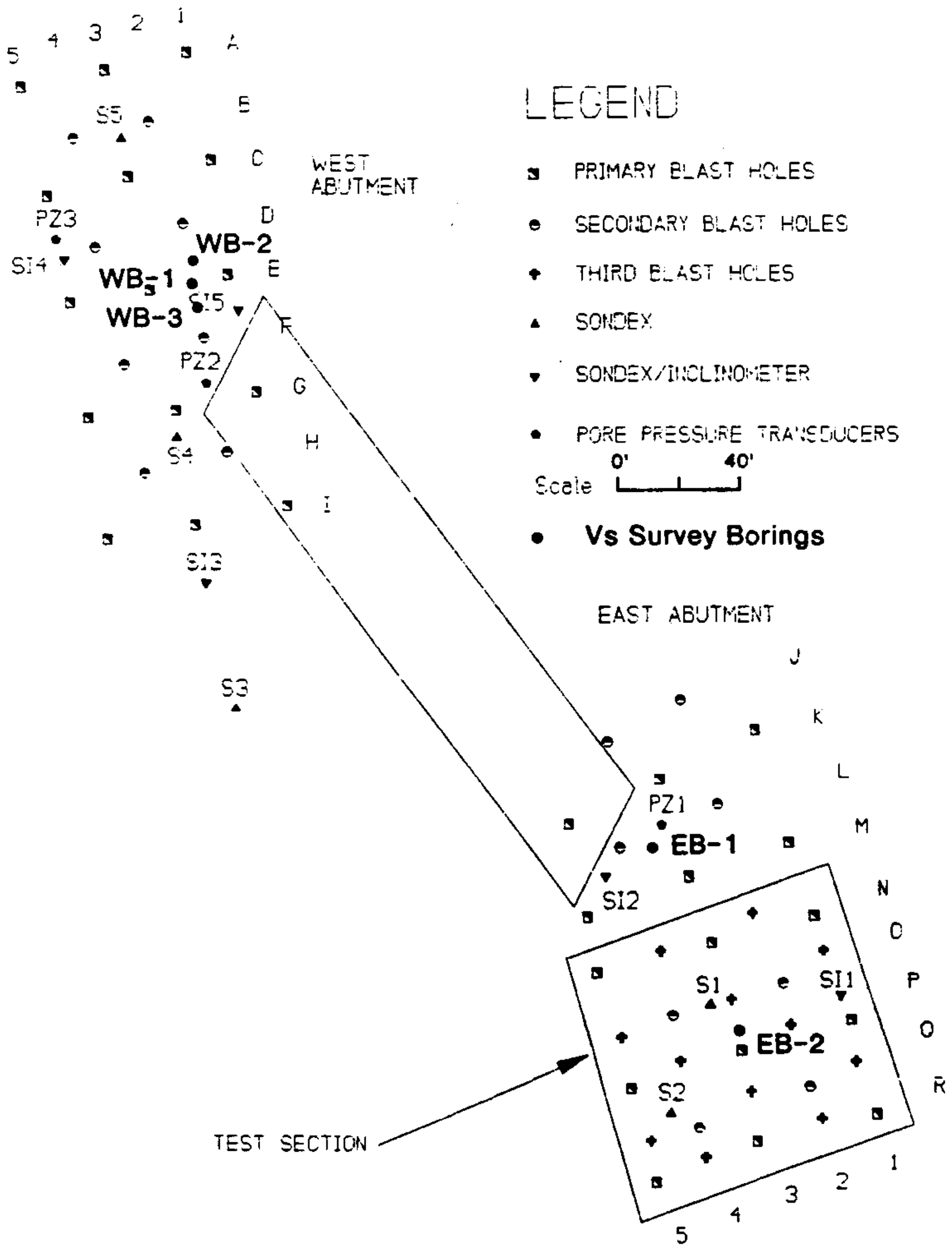


FIGURE 1 -- Location map of SR 504, Bridge 12 site showing borings used in the surface-to-downhole shear wave velocity surveying.

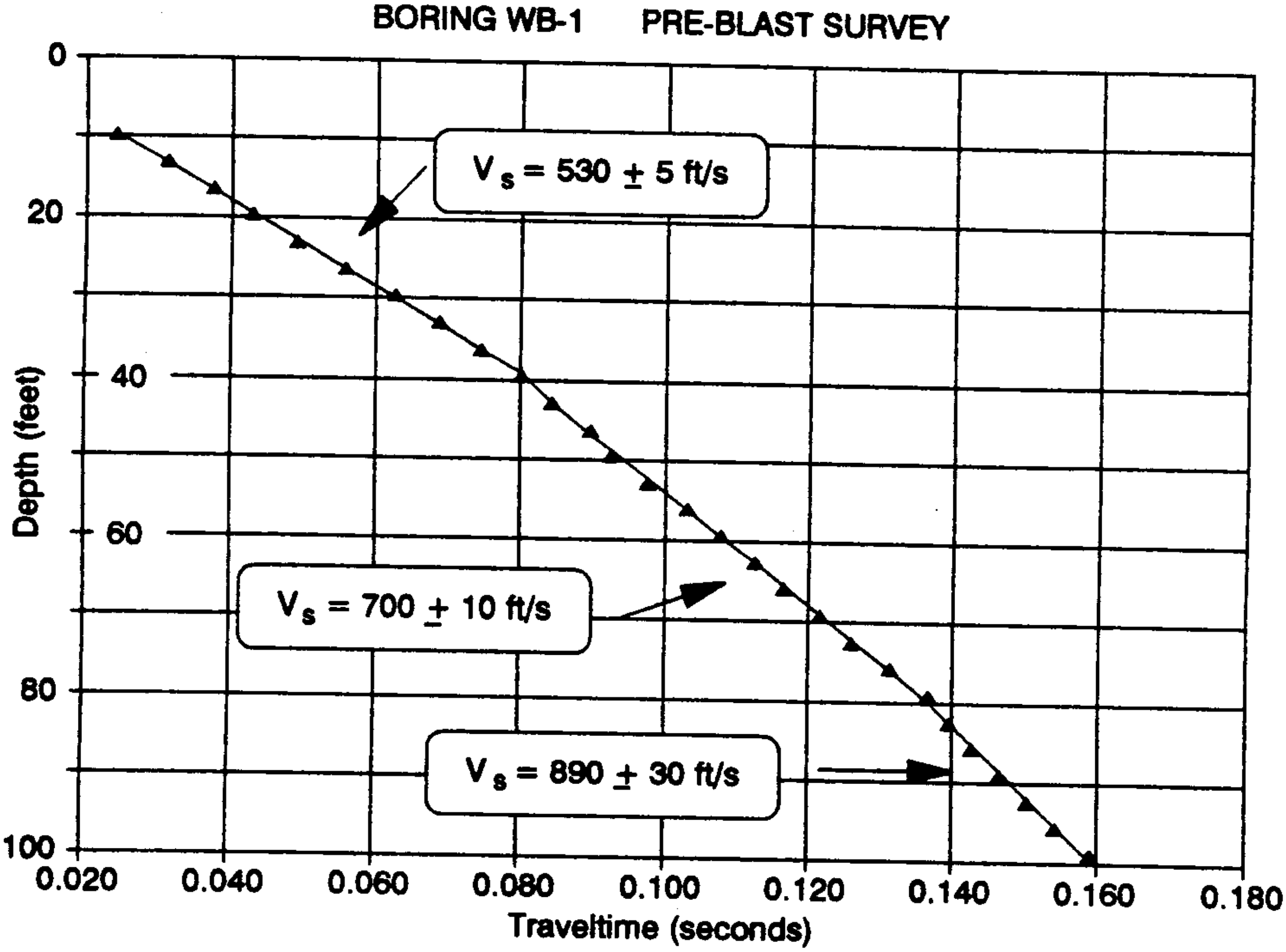


FIGURE 2 -- Traveltime data acquired in boring WB-1 on December 10-11, 1992.

FIGURE 10a -- Shear wave velocity profiles for east pad borings.

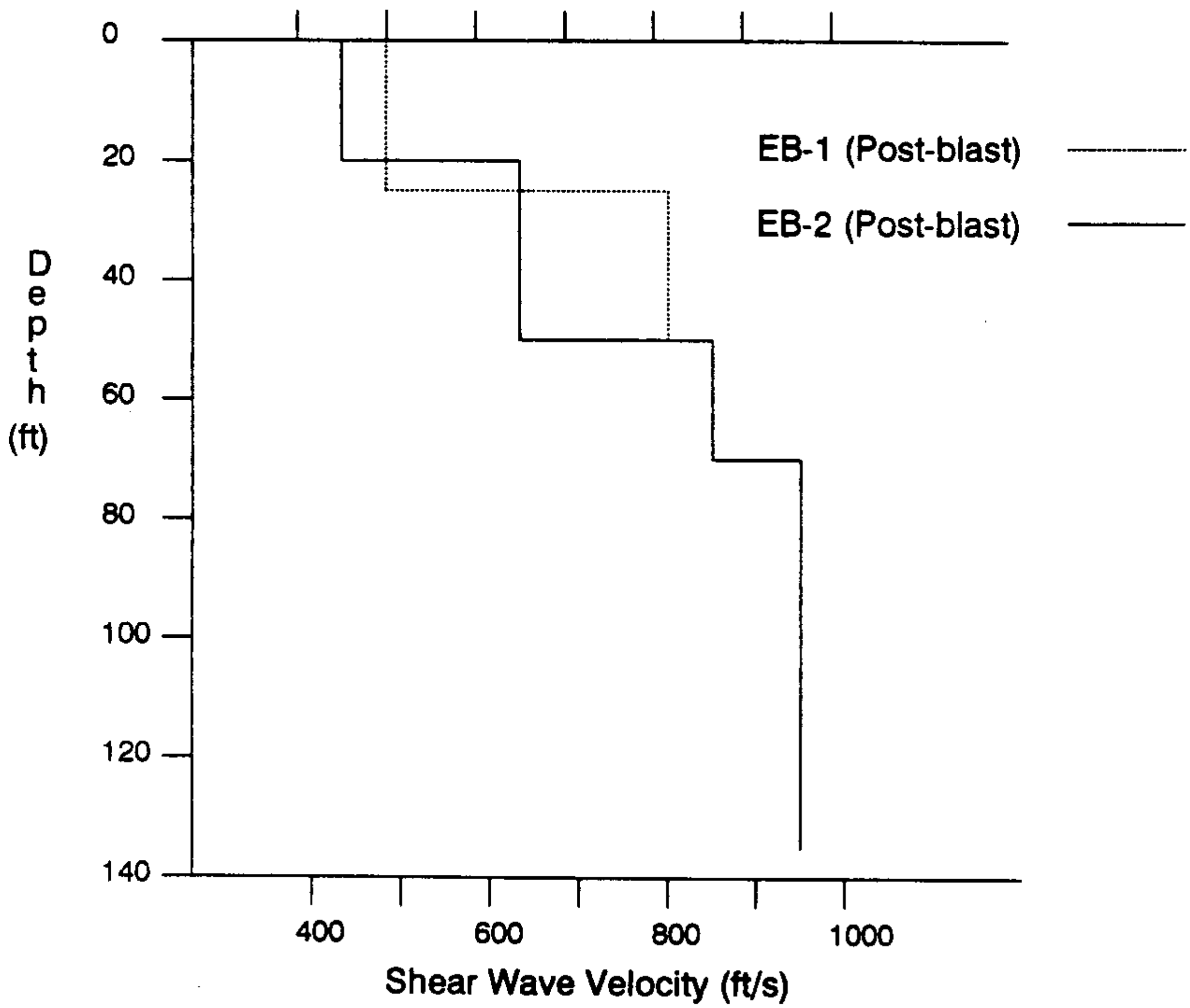


FIGURE 10b -- Shear wave velocity profiles for west pad borings.