Appendix G1 Geotechnical Engineering Summary Report

Appendices

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IRVINE GEOTECHNICAL Inc

GEOTECHNICAL ENGINEERING EXPLORATION UPDATE
AND REMEDIAL GRADING RECOMMENDATIONS
PROPOSED COMMERCIAL/RETAIL DEVELOPMENT
NU-WAY LIVE OAK LANDFILL

EAST OF 605 FREEWAY, WEST AND SOUTH LIVE OAK LANE

AND NORTH OF LIVE OAK AVENUE

IRWINDALE, CALIFORNIA

FOR MNOIAN MANAGEMENT, INC.

IRVINE GEOTECHNICAL, INC. PROJECT NUMBER IC 07034-I

NOVEMBER 23, 2010

April 4, 2011
IC 07034-I

IRVINE GEOTECHNICAL Inc

Mnoian Management, Inc.
P.O. Box 661238
Arcadia, California 91066-1238

Attention: Jim Mnoian

Subject

Geotechnical Engineering Summary Report
Nu-Way Live Oak Landfill
Remainder Parcel of Parcel Map As Per Book 186 P 79-82
Approximately 65 Acres
East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue
Irwindale, California

References: Reports by Irvine Geotechnical, Inc.:

> *Phase I - Geotechnical Engineering Exploration, Nu-way Live Oak Landfill, East of 605 Freeway, West and South Live Oak Lane, And North of Live Oak Avenue, Irwindale, California,* dated March 31, 2008;
> *Phase II - Geotechnical Engineering Exploration, Evaluation of Nu-Way Live Oak Landfill, Remainder Parcel of Parcel Map As Per Book 186 P 79-82, Approximately 65 Acres, East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue, Irwindale, California,* dated December 8, 2008;
> *Phase III - Geotechnical Engineering Exploration, Evaluation of Nu-Way Live Oak Landfill, Remainder Parcel of Parcel Map As Per Book 186 P 79-82, Approximately 65 Acres, East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue, Irwindale, California,* dated April 27, 2010 and
> *Geotechnical Engineering Exploration Update, and Remedial Grading Recommendations, Proposed Commercial/retail Development, Nu-way Live Oak Landfill, East of 605 Freeway, West and South Live Oak Lane, and North of Live Oak Avenue, Irwindale, California,* November 23, 2010

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Report by Advanced Earth Sciences:

Technical Memorandum on Settlement Analysis, dated November, 2010

Dear Mr. Mnoian;

Irvine Geotechnical has prepared this report to summarize our geotechnical investigation of the site and to discuss the geotechnical evolution of the Nu-Way pit. Irvine Geotechnical began investigating the Nu-Way pit in 2007, originally for a buy-sell team that was trying to facilitate the sale of the property to a commercial developer, and then directly for the property owner. Apparently during the due-diligence period, the geotechnical consultant for a particular buyer discovered numerous technical and reporting issues relating to pit backfilling, which was deemed to be problematic from permitting and foundation performance standpoints. Our role evolved from third party review and consultation to include physical testing of the Nu-Way pit with the goal of obtaining permits from the Building Department to develop the property with a commercial/retail project.

Fill sites, where the fills are intended for support of structures, are subject to specific quality control measures. For the Nu-Way Pit, a 1990 Agreement between the Owner and the City of Irwindale and a 1994 Conditional Use Permit (CUP) provided specific criteria for filling of the pit. During the filling process, the 1990 Agreement and the 1994 CUP required quality control testing and reporting to ensure that the filled pit would be suitable for development. The quality control testing and reporting were to be performed by the Geotechnical Engineer of Record.

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Zeiser-Geotechnical (later became Zeiser-Kling Consultants, Inc. and referred to herein as "Zeiser") was hired in 1990 to perform a preliminary geotechnical engineering investigation of the Nu-Way pit. This included documenting and testing the engineering properties of the earth materials exposed at the bottom of the pit (approximately 120 to 130 feet below the original ground surface). By drilling borings from within the base of the pit, Zeiser sampled and tested the engineering properties of: native alluvial deposits, hydraulically placed silt and older fill associated with historical mining activities. Zeiser found that all of the deposits exposed at the bottom of the pit were suitable for supporting engineered fill. Specific recommendations were provided for placing the engineered fill so that the resulting fill would be suitable for supporting commercial buildings and infrastructure. Zeiser provided specific recommendations for: preparing the ground surface to receive fill, processing and placing the fill, disposing of oversize materials and fragments, and testing the fill to ensure quality control. The Zeiser recommendations were in conformance with the Building Code in place at the time filling began. The main Zeiser recommendations also became part of the 1994 CUP, specifically: 1) all fill should be placed to at least 90 percent of the maximum dry density and 2) oversize materials (greater than 12 inches) shall be reduced to a smaller size (crushed) or disposed of in windrows. The use of windrows was furthermore restricted in the CUP by requiring documentation as to the location and depth and prior approval by the City in writing for their use.

It is clear from the requirements and stated intent of the reports and the Agreement and the CUP, that the Nu-Way gravel pit was to be filled with engineered fill intended for support of buildings and infrastructure. The approved purpose was not a "landfill" to fill in the former pit with non-structural fill.

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For a typical grading operation that results in permits for construction, the quality control testing and reporting are essential. The CUP required geotechnical progress reports to the City at least once per year or for every 5-foot vertical fill thickness, whichever comes first. Initially, Zeiser was performing frequent site observations and testing of the grading and filling processes. The frequency of the inspections, testing and reporting became less frequent in the late 1990's to and essentially nonexistent by the early 2000's.

At the end of the filling process, the Geotechnical Engineer of Record prepares a "Final Compaction Report," which contains the results of the compaction testing. The Geotechnical Engineer of Record also prepares the Engineer's Certificate of Compliance, which states that the "fill was placed in a controlled and engineered manner and is suitable for supporting engineered structures, slabs and infrastructure." The compaction report will also contain recommendations for design of foundations and slabs as well as allowable bearing pressures.

For the Nu-Way pit, Zeiser did not prepare a Final Compaction Report, a Certificate of Compliance or certify the fill for engineering support of structures. The Zeiser reporting was inadequate and less than specified in the 1994 CUP. The daily field reports indicated that the fill was being placed in lifts that were too thick and contained too many oversize materials. It is for these reasons that a previous buyer's geotechnical engineer raised serious questions about the adequacy/ability of the Nu-Way fill to support structures.

Irvine Geotechnical was retained in 2007 to perform a geotechnical engineering exploration of the Nu-Way pit to verify whether the engineering conditions of the as-placed fill were adequate to support engineered structures. The Nu-Way pit is an inert rubble fill that contains a high percentage of oversize materials, which poses challenges to geotechnical investigations not present in compacted earth fills. Mostly, it is difficult to impossible to drill and obtain samples using conventional geotechnical sampling methods. In general, the Zeiser daily field notices (and later confirmed by the pits) indicated the test Nu Way fill was placed as a series of 3 to 12-foot thick "lifts" of rubble (heterogeneous mixture of concrete, brick, glass, metal and other inert debris), which was covered with a thin soil layer and then "compacted." Subsequent rubble lifts and soil layers were placed through time until the fill was 12 to 15 feet below the finished ground surface. A "clean" 12 to 15-foot thick, earth fill cap was placed to finish grade. The clean fill cap was compacted to at least 90 percent of the relative compaction under a new geotechnical consultant, Hushmand and Associates.

C4P compliant? *

In a conventional compacted earth fill, in-situ soil samples can easily be obtained and tested in the laboratory for percent compaction, strength or consolidation potential. Since the rubble debris is mostly oversize concrete fragments larger than 8 inches in diameter, "in-situ" samples cannot be obtained. Furthermore, typical geotechnical drilling and testing devices such as: bucket augers, hollow-stem augers, mud-rotary or CPT rigs cannot be used. Large scale bulk-density tests were considered the only method for determining the in place density and compaction of the debris fill. However, this method is not economically practical to depths greater than 50 to 70 feet due to the large open excavations required and the volumes of earth moved. We concluded early on that it was not going to be possible to measure the relative compaction of most of the fill placed in the Nu-Way pit.

Our Phase I geotechnical investigation was intended to use indirect methods (blow counts and geophysical surveys) to determine the extent and depth of the fill and to try different techniques to measure the engineering properties of the fill. Becker-Hammer borings were chosen because it was believed the rig was strong enough to penetrate the fill and that "blow counts per foot" could be correlated to conventional engineering properties. Becker-Hammer borings and drilling techniques were developed to sample and test sand and gravel deposits. Research, mostly from Canada, has correlated Becker-Hammer blow counts to SPT blow counts and then to engineering properties of "sand." Research and case histories of Becker-Hammer borings used in rubble fills was not found except for a summary report prepared by the City of Irwindale Landfill Committee. It was hoped that a Becker-Hammer "signature" from a controlled fill at the Vulcan pit in northern Irwindale could be used to compare Becker-Hammer "signatures" from the Nu-Way pit.

In addition to Becker-Hammer borings, geophysical testing was employed to estimate the physical properties of the Nu-Way fill. Seismic reflection lines were intended to image the base of the fill. Downhole shear wave velocity profiling was intended to measure the shear wave velocities for use in seismic modeling. Surface wave testing was intended to determine the fill stratigraphy, uniformity and average shear wave velocities. It was hoped that average bulk densities of the fill and/or loose zones and large voids could be quantified from the shear wave velocity testing.

Our initial Phase I findings and interpretations were optimistic. The Becker-Hammer blow count data seemed to become higher (interpreted as more compact) at depths of 40 to 50 feet. The shear and compression wave velocity data generated from the geophysical testing was much higher than assumed. Based upon our indirect testing of the rubble fill, it appeared that the fill deposit was generally good below 40 to 50 feet. Removal and recompaction of the upper 40 to 50 feet of fill was opined as a likely method to create building pads for support of the proposed development.

Our Phase II testing program intended to expose and directly test the upper 50 feet of the in-situ fill. Two large test pits were excavated through existing inert debris fill between August 4, 2008 and October 27, 2008. The first objective of the test pits was to expose wide and deep sections of the pit to observe the fill quality, including lift thicknesses, the presence of voids and nesting of oversize debris. The second objective was to perform bulk density and gradation testing of the in-situ fill materials.

Both test pits were planned to extend to 50 feet below the ground surface to perform the relative compaction, bulk density and gradation testing. Because inferior debris fill was found to the total depths, the deepest portions of the pits were deepened an additional 25 ± feet (to 75 feet) in October of 2008. The deeper portions of the test pits were not considered safe to enter to perform in-place density testing, but were geologically logged and photographed.

Eight bulk density tests were performed at depths of 10 to 50 feet below the ground surface. Four of the tests had a relative compaction greater than 90 percent and four were less than 90 percent. The compaction standard for the Nu-Way Pit was that all fill was to be compacted to at least 90 percent of the maximum density. Testing of the matrix soil within the interstices (soil infilling between the rubble fragments) of the rubble revealed even lower relative compaction results (73 to 94 percent relative compaction with only 1 of the 8 tests greater than 90 percent).

It was clear from the two deep test trenches that the upper 70 to 75 feet of the fill is of variable quality. Large voids, nesting of oversize materials, thick lifts, and lack of processing were ubiquitous. In addition to personnel from Irvine Geotechnical, the pits were logged and/or observed by personnel of Hushmand and Associates, the City of Irwindale, Geo-Logic

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(geotechnical consultant to the City of Irwindale), Dr. Jonathon Bray (Professor at UC Berkeley and consultant to the City of Irwindale), and members of the Irwindale Landfill Committee. There was a consensus that the fill exposed in the pits was not properly processed and compacted to support a commercial development. It was also clear that the fill was not consistent with the 1994 CUP.

Another conclusion of the Phase II exploration was that the high shear wave velocities measured in the fill as part of the Phase I testing did not correlate with relative compaction. It is clear that the high percentage of oversize concrete fragments is masking the presence of low velocity voids and dictating the shear wave velocity.

The fill exposed in the lower portions of the pits contained less oversized materials and concrete with reinforcing bars. It was considered feasible to drill a large diameter boring from within the lower portion of Test Pit 2. The findings of our Phase III investigation are that processed and better compacted fill is present at a depth of 92 to 95 feet below grade (elevation 310 to 313 feet). The depth and elevation correlate well with the historical field testing and documentation by Zeiser and a time horizon of the late 1990's.

Detailed descriptions of the testing and engineering analysis contained in our Phase I through Phase III investigations are contained in our November 23, 2010 update report. In addition to the technical reports, Irvine Geotechnical has documented the condition of the fill through photographs and videos. All available media has been copied and made available in an electronic format.

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The main technical issue for the poorly processed and compacted fill is settlement, especially dynamic (earthquake) settlement. The potential of an earth material to settle under gravity, weight of structures, fluctuations in groundwater levels and strong shaking is represented by the void ratio (ratio of the volume of voids to the volume of solids). Depending on the distribution and size of the constituents and level of compaction, every material has a "minimum" void ratio. Thus, the settlement potential is represented by the void ratio of a deposit in the current condition relative to the minimum void ratio. The purpose of the original Zeiser recommendations and the conditions of the CUP for processing and compacting the rubble materials was to create a fill that would be suitable for future development. Advanced Earth Sciences (AES) was hired by the owner to model settlement potential of the site.

AES concluded that the settlement potential of the existing, as-placed condition of the fill was 5 to more than 14 inches. This amount of settlement exceeds State and County standards and is not acceptable for supporting buildings, slabs and utilities. Uniform settlement can be accommodated through design and siting. Differential settlements are very damaging to structures and infrastructures. The amount of differential settlement that is acceptable from a building code standpoint is generally 1 inch in 30 feet, which has been the standard for at least 30 years. Modeling performed by AES showed that by partially removing and replacing properly processed and compacted fill over the existing fill decreases the total and differential settlements. According to modeling performed by AES, a properly processed and compacted 70–foot thick cap decreases the differential settlement to 1 inch in 30 feet.

Based on our Phase I, II and III explorations and analyses by AES of the settlement potential, the Nu-Way fill is not suitable for supporting structures, slabs and infrastructure that would be part of a commercial/industrial development. Our November, 2010 report, which has been submitted to the Count y Los Angeles Geotechnical and Materials Engineering Division (GMED), shows that removing, processing and recompacting the upper 70 feet of fill materials will result in a suitable building site. The resulting fill and building sites will conform to Building Code standards with respect to performance and settlement and considered "CUP compliant." Peer review and comments from GMED are pending. It is possible that the ultimate removals will be made deeper depending on GMED's comments and requirements.

It is also the finding of Irvine Geotechnical that if the conditions of the 1990 Agreement and the 1994 CUP had been complied with, fill placed in the Nu-Way pit would have been suitable for supporting commercial/industrial development with no additional mitigation or special foundation design.

The following sections may be added depending on peer-review comments from the County of Los Angeles, Geotechnical and Materials Engineering Division and/or to rebut opinions from experts working for Waste-Management.

Comments/Additional Analysis - County of Los Angeles Technical Review of 11/23/2011

Rebuttals/Comments - Opinions from Plaintiffs' Experts

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Irvine Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this or the referenced and appended reports should be directed to the undersigned.

Respectfully submitted,
Irvine Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
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IRVINE GEOTECHNICAL Inc

GEOTECHNICAL ENGINEERING EXPLORATION UPDATE

AND REMEDIAL GRADING RECOMMENDATIONS

PROPOSED COMMERCIAL/RETAIL DEVELOPMENT

NU-WAY LIVE OAK LANDFILL

EAST OF 605 FREEWAY, WEST AND SOUTH LIVE OAK LANE

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NOVEMBER 23, 2010

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INTRODUCTION

This report has been prepared per our agreement and summarizes findings of Irvine Geotechnical's geotechnical engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution and engineering properties of the earth materials underlying the site to develop remedial grading recommendations for preparing the site for a mixed-use retail and warehouse-store building development.

INTENT

It is the intent of this report to assist in the design and completion of the proposed project. The recommendations are intended to reduce geotechnical risks affecting the project. The professional opinions and advice presented in this report are based upon commonly accepted standards and are subject to the general conditions described in the NOTICE section of this report.

PROPOSED PROJECT

Information concerning the proposed project was provided by the client. Formal plans have not been prepared and await the conclusions and recommendations of this report. Conceptually, it is planned to develop the property as a mixed-use retail and warehouse store development. The design concept envisions one or two large warehouse stores surrounded by small retail and restaurant buildings and parking lots. The structures would be planned at or near existing grade. Structural loads are anticipated to be light to moderate. Remedial grading will be employed to create building pads suitable for the proposed buildings, parking lots and infrastructure.

RESEARCH - PREVIOUS WORK

Irvine Geotechnical first started working on this project in March of 2007. Work performed to date by Irvine Geotechnical and its subcontractors have included researching public and private records, reviewing historical topographic maps and aerial photographs, performing subsurface exploration, performing geophysical testing and performing field and laboratory testing of the earth materials. The results of our historical work are contained in the following reports:

> *Phase I - Geotechnical Engineering Exploration, Nu-way Live Oak Landfill, East of 605 Freeway, West and South Live Oak Lane, And North of Live Oak Avenue, Irwindale, California,* dated March 31, 2008;
>
> *Phase II - Geotechnical Engineering Exploration, Evaluation of Nu-Way Live Oak Landfill, Remainder Parcel of Parcel Map As Per Book 186 P 79-82, Approximately 65 Acres, East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue, Irwindale, California,* dated December 8, 2008; and
>
> *Phase III - Geotechnical Engineering Exploration, Evaluation of Nu-Way Live Oak Landfill, Remainder Parcel of Parcel Map As Per Book 186 P 79-82, Approximately 65 Acres, East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue, Irwindale, California,* dated April 27, 2010.

Copies of the Phases I through III reports are included in **Appendix IV, Historical Reports By Irvine Geotechnical.** Generally, the Phase I report contains the results of research, borings, and geophysical studies. Phase II contains the results of deep test trenches and bulk density testing and Phase III presents information from a deep boring within the westerly test trench.

As part of our investigation, records and documents on file at the City of Irwindale and provided by the client were reviewed. The documents were scanned into an Adobe Acrobat (PDF) format and indexed for quick reference. It is our understanding that a copy of the scanned documents has been provided to the City of Irwindale on a Compact Disk.

Most of the documents were generated after 1991 by Zeiser-Kling and Associates and were associated with geotechnical observations and testing during backfilling of the Nuway gravel quarry. Historical topographic maps and aerial photographs before, during and after mining and filling were reviewed. Most of the historical photographs were provided by Kent McMillan, who had been reviewing the mining and filling histories at the Nu-Way and nearby United Rock

Products site for the City of Irwindale. Some of the photographs were scanned and scaled to match topographic maps and property boundaries to facilitate interpretation. Mr. McMillan also provided a hydrograph that includes yearly groundwater elevations at the site extending back to 1932. The hydrograph was used to estimate the elevation of groundwater "lakes" visible in some of the air photos.

EXPLORATION

The site was explored by Irvine Geotechnical in three Phases, I through III between October 15, 2007 and September 10, 2009 and included performing two seismic reflection line surveys, advancing six Becker Hammer borings, performing one downhole seismic shear wave survey, performing an active/passive surface wave survey, excavating two large test trenches and drilling one large diameter boring. The locations of the borings, trenches and geophysical lines are shown in the Geologic Map. Subsurface distribution of the earth materials, projected geologic structure, and the proposed project are shown on Sections A through C.

Becker Hammer Borings

The Becker Hammer borings were advanced by Great Western Drilling of Fontana using a AP 1000 Becker Hammer rig. Between October 15 and 24, 2007 borings were advanced to depths of 120 to 190 feet using the "closed bit" method. Blow counts per foot and diesel combustion chamber pressure were recorded by staff of Irvine Geotechnical. The Becker Hammer borings are graphically logged on the enclosed Log of Borings. Because of the nature of the soils encountered in the borings, in-situ samples of the fill and alluvium were not obtained in this phase.

The locations of the borings are shown on the Geologic Map. Borings 2 and 3 were terminated just below the 1991, "pre landfill backfilling surface" and settlement monuments were installed. Boring 6 was advanced to near the base of the fill and a settlement monument was installed at a depth of 149 feet. A multi-stage gas vapor well was also installed in Boring 6 under the direction of Environmental Applications for future monitoring. A solid 3" diameter PVC casing was installed to the total depth of Boring 4, with the annular space filled with clean medium sand. The boring was used for the downhole shear wave survey.

The settlement monuments consist of 1 inch diameter, steel pipes that are connected by threaded pipe couplings. The base of the monument is secured in five feet of cement. C & M Duraflex, PVC centralizers were used to keep the pipe within the center of the boring. The centralizers were spaced about 12 to 15 feet apart from top to bottom. The lower 50 feet of the annular space was filled with clean, medium sand. The remainder of the annular space was filled with bentonite pellets up to the ground surface. The monuments were constructed from within the drill stem of the Becker Hammer rig and backfilled as the stem was removed to ensure that caving did not occur. The tops of the monuments are protected by 12 inch thick, cast-in-place concrete pads. Four inch diameter PVC sleeves extend through the pads to ensure that the pads can move (settle) independently of the monument pipes.

Geophysical - Seismic Reflection

The seismic reflection lines and downhole shear wave survey were performed by Terra Physics, with interpretation assistance from Wilson Geosciences. The locations of the seismic reflection profiles are shown on the Geologic Map. The PVC casing installed in Boring 4 was used for the downhole shear wave survey. The procedures and results of the seismic reflection and downhole survey are contained in the Terra Physics report, "*Seismic Reflection and Borehole Seismic Velocity Surveys to Delineate Subsurface Backfill Material and Underlying Native Soil*

Boundaries Nu-way Reclaimed Aggregate Mine Landfill - Irwindale, California," which is appended to our Phase I report. Additional interpretation of the geophysical study and correlation between borings and geologic stratigraphy was performed by Wilson Geosciences (*Technical Report: Seismic Reflection and Borehole Seismic Investigation: Nu-way Live Oak Landfill Reclamation, Northeast of the Interstate 605 Freeway and Live Oak Avenue Intersection, Irwindale, California*) and is appended to our Phase I report.

Geophysical - Shear Wave Velocity Profiles

GeoVision performed surface wave soundings and created shear wave velocity profiles through three areas of the landfill. The results of the surface wave study are contained in the GeoVision report, "*Geotechnical Investigation, Nu-Way Reclaimed Aggregate Mine, Irwindale, California,"* dated February 25, 2008. The surface wave study included collecting 1-D surface wave soundings at 230 foot intervals along three profiles totaling about 3,700 linear feet. The purpose of the surface wave soundings is to provide 2-D shear wave velocity models of the upper 200 to 300 feet along each profile. Both active (spectral analysis of surface waves [SASW] or multi-channel analysis of surface waves [MASW]) and passive (array or ReMi) were used. The active techniques were able to image the S-wave velocity of the upper 100 to 130 feet, while passive techniques extended the depth of investigation past 300 feet. The 2-D shear wave profiles are in the shape of a large triangle as shown on the Geologic Map.

Large Test Pits

Two large test pits were excavated through existing inert debris fill between August 4, 2008 and October 27, 2008. The first objective of the test pits was to expose wide and deep sections of the pit to observe the fill quality, including lift thicknesses, the presence of voids and nesting of oversize debris. The second objective was to perform bulk density and gradation testing of the in-situ fill materials.

The northerly pit, (Test Pit 1) was chosen to coincide with Boring 5 and the intersection of Shear Wave Velocity Profiles A and B. The westerly pit (Test Pit 2) was chosen to coincide with Boring 3 and the intersection of Shear Wave Velocity Profiles B and C. Test Pits 1 and 2 are shown on the Geologic Map. The corners of the pits were located by Geo-Logic Associates using a Trimble GeoXH 2005 series GPS receiver, which is a mapping grade GPS device that is generally accurate to within 12 inches.

Both pits were planned to extend to 50 feet below the ground surface, and were then ultimately deepened to 70 to 75 feet. The initial footprint of the pits was based upon a 50-foot high, 1:1 slope on 3 sides of the pit and an entry ramp to four benches. Four level benches (benches 1 - 4) were created in both pits at approximately 12 foot vertical increments. The approximate corners of the benches and elevations were determined by Geo-Logic using the GPS mapping device and plotted onto the Geologic Map. The benches and elevations are shown on the Test Pits Map appended to this report.

Excavation of the trenches began on August 4, 2008 using excavators, dozers and loaders. Fill soils removed from the pits were segregated by depth and stockpiled outside of the pits. The stockpiles were later used as source material to create Maximum Achievable Density (MAD) test pads. Excavation and testing of the test pits and 8 benches had been completed by September 3, 2008.

The deepest portions of the pits were deepened an additional 25 ± feet between October 21 through 27, 2008 with an excavator. The deeper portions of the test pits were not considered safe to enter to perform physical testing. The deeper portions of the test pits were logged, photographed and videotaped by the engineering geologist and soils engineer. Profiles through the test pits are shown on Section A and the shear wave Velocity Profile plates.

Excavation of the trenches and stockpiling were performed under the observation of the project engineer and geologist, who also photo-documented and videotaped the process. Periodic observations of the excavation process were also performed by personnel of Hushmand and Associates and the City of Irwindale.

Large Diameter Boring

Between September 1 and 10, 2009 a three -foot diameter boring was drilled to a depth of 70 feet below the drill pad (to approximate elevation 300) from within Test Pit 2 (westerly of the two pits). The boring is situated along the downhill side of Bench 3 and was drilled from an elevation of approximately 370 feet. Because of abundant rebar, large concrete fragments and caving conditions, drilling was difficult. The geologist was onsite during the drilling to log the drilling spoils as they were removed from the boring. Depths were determined using a weighted tape measure. The completed boring was deemed unsafe for manual downhole logging. The boring was video-logged on September 10, 2009.

A description of the earth materials encountered in the boring is contained on the Log of Boring. The location of the boring is shown on the Boring Map.

SITE DESCRIPTION - HISTORY

The study area is located in the western portion of the City of Irwindale, California (117.976W; 34.110N) and consists of approximately 65 acres of a mostly level, former gravel pit that has been filled and is known as the "Nu-Way Live Oak Landfill." The Nu-Way site is located just south of Arrow Highway, southeast of the Santa Fe Flood Control Basin, west of the San Gabriel River channel and north of Live Oak Avenue. Nu-Way is bounded by the 605 Freeway on the west and Live Oak Lane and industrial properties on the east and north, respectively. Elevations range from about 408 to 410 feet along the eastern portions of the property to 375 feet in a basin along the western edge of the pad and within the Southern California Edison easement.

Geomorphically, this area of Irwindale is characterized as a gently, south-southwest sloping alluvial fan that emanates from San Gabriel Canyon. The alluvial fan is comprised of sand and gravel deposits that have been historically mined for construction aggregate. The depositional source has been blocked by the Santa Fe Flood Control Basin.

Mining within the study area started in the late 1950's by the Owl Rock Company. Owl Rock did not own the entire Nu-Way site and the boundary between the Owl Rock (east) and Blue Diamond (west) properties trended north-south and nearly bisected the study area. The majority of the Blue Diamond property extended westerly, beyond what was to become the 605 Freeway, to near the intersection of Arrow Highway and Live Oak Avenue. Mining was performed solely on the Owl Rock property (eastern portion of Nu-Way) until 1962. In 1962, mining commenced in the western portion of the Nu-Way pit, with material moved by conveyor belts westerly toward the Blue Diamond processing area. The Owl Rock and Blue Diamond properties were mined independently until the mid-1960's, when the pits merged.

From 1957 to the mid-1960's, waste material (silt) from the Owl Rock operation appears to have been disposed of offsite and north of the limits of the Nu-Way pit. Waste material generated from mining on the Nu-Way site by Blue Diamond appears to have been disposed of in silt ponds near the intersection of Arrow Highway and Live Oak Avenue. After the mid-1960's, all waste materials from mining appear to have been disposed of within and north of the Nu-Way site. The 605 Freeway had been graded in 1970, formally separating the Blue Diamond and Owl Rock properties. However, access beneath the freeway between the two pits was preserved. In the mid-1970's, permission was granted to dispose of liquid waste material (silt) within the Nu-Way pit. Mining within the Nu-Way site was mostly complete by the mid to late-1980's. A Liquid Waste Permit was obtained from the Water Quality Control Board in 1985 to allow the placement of imported silt in the pit. The "waste" silt was reportedly dredged from active pits on the west side of the 605 Freeway and transported as a slurry beneath the freeway and placed into the pit, predominantly in the southern portion of the study area. Reportedly, 800,000 cubic yards of silt slurry was accepted at the site, mostly within the southwestern half.

The waste permit was amended in 1990 to include inert materials such as: concrete, bricks, rocks, asphalt, ceramics, sand and non-contaminated soils. Drywall was originally accepted within the landfill and later rejected; although, dry wall recycling and processing was apparently conducted onsite until a much later date. A geotechnical study was performed by Zeiser Geotechnical (later became Zeiser-Kling Consultants, Inc. and referred to herein as "Zeiser") to provide recommendations for placing and compacting fill into the pit. Zeiser reported 5 to 40 feet of existing fill throughout the base of the pit, which was around elevation 280 to 285 feet (120 to 130 feet below existing grade). Borings were not drilled in the southern portion of the "silt pond," which was present in the southwestern corner of the pit. The locations of the Zeiser 1991 borings are shown on the Geologic Map and copies of their boring logs are appended to this report. Zeiser provided specific recommendations for the placement of engineered fill and measures to ensure quality control.

Along with active filling, additional mining was performed in the early 1990's. Primarily, the mining consisted of "pushing" the slopes toward Live Oak Lane, Live Oak Avenue and the 605 Freeway. Starting at the pit boundaries, slopes were trimmed down at a 1:1 (horizontal:vertical) or steeper gradient. The lower 10 to 20 feet of the trim was made vertically.

Backfilling of the pit with inert debris fill was to comply initially with an Agreement dated January 25, 1990 between the City of Irwindale and Nu-Way Industries, Inc. (Quarry Rehabilitation Plan) and later with the Conditional Use Permit (CUP) dated December 15, 1994 granted by the City of Irwindale to Sanifill (the property owner). The key fill placement requirements specified in these documents called for the following:

- Provide, place and compact to 90 percent density clean earth and inorganic solid fill materials (e..g, broken concrete and A.C.). No organic materials will be imported to the site.

- The oversize materials (greater than 12 inches in size) shall be either crushed or placed in windrows in accordance with the standard windrow detail provided by Zeiser (this is L.A. County's Standard Windrow Detail in their Grading Code). The operations shall be such that nesting of oversize material does not occur and that the oversize material is completely surrounded by compacted and densified fill. The locations, materials and disposal methods for oversize had to be approved by the City.

- Owner/Applicant shall submit to the City the geotechnical progress reports at least once per year or for every 5 feet vertical fill thickness, whichever comes first.

Between 1991 and 2005, Zeiser performed periodic geotechnical observations and testing. The results of the compaction testing and a description of the grading observed at the time of the site inspections are contained in numerous Zeiser field notices and file documents, which are contained on the CD.

The records indicate that Hushmand and Associates replaced Zeiser-Kling as the geotechnical engineer of record in 2005 for the placement of the "clean" compacted fill cap. Fill placed from

2005 to present was performed under the geotechnical supervision of Hushmand. The results of compaction testing by Hushmand are contained in their report, *Construction Quality Assurance Services Nu-Way Live-Oak Landfill, Irwindale, Los Angeles County, California,* dated March, 2007.

Groundwater lakes are visible on the historical photographs. It is our understanding that active mining operations would extend to and stop at the groundwater table. Relative low groundwater years would allow for deeper mining. The limits of the groundwater lakes, combined with the hydrograph data, was used to estimate the approximate maximum depths of mining. During the mining period, relative groundwater elevation lows occurred in 1964-1965 (210 feet) and 1978 (205 feet). Groundwater was encountered in this exploration in Boring 4 at an elevation of about 242 feet (165 feet below ground surface).

A comprehensive analysis of the Zeiser inspections and the filling operations was also performed by Advanced Earth Sciences (AES) and their findings are discussed in detail in their Technical Memorandum included in Appendix I. The key highlights of backfilling history including fill placement methods and field inspection and testing frequencies are presented below. results appended to this report.

1. In general, the inert debris fill consisted primarily of concrete with abundant rebar, floor tile, cement and asphalt shingles, bricks, soil and crushed glass. In the early stages of fill placement drywall was reportedly accepted as backfill material and the drywall areas were moisture conditioned to break up the material and mix with the soil.

2. The rubble-soil mixture ratio reportedly varied through the pit backfilling history depending upon composition of incoming loads. Observations of the two large test pits revealed presence of significant (approximately 15 to 20 percent) oversize, larger than 12 inches. Of the

oversize fraction, steel-reinforced concrete fragments, foundation elements, columns and other construction demolition debris with dimensions ranging from 24 inches to more than 20 feet were present in the fill.

3. During the earlier parts of backfill placement, a mobile crusher was reportedly used to crush material larger than 12 inches prior to placement in the fill. However, the regularity of crusher operations was not well documented. The Zeiser daily field reports indicate that the crusher was working during most of their field visits through 1990s. Beyond late 1990s, crusher was reportedly inoperable.

4. The patterns of fill placement and lift thicknesses did not follow the CUP requirements. The fill lifts were reportedly 3 to 4 feet thick during earlier stages of fill placement and were routinely in excess of 5 feet during the late 1990s through 2005. Also, the oversize material and thick lifts were placed in "blanket type" pattern, with 5- to 8-foot thick lift of debris fill placed by end dumping and topped with a 6- to 12-inch thick soil layer/blanket. The testing performed by Zeiser was always in the soil fill material or matrix, either within the soil blanket or in the bulk fill layer where the soil component was significant. On numerous occasions, the Zeiser daily reports indicate that the fill was not suitable for testing, i.e., oversize fraction was too excessive for any of the conventional field density test methods (nuclear or sand cone) to be of any meaningful value.

5. The lift thicknesses from late 1990s through 2005 were frequently greater than 7 to 8 feet and the oversize material was never placed in windrows, as called for in the CUP. A typical description of rubble fill and soil blanket layers, as provided by Geo Logic Associates (GLA) in their log of the test excavations to a depth of about 65 feet, is provided in AES' Technical Memorandum.

6. Because of the large pieces of concrete/rubble and placement in thick layers, "nesting" of large fragments and presence of voids in the fill were a common phenomenon. These have frequently been reported in Zeiser field reports and were observed in the test trenches excavated during Phase 2 investigation. The CUP had clearly specified that oversize material be either crushed or placed in windrows, surrounded by soil and that the voids between the oversize fragments be filled with granular material and densified by flooding. This was not followed in actual filling practice.

Fill Inspection/Testing Frequencies

1. Between 1991 and 1993, the average frequency of Zeiser field inspection visits was about once per week and the fill density test frequency varied from 1 test per 4,300 cu. yds. in 1991 to about 1 test per 2,600 cu. yds. in 1992 and 1993. The lower frequency of testing per fill volume in 1991 may be related to a large volume of initial filling comprising thick rock/debris blanket over the existing uncertified fill (including saturated silts) to stabilize the pit bottom. This blanket fill apparently did not have to be and could not be tested.

2. In 1994, the frequency of Zeiser field visits averaged about 1 visit every 2 weeks. However, testing frequency by volume remained consistent as in 1992 and 1993 and averaged approximately 1 test per 2,800 cu. yds.

3. From 1995 through 1998, the frequency of field visits decreased to an average of 1 visit per month and fill testing frequency progressively decreased, ranging from 1 test per 5,800 cu. yds. in 1995 to about 1 test per 9,500 cu. yds. in 1996 and 1997, and to about 1 test per 14,000 cu. yds. in 1998. The frequency by number of inspection visits ranged from about 1 visit for every 25,000 cu. yds. in 1995 to about 1 visit per 55,000 cu. yds. in 1998.

4. Between 1999 and 2002, the frequency of visits and testing reduced to a level that would be characterized as almost "no supervision/oversight." There were 4 visits in 1999, representing 1 visit per approximately 175,000 cu. yds., 1 visit in Year 2000 when over 700,000 cu. yds. of fill was placed, 5 visits in 2001 representing 1 visit for approximately 156,000 cu. yds. of fill, and 6 visits in 2002 representing 1 visit per approximately 100,000 cu. yds. of fill placement.

5. In 2003 and 2004, the frequency of visits average 1 per month to 1 every 2 months representing 50,000 to 100,000 cu. yds. of fill placed between visits.

6. In 2005, visits increased to an average of 1 visit per week representing 1 visit every 25,000 cu. yds. However, excessive fill lift thicknesses ranging between 8 feet and 12 feet were still consistently reported throughout the year.

7. In 2006, much of the fill placement activity was for the upper 10- to 15-foot thick soil cap that received full-time supervision and testing by HAI with an average test frequency of 1 test per 1,400 cu. yds.

LIMITS AND THICKNESS OF FILL

The thickness and distribution of earth materials within and around the Nuway pit was accomplished using: the seismic reflection surveys; Becker-Hammer borings; Zeiser-Kling's 1991 borings; historical topographic maps; historical photographs and home videos; and the surface wave profiling. The interpreted geologic profile across three areas of the Nu-Way pit are shown on Sections A, B, and C. The maximum depth of fill appears to be around 180 to 185 feet below ground surface as shown on the cross sections.

The March 1984 topographic survey by Hekimian - Van Dorpe Associates, which was used by Zeiser-Kling as the basis for their 1991 Geologic Map, was assumed to roughly represent the pre-controlled fill conditions in the pit. Fill deposits (silt waste, soil and rubble) and with elevations lower than the contours shown on the Zeiser-Kling Geologic Map, are certainly "uncontrolled fill" or "unpermitted fill." It should be noted that this topographic map, predates the 800,000 cubic yards of silt that were accepted at the site in 1987/1988. Also, based upon aerial photographs, some additional mining (on the south) and "dumping" of fill (on the north) appears to have been on going up until 1990. The Zeiser boring logs contain elevations of the top of the boring. It is presumed that the elevations for the top of borings were checked against a known datum elevation.

Home videos between 1990 and 1993 show significant slope trims along the eastern, southern, and western margins of the pit. Processing and exporting of aggregate were occurring in early stages of controlled filling. Slope trims shown on Section B and C were estimated based upon the video evidence.

The aerial photos that showed "groundwater lakes" were used in determining limits and minimum depths of mining. The approximate elevations of the lakes were estimated from the hydrograph. The limits of the lake with a corresponding elevation were then plotted onto the base topographic survey, with the composite used to define the minimum depths and extent of mining.

Surface wave soundings and resulting shear wave velocity models found a velocity inversion beneath the central to north-central portion of the pit. The thickness and distribution of the lower velocity layer appears to correlate with the lower, "uncontrolled fill" found by Zeiser. The contrasts between the deeper low-velocity and shallow high-velocity materials are enhanced by normalizing the shear wave velocities to a constant over-burden.

ENGINEERING PROPERTIES OF THE FILL

The engineering properties of the earth materials were determined both in the field by personnel of Irvine Geotechnical and in the soils laboratory of Soil Labworks LLC.

Bulk Density Tests

One Bulk Density (BD) test was performed on each of the benches exposed in the two deep test pits, with the approximate locations shown on the Test Pits plate and the Geologic Map. By design, no testing was performed of clean, compacted fill cap that overlies the rubble fill. The tests were performed in conformance with ASTM 5030-04 and the City of Irwindale guidelines (Guidelines for Above-Water Backfilling of Open-Pit Mines, Irwindale, California in Technical Guidelines for Open-Pit Mines, City of Irwindale, 2005). A large, ¾-inch thick steel plate with a 6-foot diameter hole in the center, was used as a template. It was decided to level the template on the bench in lieu of providing a raised lip because of cost and timing constraints. The bulk density test holes were excavated to depths of 4 to 5 feet using a small backhoe and hand labor. Material excavated from the BD tests was transferred to a roll-off bin via a loader. Care was taken to minimize the loss of soil via spillage by using tarps and using care. Re-bar and other non-soil and concrete debris protruding into the BD test holes were cut flush with the sides of the hole. The holes were excavated as close to vertical as possible and cleaned by hand. The bottom of the excavation was also cleaned to a smooth surface by hand. Upon completion, the BD test hole was photographed, measured and logged by the project engineering geologist.

Earth materials from the BD density test holes were stored in the roll-off bins and covered with tarps to prevent moisture and fines loss due to evaporation and wind. The moisture content of the in-situ soils removed from the BD test was also measured and recorded. The bins were carried by truck to the Nu-Way recycling center in Monrovia for weighing and then transferred back to the site for Bulk Gradation testing.

The volumes of the BD tests were determined using "water-replacement" by accurately measuring the volume of water required to exactly fill the BD test pit to the bottom edge of the template. An inline water meter (Blue-White Industries RT-200MI-GPM3) connected to a water truck via a fire hose was used to fill the BD test pits. The meter is accurate with flow rates between 10 and 100 gpm and was calibrated on August 1, 2008. PVC pipe transitions were placed on either side if the in-line valve to ensure laminar flow past the venturi.

Prior to filling the pits with water, two layers of visqueen were placed to form a water-tight container. Two layers were considered necessary due to the sharp concrete, glass and re-bar debris exposed in the sides and bottoms of the BD pits. From within the hole, the engineer verified that the plastic liner was in firm contact with the underlying soils and not stretched across voids.

Upon filling the hole and metering of the volume, the liners were perforated and removed. The time required for the holes to drain were recorded by the staff engineer and project geologist. The measured volume was also compared to the mathematical volume based upon the actual dimensions of the BD test pits.

The moist bulk density is the ratio of the weight removed from the pit to the volume of the pit. The dry bulk density is the corrected weight after subtracting the water content from the fraction of the mass finer than ¾ inch. Material coarser than ¾ inch consists of steel, brick and concrete fragments, which generally do not contain appreciable moisture. The moisture content relative to the dry density was plotted on the Moisture-Density Relationship chart.

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Using soil and inert debris retrieved from the pits, Maximum Achievable Density (MAD) pads were created following Irwindale Guidelines. Bulk density tests and gradation tests were also performed in the MAD pads. A clustering of dry densities near 133.0 pcf with a moisture content of 11 percent for the MAD pads is believed to represent the maximum dry density.

Bulk Gradations

All of the material from the BD tests were sieved to determine the distribution of material sizes. A rack containing screen sizes 12x12, 8x8, 6x3, 3x3, and 1x1 inch grids was manufactured by the client and placed near the entrance to the Nu-Way pit. Materials collected on the screens

were sorted in bins and weighed. The sorting and weighing were performed in a paved portion of the site, which facilitated weighing of bins and large samples and in controlling spillage. A representative sample of the materials passing the 1 inch sieve was transferred to the soils laboratory to determine the additional fractions through the sand-size range and the percentage of fines (percent passing the #200 sieve). The gradations and weighing were performed by the project geologist and staff engineer of Irvine Geotechnical. The results of the gradation testing are shown on the Grain Size Distribution graphs.

The following table summarizes the results of the bulk density testing. Refer to the Geologic Map and Test Pit Map plate for the locations of the benches, depths and individual tests.

SUMMARY OF BULK DENSITY TESTING						
SAMPLE	TYPE	WET DENSITY (PCF)	DRY DENSITY (PCF)	VOID RATIO (%)	MAX. DENSITY (PCF)	RELATIVE COMPACTION (%)
TP1 - Bench 1	Bulk Density - In situ	79.7	57.0	1.100	133.0	42.9
TP1 - Bench 2	Bulk Density - In situ	133.2	125.6	0.280	133.0	94.4
TP1 - Bench 2	Bulk Density - Test Pad	142.0	131.5	0.179	133.0	98.9
TP1 - Bench 3	Bulk Density - In situ	114.9	109.6	0.473	133.0	82.4
TP1 - Bench 3	Bulk Density - Test Pad	142.6	132.4	0.249	133.0	99.5
TP1 - Bench 4	Bulk Density - In situ	91.0	86.1	1.100	133.0	64.7
TP1 - Bench 4	Bulk Density - Test Pad	142.1	132.5	0.181	133.0	99.6
TP2 - Bench 1	Bulk Density - In situ	127.8	121.1	0.334	133.0	91.1
TP2 - Bench 2	Bulk Density - In situ	133.6	126.4	0.260	133.0	95.0
TP2 - Bench 2	Bulk Density - Test Pad	139.6	129.7	0.211	133.0	97.5
TP2 - Bench 3	Bulk Density - In situ	129.2	120.2	0.319	133.0	90.4
TP2 - Bench 3	Bulk Density - Test Pad	137.1	127.3	0.235	133.0	95.7
TP2 - Bench 4	Bulk Density - In situ	126.5	117.3	0.329	133.0	88.2
TP2 - Bench 4	Bulk Density - Test Pad	136.1	126.5	0.239	133.0	95.1
TP2 - Bench 4	Bulk Density - Test Pad2	135.6	127.3	0.250	133.0	95.7

Sand Cone Density Tests

A sand cone conforming to ASTM 1556 was used to determine the in-situ moisture and density of the soil exposed at the surface elevation of each of the benches. Bulk samples of the soils at the locations of the soil samples were also obtained and transferred to the soils laboratory for maximum density testing (ASTM 1557). The sand cone tests were performed by the soils technician.

The results of the sand cone testing are shown on the following table. Refer to the Test Pits plate for the locations of the individual tests.

SUMMARY OF SAND CONE DENSITY TESTING						
SAMPLE	TYPE	WET DENSITY (PCF)	DRY DENSITY (PCF)	VOID RATIO (%)	MAX. DENSITY (PCF)	RELATIVE COMPACTION (%)
TP1 - Bench 1	Sand Cone - In situ	119.4	116.1	---	123.5	94.0
TP1 - Bench 2	Sand Cone - In situ	105.9	96.7	---	122.5	78.9
TP1 - Bench 3	Sand Cone - In situ	112.3	102.4	---	115.5	88.7
TP1 - Bench 4	Sand Cone - In situ	98.9	90.6	---	125.0	72.5
TP2 - Bench 1	Sand Cone - In situ	116.4	107.5	---	129.5	83.0
TP2 - Bench 2	Sand Cone - In situ	107.2	98.1	---	130.5	75.2
TP2 - Bench 3	Sand Cone - In situ	109.9	97.4	---	132.0	73.8
TP2 - Bench 4	Sand Cone - In situ	124.8	107.5	---	131.5	81.7

Visual Observations

Visual observations and mapping performed of the test pit walls, bulk density tests and stockpiles reveal that landfill deposit is highly variable. Significant (approximately 15 to 20 percent of the landfill deposit) oversize (larger than 12 inch) fragments are present within the fill. Of the oversize fraction, steel-reinforced concrete fragments, foundation elements, columns and other construction demolition debris are present with dimensions that range from 24 inches to more than 20 feet. One reinforced beam exposed in Test Pit 1 near bench 3 was 6 feet wide, 3 feet deep and more than 40 feet long (the ends were not exposed as the beam is longer than the width of the test pit). For many of the fragments, numerous and large reinforcing bars protrude from the concrete. In addition to concrete, drywall, roofing materials, wood, glass, steel beams, asphalt, water-filtration cake, a gas pump, a tire and other debris were observed and photo-documented. The amount, distribution and content of debris within the fill appears to be similar between the two pits.

Fill lifts are clearly visible in the walls of the test pits. In general, the tops of the rubble lifts are identified by a level to gently sloping soil caps. Lifts thicknesses are visible that vary from a few feet thick to more than 8 feet. The lifts also reveal little processing and spreading. Individual piles of debris that were apparently "end-dumped" in the landfill are surrounded by other end-dumped piles (tipping face) and in turn buried by additional lifts. The thick rubble fill lifts do not appear to have been processed, sorted or moisture conditioned. Nesting and bridging were primarily observed adjacent to very large oversize fragments and where oversize fragments were concentrated. Nesting and bridging also occurred where layers of drywall covered concrete fragments.

GROUNDWATER

Seeps and perched layers of water were encountered in the borings and deep pits. Groundwater was encountered in Boring 4 and a depth of 165 feet (elevation 242). At the subject property, historically high groundwater has been estimated to range between elevation 325 feet on the south and 328 feet on the north (Figure 10-3 - *Estimated Historic High Groundwater Contours* in Technical Guidelines for Open-Pit Mines, City of Irwindale, 2005). The equates to depths of 72 to 75 feet. This is shallower than the depth of 110 feet shown on (Plate 1.2, *Historically Highest Groundwater Contours and Borehole Log Data Locations, Baldwin Park 7½ Minute Quadrangle in Seismic Hazard Zone Report for the Azusa Quadrangle,* SHZR-022). Historical records indicate that the deepest groundwater elevation was approximately 200 feet.

GENERAL SEISMIC CONSIDERATIONS

Southern California is located in an active seismic region and numerous known and undiscovered earthquake faults are present in the region. Hazards associated with fault rupture and earthquakes include direct affects such as strong ground shaking and ground rupture, as well as secondary affects such as liquefaction, landsliding and lurching. The United States Geological Survey (USGS), California Geologic Survey (CGS), Southern California Earthquake Center (SCEC), private consultants and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and early warning of strong ground shaking. Research and practice have shown that earthquake prediction is not practical or sufficiently accurate to benefit the general public. Also, several recent and damaging earthquakes have occurred on faults that were unknown prior to rupture. Current standards and the California Building Code call for earthquake resistant design of structures as opposed to prediction.

Alquist-Priolo Fault Rupture Hazard Study Zone

California faults are classified as active, potentially active or inactive. Faults from past geologic periods of mountain building, but do not display any evidence of recent offset are considered "inactive" or "potentially active." Faults that have historically produced earthquakes or show evidence of movement within the Holocene (past 11,000 years) are considered "active faults." Active faults that are capable of causing large earthquakes may also cause ground rupture. The Alquist-Priolo Act of 1971 was enacted to protect structures from hazards associated with fault ground rupture. No known active faults cross the subject property and the site is not located within an Alquist-Priolo Fault Rupture Hazard Study Zone. The ground rupture hazard at the site is considered nil.

Building Code Seismic Coefficients

Seismic design parameters within the Building Code include amplification of the seismic forces on the structure depending on the soil type, distance to seismic source and intensity of shaking.

The purpose of the code seismic design parameters is to prevent collapse of structures and loss of life during strong ground shaking. Cosmetic damage should be expected.

The site is located within two kilometers of a known seismic source (Santa Monica fault). The following table lists the applicable seismic coefficients for the 2007 Building Code.

SEISMIC COEFFICIENTS (2007 California Building Code)		
Latitude = 34.110°N Longitude = 117.976°W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 1613.5.2 and Section 1613.5.2	Compacted Fill - S_D	
Seismic Design Category from Table 1613.5(1) and 1613.5(2)	D	
Spectral Accelerations from Figures 1613.5(3) and 1613.5(4)	S_s = 1.957 (g)	S_1 = 0.692 (g)
Site Coefficients from Tables 1613.5.3 (1) and 1613.5.3 (2)	F_A = 1.0	F_V = 1.5
Spectral Response Accelerations from Equations 16-37 and 16-38	S_{MS} = 1.96 (g)	S_{M1} = 1.04 (g)
Design Accelerations from Equations 16-39 and 16-40	S_{DS} = 1.30 (g)	S_{D1} = 0.69 (g)

Seismic Hazards

The principal seismic hazard to the subject property and proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear panels, moment-resisting frames and reinforcement. Additional precautions may be taken to protect personal property and reduce the chance of injury, including securing equipment and racks. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Seismic Hazard Zones

The California State Legislature enacted the Seismic Hazards Mapping Act of 1990, which was prompted by damaging earthquakes in California, and was intended to protect public safety from the effects of strong ground shaking, liquefaction, landslides, and other earthquake-related hazards. The Seismic Hazards Mapping Act requires that the State Geologist delineate various "seismic hazards zones." The maps depicting the zones are released by the California Geological Survey.

The Seismic Hazards Mapping Act requires a site investigation by a certified engineering geologist and/or civil engineer with expertise in geotechnical engineering, for projects sited within a hazard zone. The investigation is to include recommendations for a "minimum level of mitigation" that should reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy. The Seismic Hazards Mapping Act does not require mitigation to a level of no ground failure and/or no structural damage.

Seismic Hazard Zone delineations are based on correlation of a combination of factors, including: surface distribution of soil deposits; physical relief; depth to historic high groundwater; shear strength of the soils; and occurrence of past seismic deformation. The subject property is located within the United States Geologic Survey, Baldwin Park Quadrangle. Seismic hazards within the Baldwin Park Quadrangle were evaluated by the CGS in their report, *"Seismic Hazard Zone Report for the Baldwin Park 7.5-minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report* 022."

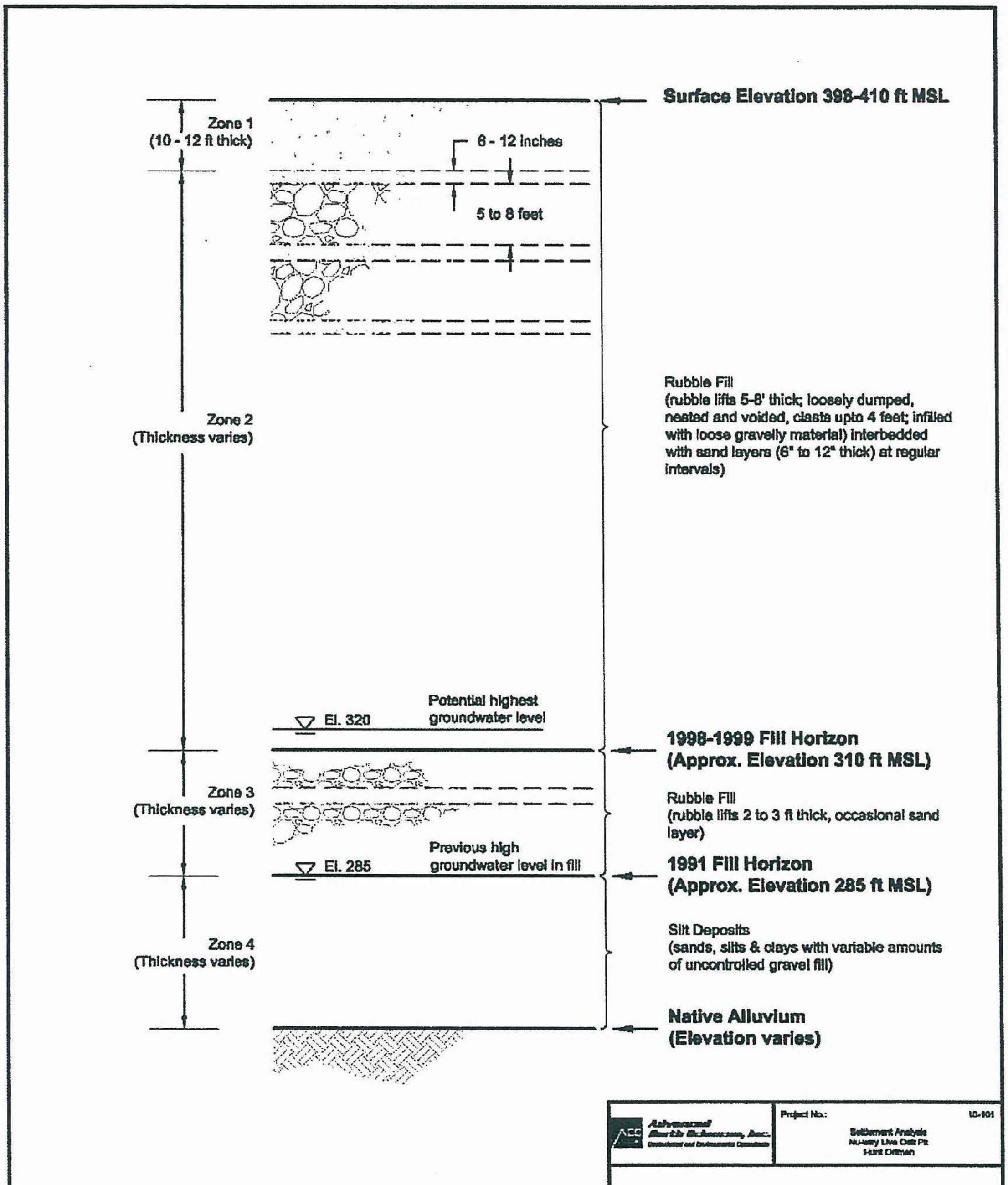
According to the Seismic Hazard Zones Map, the subject property is **not** within an area that has been subject to, or may be subject to liquefaction. The steep walls of the former pit are shown to have a potential earthquake induced ground deformation.

SETTLEMENT ANALYSIS - RESULTS OF STUDY BY ADVANCED EARTH SCIENCES

Advanced Earth Sciences, Inc. (AES) of Irvine, California was retained by the owner to perform a settlement analysis of the landfill. The result of their study are contained in *Technical Memorandum on Settlement Analysis* dated November, 2010, which is appended to this report.

Soil Profile

The soil profile for settlement analysis was based on the reported filling methods and stratigraphy of the debris fill as revealed during the pre-filling geotechnical investigations by Zeiser and post-filling investigations performed by Irvine Geotechnical. This idealized profile is illustrated in Figure 3-1 from the AES, which included on the following page.



Idealized Soil Profile

09-10

Figure 3-1

Settlement Analyses - General

Static and seismic settlement analysis models should reflect the actual fill conditions and placement practices. Conventional methods of settlement analysis, particularly for unsaturated materials under seismic loading conditions, are based primarily on laboratory cyclic shear studies on homogeneous sands. There are no industry-accepted standards to predict settlements of inert debris fill that contain significant oversize fragments and significant open voids as is the case at the Nu-Way pit. Also, there is no database on observed settlements of such debris filled pits in the area or vicinity. Settlement models to be used for predicting seismic and static settlement must take into account the lack of uniformity and control in fill placement operations, and the layered sequence of actual fill placement reported and observed for this site. Due to these reasons, there will be a significant degree of uncertainty associated with settlement predictions. The approach taken by AES was to provide a range of anticipated settlements supported by a rational settlement model, reasonable assumptions and parametric analysis.

The settlement model developed for the Nu-way pit considered the layered nature of much of the debris fills, particularly above the 1998/1999 horizon (approximate elev. 310-330), consisting of a succession of loose, voided and nested rubble lifts, typically 5 to 8 feet thick and capped by a 6 to 12 inch thick layer of loose to medium dense sands.

Seismic Settlements

Under seismic shaking, mechanisms contributing to settlement include the densification of the loose sand layers between the rubble layers, densification of the rubble layers, and the filling of open voids within the rubble layers due to a combination of sand migration into open voids and collapse of the nested clasts. The cumulative thickness of the sand layers and rubble layers, and the depth interval over which the succession of thin sand layers over thick rubble layers with open voids occur, were estimated at each of the six Becker Borehole locations by evaluating the respective blow count profiles. The volume of open voids in the rubble layers was calculated based on the results of large-diameter bulk density tests in the rubble fill, sand cone density tests in the infill soils and on the particle size distribution tests in the rubble fill materials excavated from the large-diameter bulk density test holes.

The seismic settlement caused by densification of the sand layers was estimated using the conventional Tokimatsu & Seed (1987) procedure, using the measured Becker hammer blow counts as input. The seismic settlement caused by densification of the rubble layers was estimated also using the Tokimatsu & Seed procedure, using as input the measured seismic shear wave velocity profile and the measured blow counts in the rubble fill. Although the applicability of the Tokimatsu & Seed (1987) procedure for the seismic settlement of the rubble layer and the reliability of the predictions are questionable, it was used, nevertheless, to obtain a rough order of magnitude estimate. The third, more dominant, component of seismic settlement is caused by the partial filling of open voids in the rubble fill by a combination of fines migrating from overlying sand layers into the voids and collapse of the nested rubble clasts. The average volume of open voids (as a percentage of total volume) in the rubble layer was estimated to be 6.7 percent. Not all of the open voids will be filled as a result of seismic shaking. The proportion of open voids that get filled will depend on the amplitude, frequency and duration of shaking, but is not known in the absence of specific physical laboratory modeling. As an initial estimate, it was assumed that approximately 20% of the open voids are filled due to seismic shaking. Subsequently, sensitivity analyses were performed by varying this percentage. The total seismic settlements were evaluated at each of the six Becker hammer borehole locations. Based on these assumptions, the total seismic settlement of the in-place debris fill is estimated to range from 5.6 inches to 14.4 inches with an average of 11.2 inches. Considering that the total thickness of the debris fill contributing to the settlements is approximately 110 feet, the average settlement of 11.2 inches is roughly equivalent to 0.85 percent of fill thickness. This estimate compares well with some well documented case histories of settlement of dry compacted sandy fills in southern California which settled approximately 0.6 to 0.9 percent of fill thickness during the 1971 San Fernando and 1994 Northridge earthquakes, under ground accelerations comparable to the design ground accelerations for the Nu-Way pit. However, considering that the rubble fill consists of uncompacted fill with significant voids, the actual settlement could be even higher.

One of the remedial measures for controlling seismic settlement at the site will consist of partial removal and replacement of the existing debris fill with a properly processed and compacted fill cap. With increasing depth of removal and replacement of the existing fill, the remaining fill thickness vulnerable to seismic settlements would decrease, thereby resulting in lower seismic settlement potential. The presence of the cap will also serve to attenuate the total and differential settlement taking place at depth as it manifests at the surface of the fill cap. The non-linear finite difference Computer Program FLAC was used to model the impact of cap thickness on surface manifestation of total and differential settlements. The analysis considered the surface manifestation of differential settlement at a specific location (caused by an isolated large void) and the surface manifestation of randomly varying settlements applied at the base of the fill cap, for increasing thicknesses of fill cap. The results, presented as plots of surficial total and differential settlements versus thickness of fill cap (for a range of assumed values of percentage open voids filled by migration of fines and collapse), show decreasing values of surficial settlement with increasing fill cap thickness. For example, the results show that for a 40-foot thick fill cap the maximum total settlements are on the order of 4 to 10 inches, while the maximum surficial differential settlements range from 1 to 3.5 inches over a 30-foot length. If the fill cap thickness is increased to 70 feet, the total surficial settlements are less than 3.7 inches and differential surficial settlements are less than 1 inch over a 30-foot length, the latter satisfying the regulatory requirements for a site that can be developed with engineered structures.

Static Settlements

Under static loading, the components of settlement include the settlement of the debris fill under its self weight, long-term primary and secondary settlement of the of the silt deposits underlying the debris fill, and settlement caused by filling of open voids due to migration of fines (sands) and collapse as a result of fluctuations in the groundwater levels. Of these, the settlement of the debris fill due to self weight is anticipated to occur during and immediately following completion of filling. The majority of the long-term settlement of the silt deposits due to the debris fill loads is estimated to have been complete over the 4+ years that have elapsed since completion of filling operations.

Historical groundwater data indicate that the groundwater levels fluctuate, with an estimated high groundwater level at elevation 330 feet. The placement history (1991-2006) suggests that the fills placed above approximate elevation 290 feet may not have been subjected to saturation due to groundwater fluctuations during the pit filling period. The estimated total settlement caused by groundwater fluctuations will depend on the percentage of open voids that get filled to due to migration of fines and collapse. Assuming that 20 to 40 percent of the open voids in the rubble fills get filled due to fines migration and collapse caused by groundwater fluctuations during the pit filling period, the resulting total settlements are estimated to range from 3.2 to 6.4 inches. These settlements occur at a depth of 80 feet below the finish ground surface. The corresponding differential settlements at the ground surface are estimated to be less than 1 inch over 30 feet.

Because the same mechanism (migration of sands into open voids and collapse) control both seismic settlement and settlement due to groundwater fluctuations, the two components of settlement are not considered to be cumulative, and the maximum differential settlement due to both components may still be less than 1 inch in 30 feet.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon subsurface exploration, field geologic mapping, research of available records, consultation, and years of experience observing similar properties in similar settings and review of the development plans. It is the finding of Irvine Geotechnical that construction of the proposed project is feasible from a geotechnical engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The site is underlain by up to 180 feet of fill, which is of variable composition, density and quality. However, it is clear that the upper 65 to 75 feet of the fill consists of highly uncontrolled fill placed in excessively thick lifts and with minimal oversight and contains voids and nesting of oversized material. Settlement analyses performed by AES (Appendix I) indicates that the fill deposit in the current condition has a potential for static settlement of about 3.2 inches. For the maximum considered earthquake, the dynamic settlement potential of the existing fill ranges from 5.6 to 14.4 inches. The corresponding differential settlement at the surface will be well in excess of 1 inch in 30 feet.

It is not considered feasible to remove and recompact the entire Nu-Way landfill. Nor is it considered feasible to penetrate the fill deposit with deep foundations that derive support in the native alluvial deposits below the fill. It is proposed to create a compacted fill cap to support the proposed structures. Static settlement of the fill or induced settlement of the fill under structural building loading is not considered an issue. It is desired to limit differential settlements under static and dynamic loading to less than 1 inch in 30 feet.

AES modeled engineered compacted fill caps of varying thicknesses (40 feet, 60 feet and 80 feet). A compacted fill cap as thick as 80 feet resulted in very little seismically induced settlement. For a 70-foot thick fill cap, total dynamic surficial settlement is reduced to 1.4 to 3.7 inches, with an average of 2.5 inches. For a 70-foot thick cap, differential surficial settlement is reduced to a range of 0.30 inches in 30 feet to just under 1.0 inches in 30 feet. It is recommended that the upper 70 feet of fill be removed, processed (crushed to a maximum particle no greater than 12 inches) and recompacted for structural support of buildings, slabs, paving and infrastructure. Conventional foundations and slabs will be appropriate after the recommended remedial grading.

Geotechnical Issues

Geotechnical issues affecting the site include deep over-excavation, a high volume of oversize material and debris, and a potential for differential settlement around the margins of the pit that is not to be mitigated. Special detailing and design will be required where utility lines and pipes enter and exit the property. The lines will need to be flexible to accommodate the potential differential settlement. The transition area between the native and fill soils should be over-excavated five feet and recompacted. The cap and over-excavation should extend 10 feet into native soils beyond the transition. For the transition zone, two layers of geogrid are recommended to minimize ground cracking resulting from differential settlement. The lower and upper layers should be placed at depths of 4 and 2 feet below the ground surface, respectively. The geogrid reinforcement should extend 10 feet to either side of the transition contact.

Code Section 111

Relative to Code Section 111, provided that the recommendations contained in this report are included in the design and implemented in the field, the proposed development will not be subject to geologic and geotechnical hazards associated with settlement, slippage, landsliding, expansive soils, liquefaction or chemical attack. Also, construction of the project will not have an adverse effect on the offsite properties and the public right-of-way.

REMEDIAL GRADING RECOMMENDATIONS - SITE PREPARATION

Surficial materials consisting of poorly processed and compacted fill are present on the site. Remedial grading is recommended to improve site conditions. The earth materials should be processed and the fill placed in conformance with City of Irwindale guidelines (Guidelines for Above-Water Backfilling of Open-Pit Mines, Irwindale, California in Technical Guidelines for Open-Pit Mines, City of Irwindale, 2005).

General Grading Specifications

The following guidelines may be used in preparation of the grading plan and job specifications. Irvine Geotechnical would appreciate the opportunity of reviewing the plans to insure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The site should be prepared to receive compacted fill by removing all vegetation, debris and upper 70 feet of existing fill. The exposed excavated area should be observed by the soils engineer prior to placing compacted fill. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 93 percent of the maximum achievable density.

B. Fill, consisting of soil approved by the soils engineer and in conformance with the City of Irwindale standards, shall be placed in horizontal lifts and compacted in maximum 12-inch thick, loose layers with suitable compaction equipment. Upon processing, including crushing and/or screening to remove oversize materials and other organic debris, the excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks and concrete/rubble fragments larger than 12 inches in largest dimension shall not be used in the fill.

C. The fill shall be compacted to at least 93 percent of the maximum achievable density for the material used. The fill should be placed at a moisture content that is at or within 3 percent over optimum. The maximum density and optimum moisture content shall be determined by following the City's backfilling and compaction standards, which are appended to this report.

D. Field observation and testing shall be performed by the soils engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until 93 percent compaction is obtained. The type and frequency of testing should conform to City's Above-water Backfilling Guidelines, which are appended to this report.

Excavation Characteristics

The test pits did encounter large reinforced concrete piles, piers and beams and other construction debris. Significant processing of the fill will be required for re-use in the structural fill.

FOUNDATION DESIGN

General Conditions

The following foundation recommendations are minimum requirements. The structural engineer may require footings that are deeper, wider, or larger in diameter, depending on the final loads. Mat foundations are not anticipated.

Spread Footings

Continuous and/or pad footings may be used to support the proposed structures provided they are founded in approved compacted fill. Continuous footings should be a minimum of 12 inches in width. Pad and column footings should be a minimum of 24 inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Approved Compacted Fill	18	2,000	0.40	250	4,000

Increases in the bearing value are allowable at a rate of 400 pounds per square foot for each additional foot of footing width or depth to a maximum of 4,000 pounds per square foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one third.

The on-site soils are non-expansive. All continuous footings should be reinforced following the recommendations of the structural engineer. As a minimum, it is recommended that continuous footings be reinforced with four #4 steel bars; two placed near the top and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geotechnical engineer prior to placing forms, steel or concrete.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A settlement of ¼ to ½ inch may be anticipated under the normal building loads. Differential settlement should not exceed ½ inch in 30 feet.

Differential settlement of the ground surface is predicted for the maximum considered earthquake. Based upon the analyses by AES, the differential settlement is expected to be less than 1 inch in 30 feet.

RETAINING WALLS

General Design

Significant retaining walls are not anticipated for the site and the proposed project. Retaining walls will mostly be restricted to loading docks, ramps and planters. Cantilevered retaining walls up to 6 feet high that support approved retaining wall backfill, may be designed for an equivalent fluid pressure of 35 pounds per cubic foot. Select granular backfill approved by the geotechnical engineer is recommended.

Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾ inch crushed gravel.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-09. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾ inch crushed gravel to within 2 feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper 2 feet of backfill above the gravel should consist of a compacted fill blanket to the surface. Retaining wall backfill should be capped with a paved surface drain or a concrete slab.

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TEMPORARY EXCAVATIONS

Temporary excavations will be required to remove and recompact the upper 70 feet of fill. Where not surcharged by existing footings or structures, the fill is capable of maintaining vertical excavations up to 5 feet. Where vertical excavations in the fill exceed 5 feet in height, the upper portion should be trimmed to 1:1 (45 degrees). Temporary 1:1 excavations in the rubble fill are considered stable up to 70 feet high.

A representative of the geotechnical engineer or geologist should be present during grading to see temporary slopes. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward them. No vehicular surcharge should be allowed within three feet of the top of the cut.

CORROSION

The compaction report should contain the results of chemical testing for fill placed within 5 feet of finished grade. Soils with high sulfate concentrations should not be imported to the site.

FLOOR SLABS, CONCRETE DECKING AND PAVING

Floor slabs and concrete decking should be cast over the approved compacted fill cap. Slabs should be at least 5 inches thick and reinforced with a minimum of #4 bars on 16 inch centers, each way. Care should be taken to cast the reinforcement near the center of the slab. Slabs which will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be covered with a thin layer of sand, about two inches, to prevent punctures and aid in the concrete cure.

Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

It should be noted that cracking of concrete floor slabs is very common during curing. The cracking occurs because concrete shrinks as it dries. Crack control joints which are commonly used in exterior decking to control such cracking are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the slab's performance. The minor shrinkage cracks which often form in interior slabs generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile. A mortar bed or slip sheet is recommended between the slab and tile to limit, the potential for cracking.

Slabs should be protected with a polyethylene plastic vapor barrier placed beneath the slab. This barrier is intended to prevent the upward migration of moisture from the subgrade soils through the porous concrete slab. It should be noted that vapor barriers are penetrated by any number of elements including water lines, drain lines, and footings. These barriers are therefore not completely watertight. It is recommended that a surface seal be placed on slabs which will receive a wood floor. The floor installer should be consulted regarding an adequate product.

The paving section should be cast over approved compacted fill. R-values of the near surface soils should obtained for representative soils near finished grade. The paving section may be fine-tuned or modified depending on the as-graded conditions of the site. The following table contains preliminary paving sections assuming a minimum R-value of 50. It should be noted that the onsite materials have been historically used to produce commercial CMB.

Trench backfill below paving, should be compacted to 93 percent of the maximum dry density. Irrigation water should be prevented from migrating under paving. The following table shows the recommended pavement sections:

Service	Pavement Thickness (Inches)	Base Course (Inches)
Traffic Index = 4	3	0
Traffic Index = 5	4	0
Traffic Index = 6	4/5	3/0

Base course should be compacted to at least 95 percent of the maximum dry density.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad. The 2007 California Building Code specifies that the grade within 10 feet of the foundation be sloped to drain at a 5 percent gradient away from the building. Drainage control devices require periodic cleaning, testing and maintenance to remain effective.

PLAN REVIEW

Formal plans ready for submittal to the Building Department should be reviewed by Irvine Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

Please advise Irvine Geotechnical at least 24 hours prior to any required site visit. The agency approved plans and permits should be at the jobsite and available to our representative. The project consultant will perform the observation and post a notice at the jobsite of his visit and findings. This notice should be given to the agency inspector.

During construction, a number of reviews by this office are recommended to verify site geotechnical conditions and conformance with the intent of the recommendations for construction. Although not all possible geotechnical observation and testing services are required by the reviewing agency, the more site reviews requested, the lower the risk of future problems. It is recommended that all grading, foundation, and drainage excavations be seen by a representative of the geotechnical engineer PRIOR to placing fill, forms, pipe, concrete, or steel. Any fill which is placed should be approved, tested, and verified if used for engineering purposes. Temporary excavations should be observed by a representative of the Geotechnical Engineer.

The following site reviews are advised or required. Should the observations reveal any unforeseen hazards, the geologist/engineer will recommend treatment.

Pre-construction meeting	Advised
Temporary excavations	Required
Bottom excavation for removals	Required
Compaction of fill	Required
Foundation excavations	Required
Slab subgrade moisture barrier membrane	Advised
Slab subgrade rock placement	Advised
Slab steel placement	Advised
Subdrain and rock placement behind retaining walls	Required
Compaction of retaining wall backfill	Required
Compaction of utility trench backfill	Advised

Irvine Geotechnical requires at least a 24 hour notice prior to any required site visits. The approved plans and building/grading permits should be on the job and available to the project consultant.

FINAL INSPECTION

Many projects are required by the agency to have final geologic and soils engineering reports upon completion of the grading.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist on a site, the area should be fenced and warning signs posted. Soil generated by foundation and subgrade excavations should be either removed from the site or properly placed as a certified compacted fill. Soil must not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep.

GENERAL CONDITIONS

This report and the exploration are subject to the following NOTICE. Please read the NOTICE carefully, it limits our liability.

NOTICE

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by us and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein and shown on the enclosed cross sections have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations that may occur between these excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications or recommendations during construction requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report is issued and made for the sole use and benefit of the client, is not transferable and is as of the exploration date. Any liability in connection herewith shall not exceed the fee for the exploration. No warranty, expressed or implied, is made or intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Irvine Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this or the referenced and appended reports should be directed to the undersigned.

Respectfully submitted,
Irvine Geotechnical, Inc.

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Enc: Moisture-Density Relationship
Grain Size Distribution (14 plates)
Vicinity Map
Log of Borings - Becker Hammer Borings by Irvine (6 pages)
Log of Large Diameter Boring (4 sheets)
Log of Borings - Rotary Wash Borings by Zeiser (9 sheets)
Blow Count Comparison Charts (6 sheets)
NAVFAC Density Charts (2)
Test Pit Map
Velocity Profiles (6)
APPENDIX I - Report by Advance Earth Sciences
APPENDIX II - Excepts from City of Irwindale Grading Requirements
APPENDIX III - Report by GeoVision
APPENDIX IV - Phase I, II, & III reports by Irvine Geotechnical
In pocket: Geologic Maps and Sections A, B and C

xc: (10) Addressee

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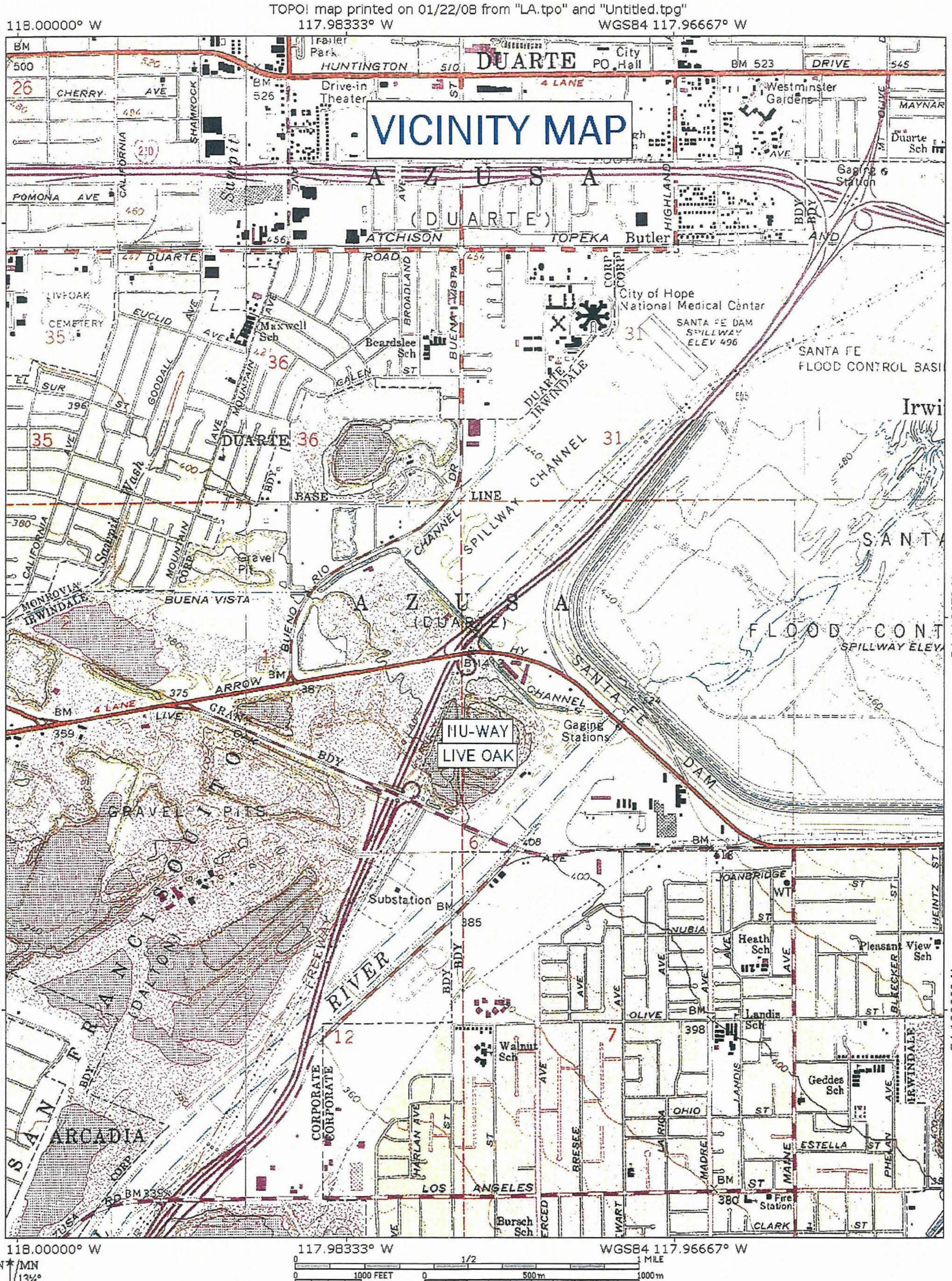

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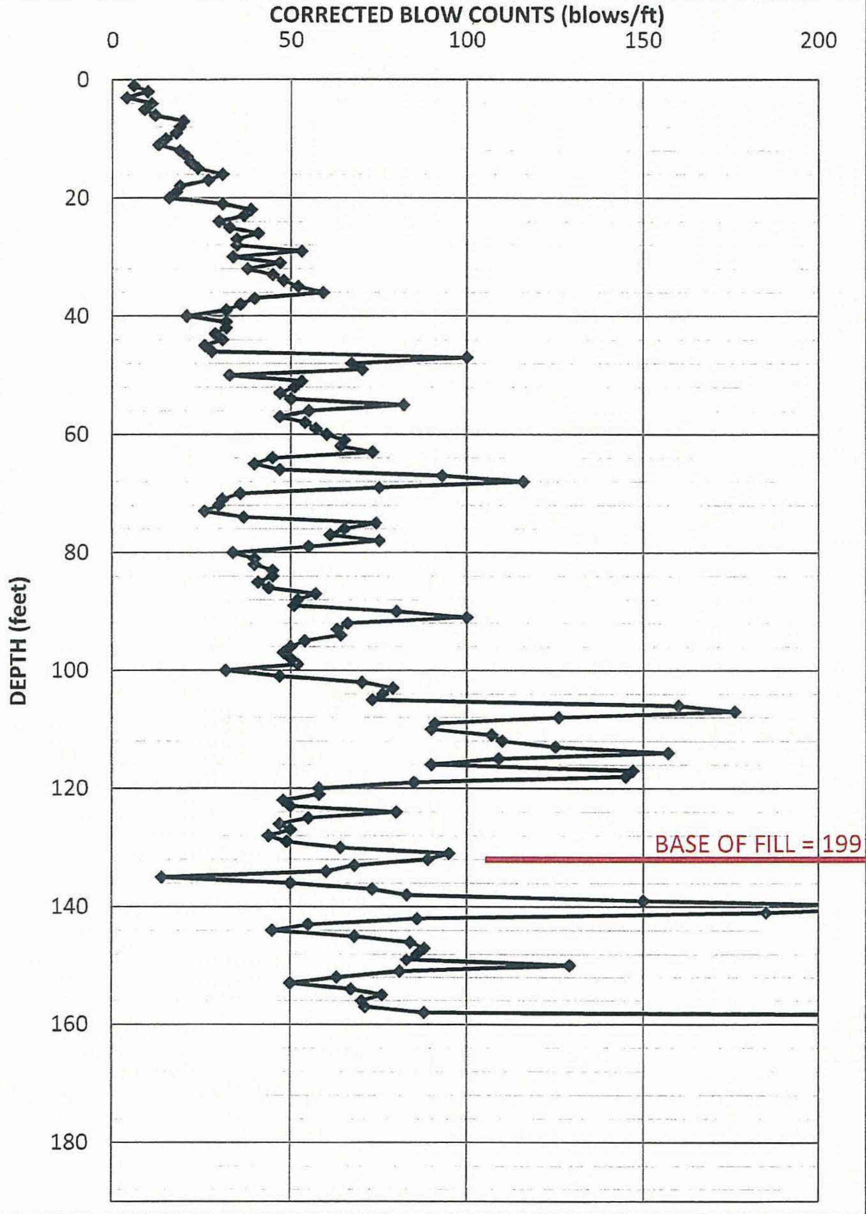
LOG OF BORINGS

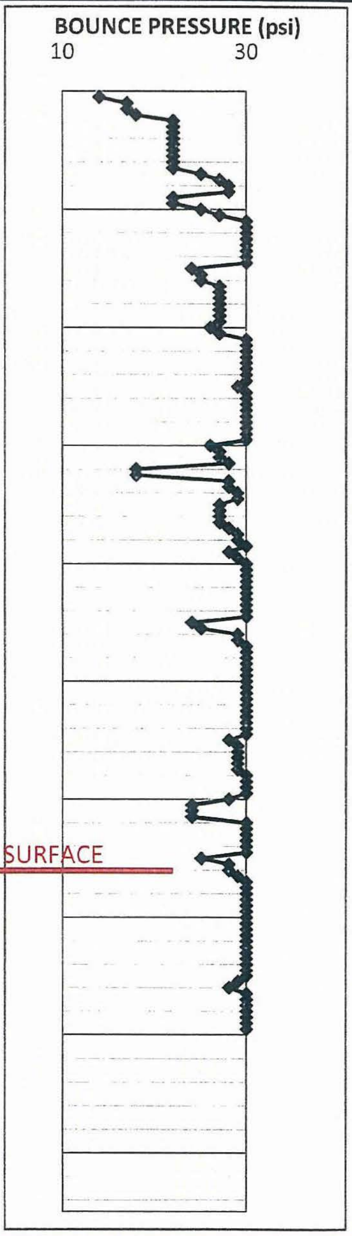
IRVINE GEOTECHNICAL Inc

PROJECT	IC 07034-I NU-WAY
DRILL DATE	10/15/2007
LOG DATE	10/15/2007
LOGGED BY	JAI
DRILL TYPE	BECKER HAMMER - CLOSED BIT
DIAMETER	8 INCHES

SURFACE ELEVATION	405 feet
DRILLING CONTRACTOR	GREAT WEST DRILLING
SURFACE CONDITIONS	Level dirt surface

BORING 1



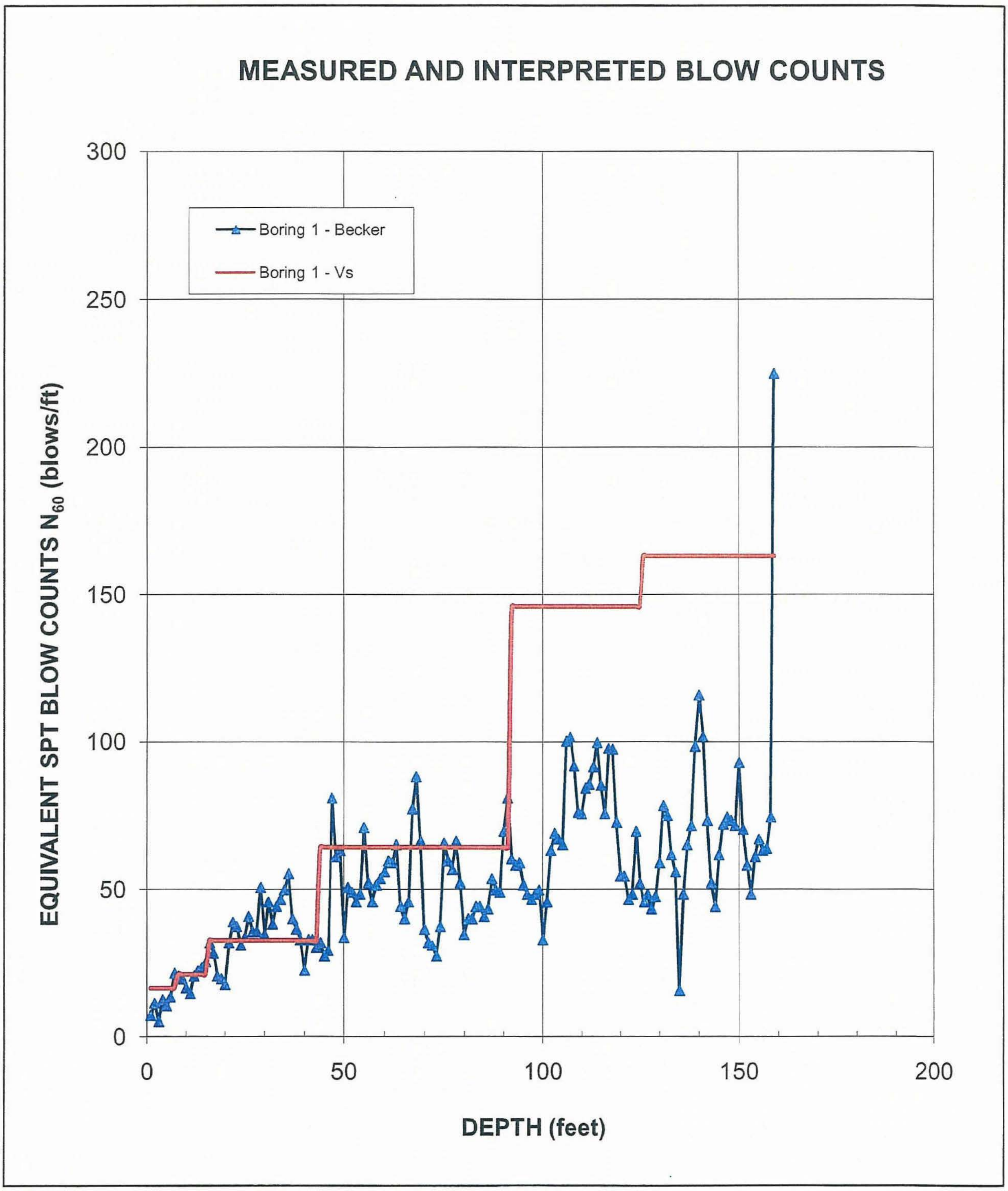


IRVINE GEOTECHNICAL Inc

BLOW COUNT COMPARISON

IC: 07034 CONSULT: JAI
CLIENT: IC 07034-I NU-WAY

GRAPH # 1



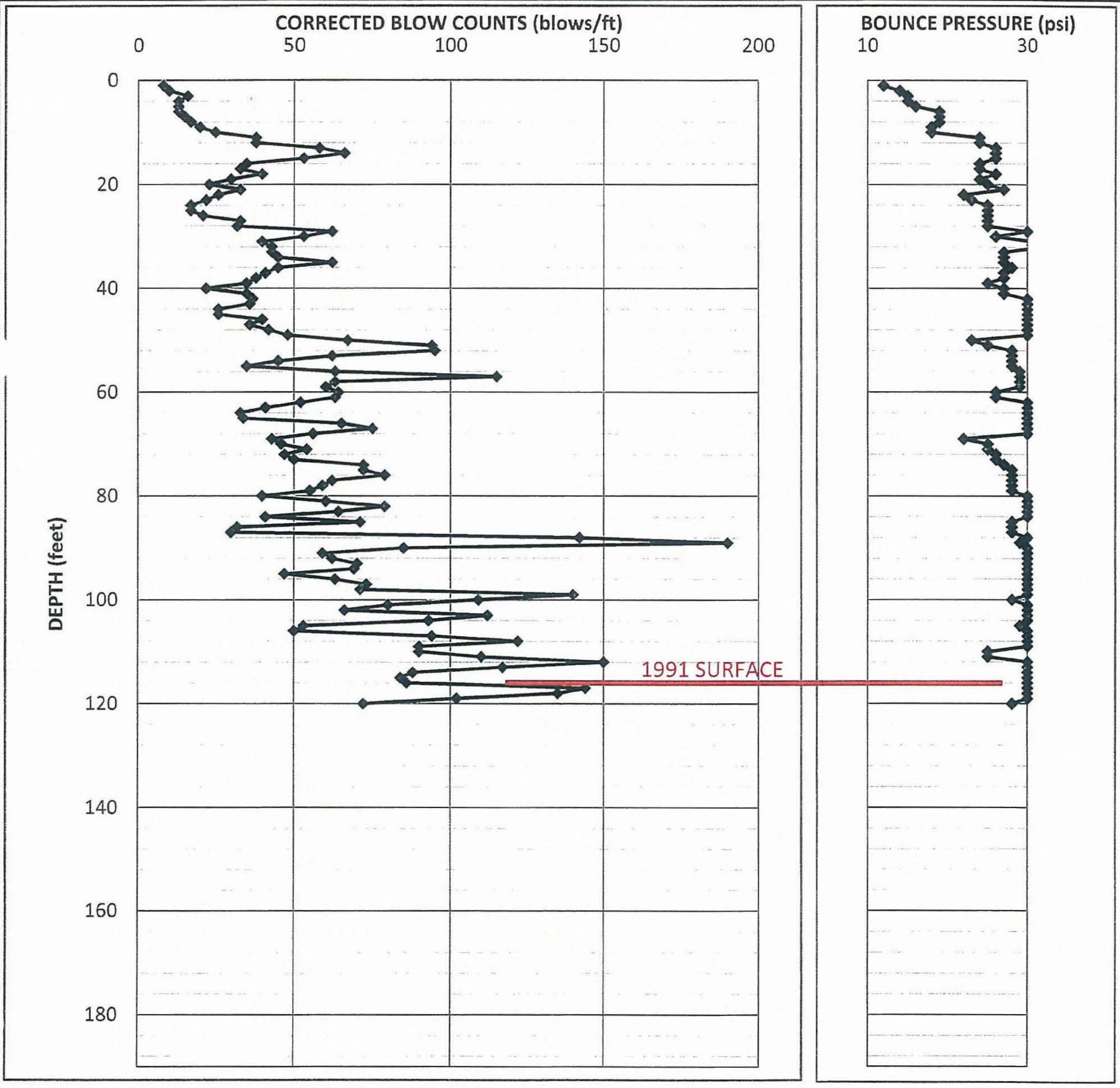
LOG OF BORINGS

IRVINE GEOTECHNICAL Inc

PROJECT	IC 07034-I NU-WAY
DRILL DATE	10/16/2007
LOG DATE	10/16/2007
LOGGED BY	JAI
DRILL TYPE	BECKER HAMMER - CLOSED BIT
DIAMETER	8 INCHES

SURFACE ELEVATION	405 feet
DRILLING CONTRACTOR	GREAT WEST DRILLING
SURFACE CONDITIONS	Level dirt surface

BORING 2

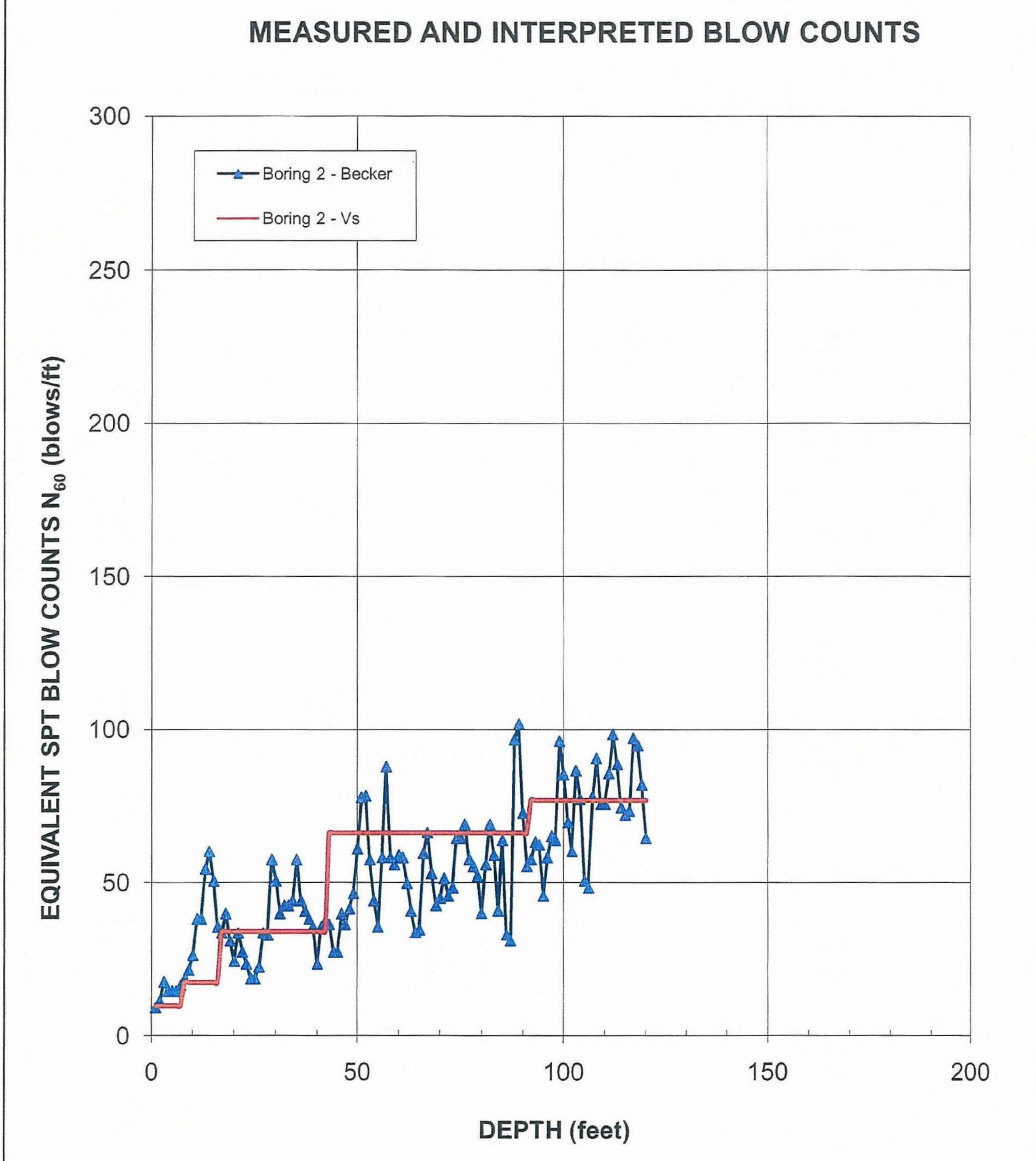


IRVINE GEOTECHNICAL Inc

BLOW COUNT COMPARISON

IC: 07034 CONSULT: JAI
CLIENT: IC 07034-I NU-WAY

GRAPH # 2



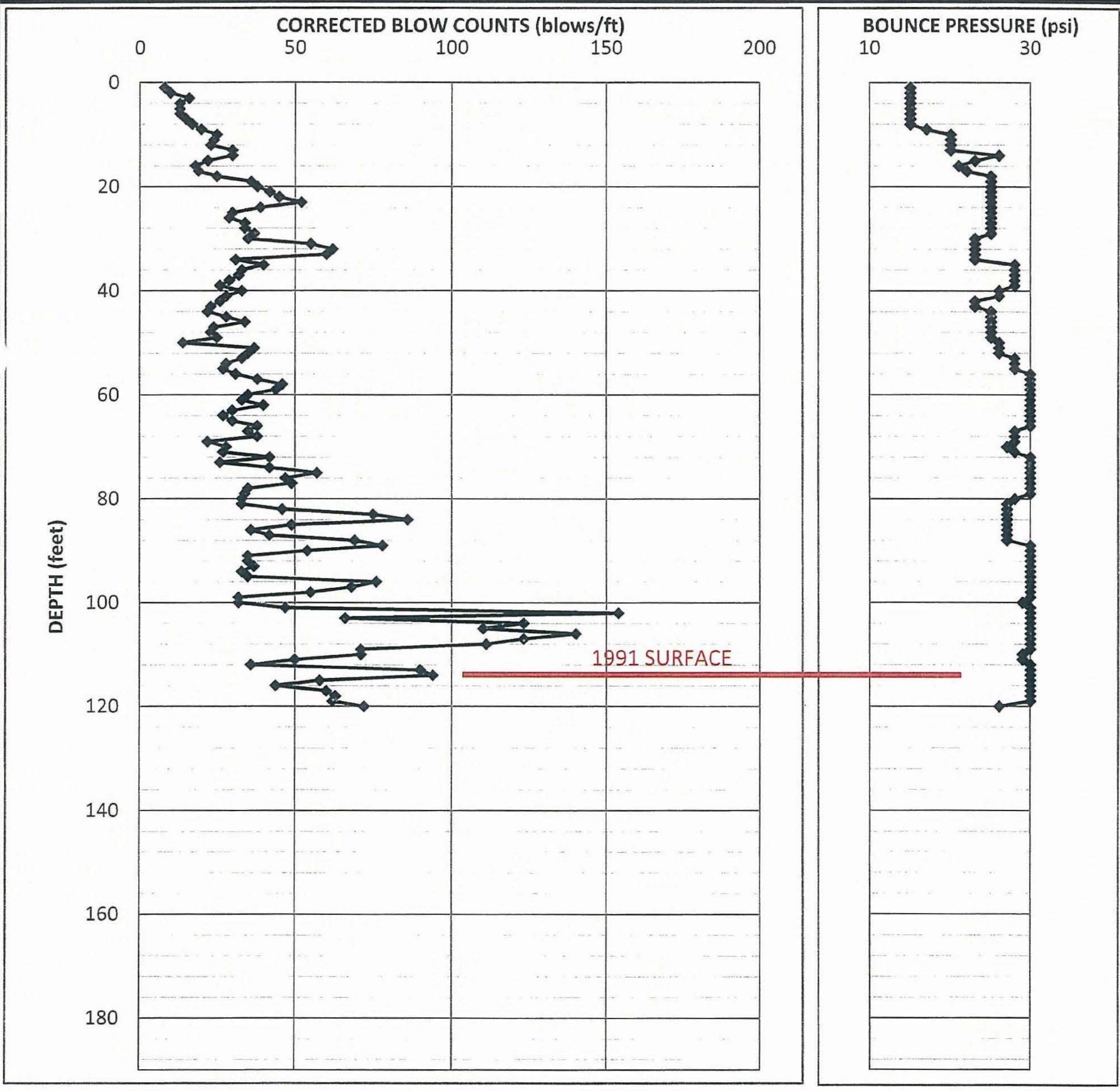
LOG OF BORINGS

IRVINE GEOTECHNICAL Inc

PROJECT	IC 07034-I NU-WAY
DRILL DATE	10/17/2007
LOG DATE	10/17/2007
LOGGED BY	JAI
DRILL TYPE	BECKER HAMMER - CLOSED BIT
DIAMETER	8 INCHES

SURFACE ELEVATION	398 feet
DRILLING CONTRACTOR	GREAT WEST DRILLING
SURFACE CONDITIONS	Level dirt surface

BORING 3

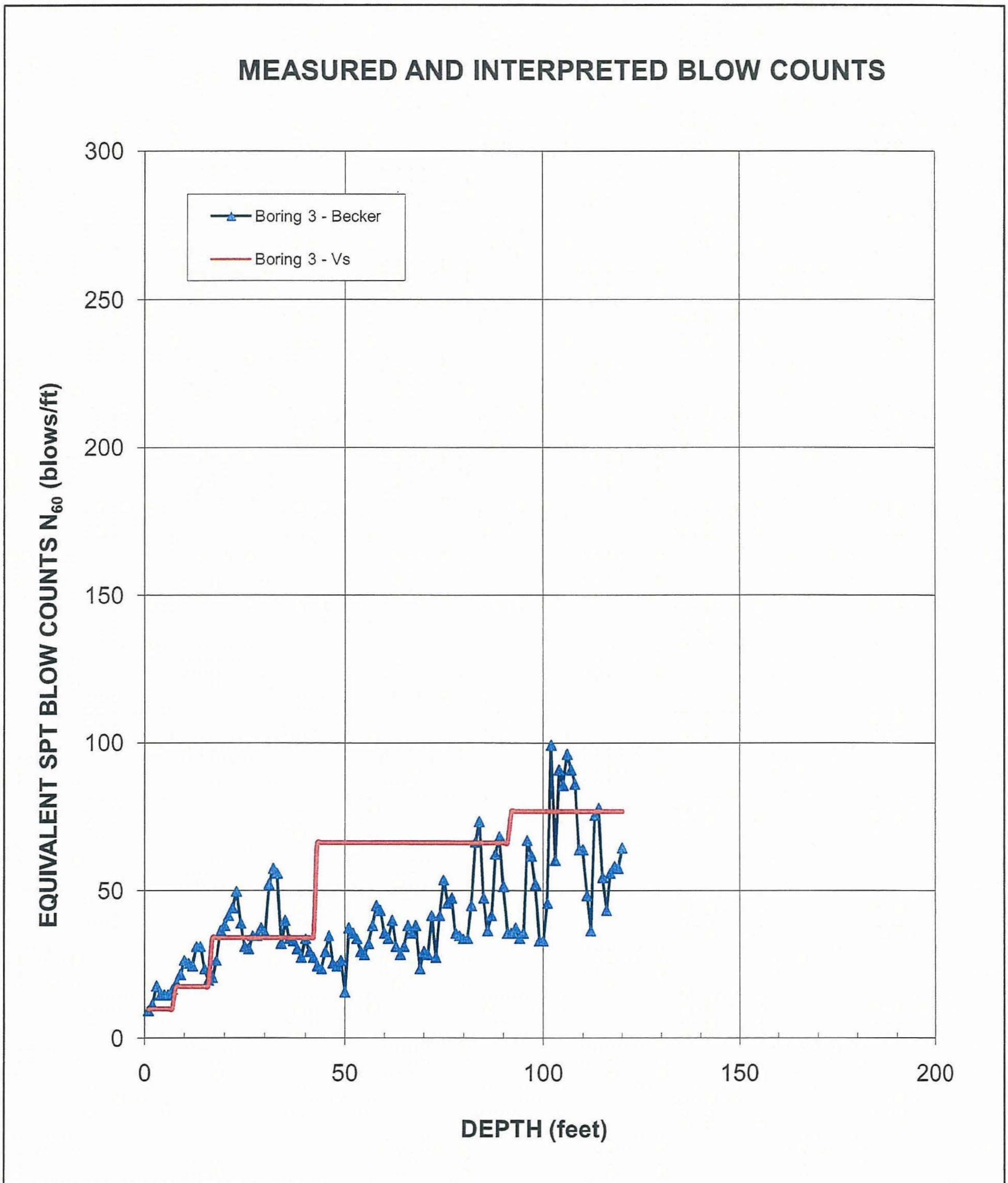


IRVINE GEOTECHNICAL Inc

BLOW COUNT COMPARISON

IC: 07034 CONSULT: JAI
CLIENT: IC 07034-I NU-WAY

GRAPH # 3



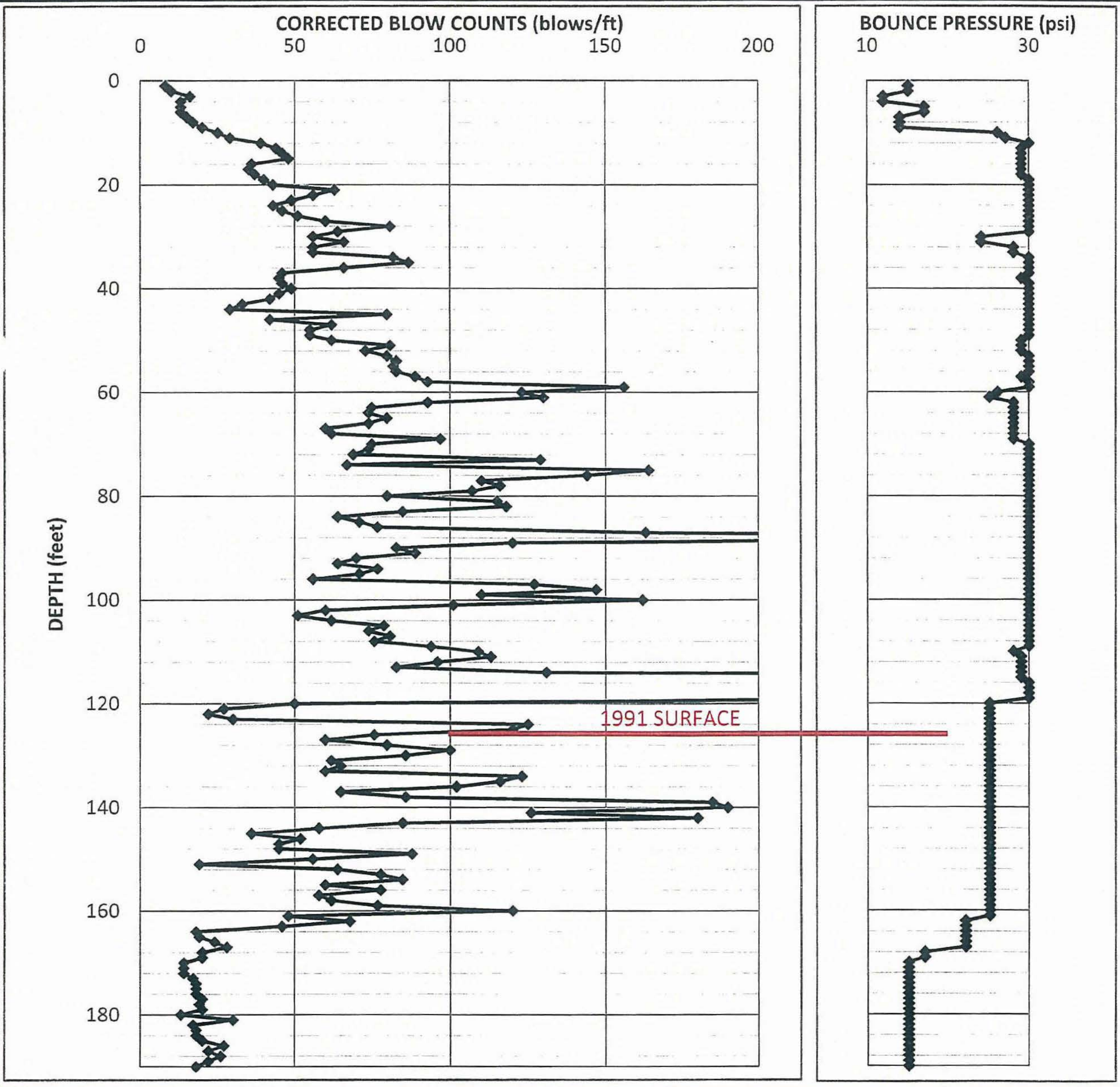
LOG OF BORINGS

IRVINE GEOTECHNICAL Inc

PROJECT	IC 07034-I NU-WAY
DRILL DATE	10/18/2007
LOG DATE	10/18/2007
LOGGED BY	JAI
DRILL TYPE	BECKER HAMMER - CLOSED BIT
DIAMETER	8 INCHES

SURFACE ELEVATION	405 feet
DRILLING CONTRACTOR	GREAT WEST DRILLING
SURFACE CONDITIONS	Level dirt surface

BORING 4



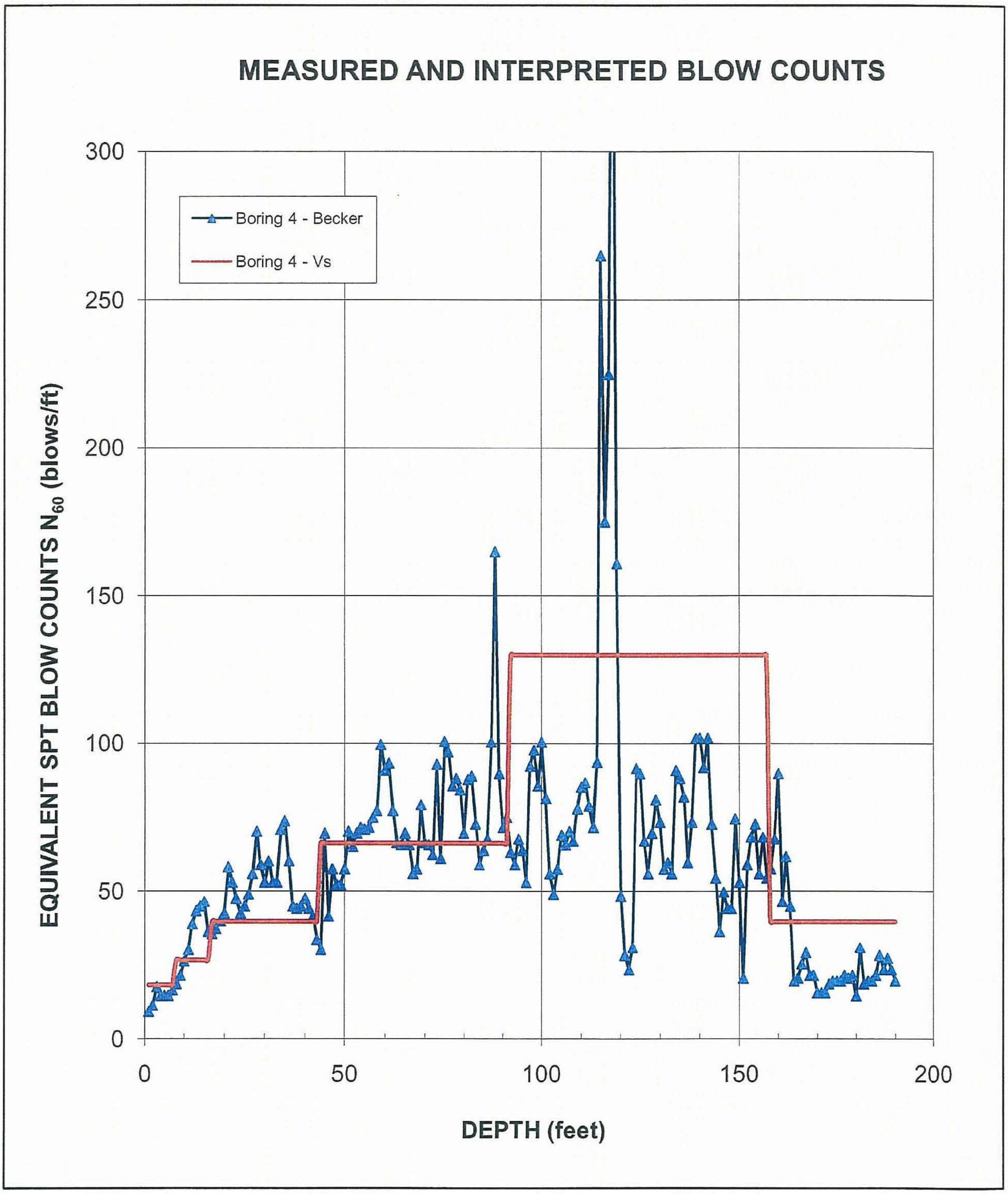
IRVINE GEOTECHNICAL Inc

BLOW COUNT COMPARISON

IC: 07034 CONSULT: JAI

CLIENT: IC 07034-I NU-WAY

GRAPH # 4

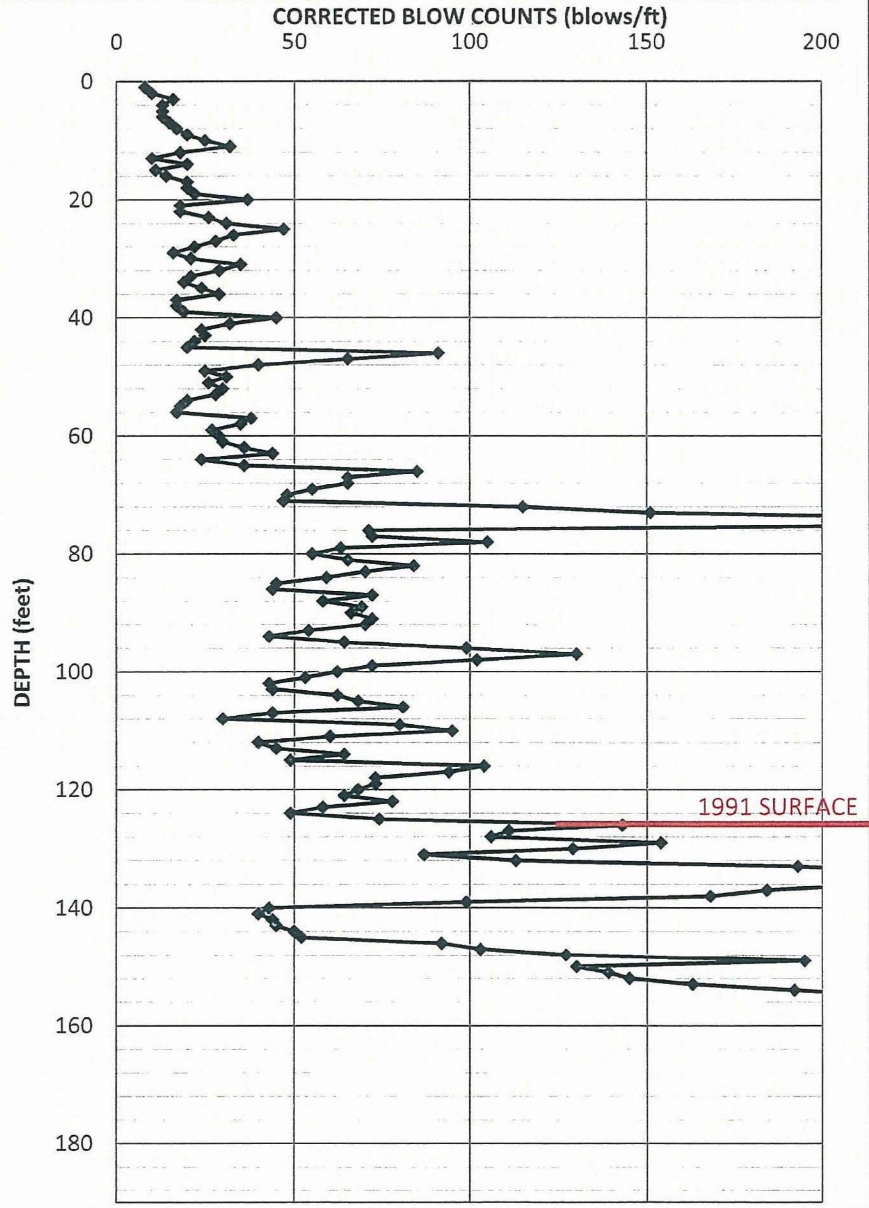


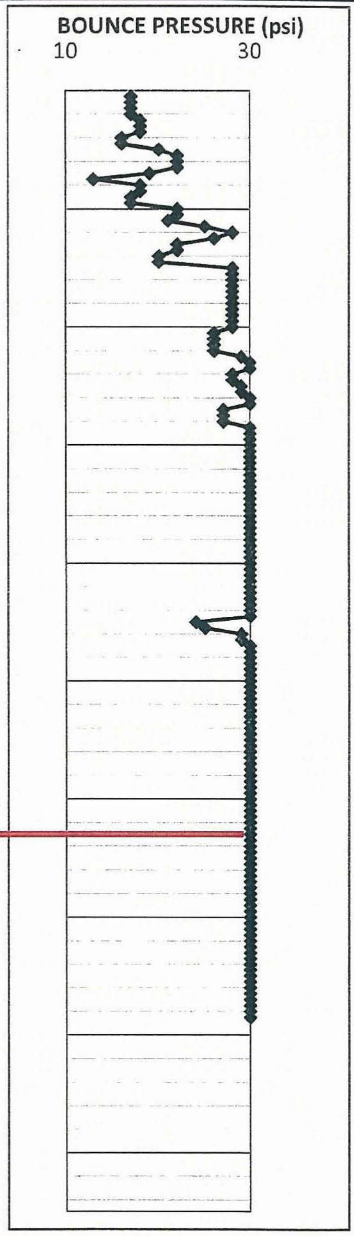
LOG OF BORINGS

PROJECT	IC 07034-I NU-WAY
DRILL DATE	10/19/2007
LOG DATE	10/19/2007
LOGGED BY	JAI
DRILL TYPE	BECKER HAMMER - CLOSED BIT
DIAMETER	8 INCHES

SURFACE ELEVATION	405 feet
DRILLING CONTRACTOR	GREAT WEST DRILLING
SURFACE CONDITIONS	Level dirt surface

BORING 5

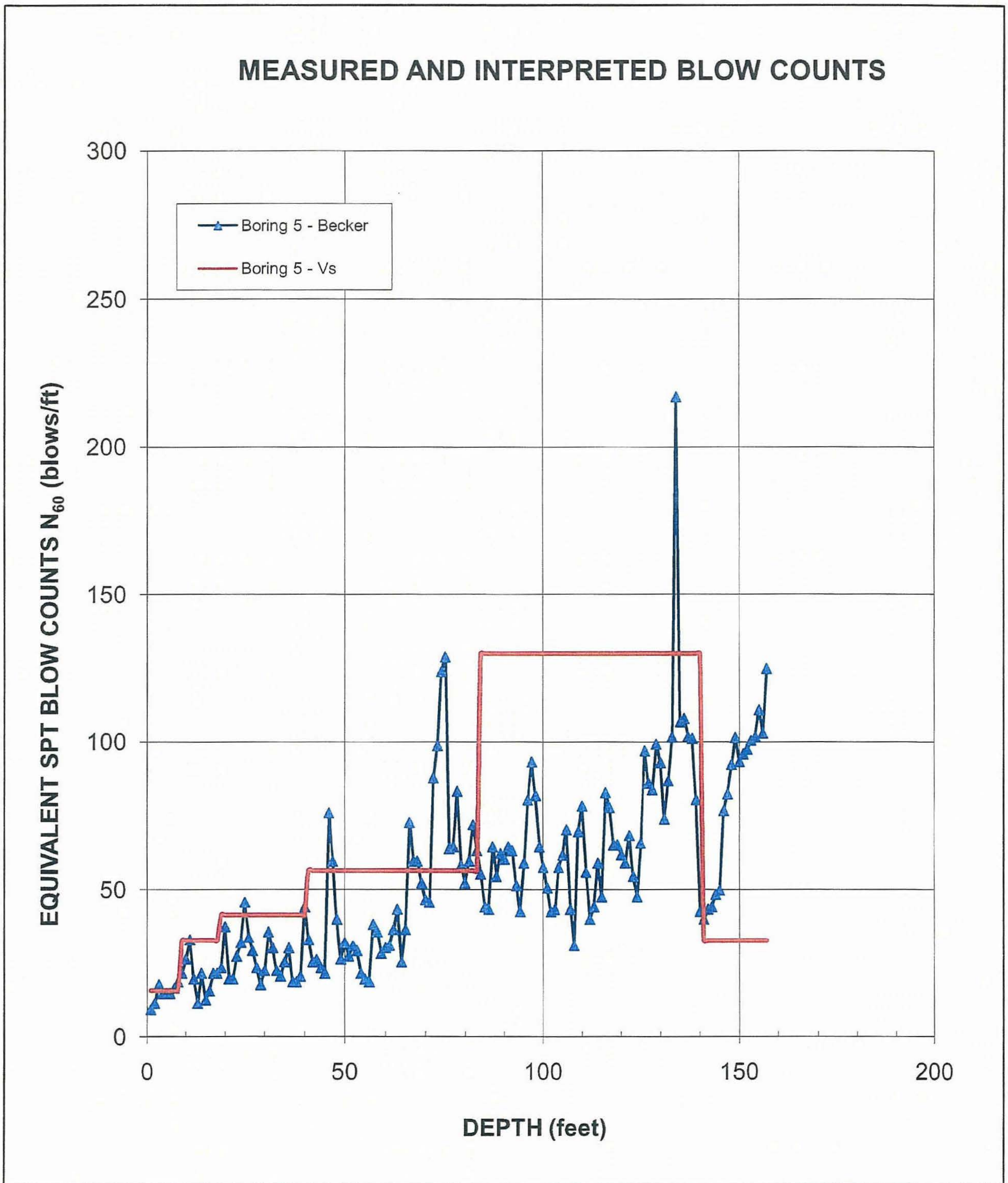




IRVINE GEOTECHNICAL Inc

BLOW COUNT COMPARISON

IC:	07034	CONSULT:	JAI
CLIENT:	IC 07034-I NU-WAY		
GRAPH #	5		



LOG OF BORINGS

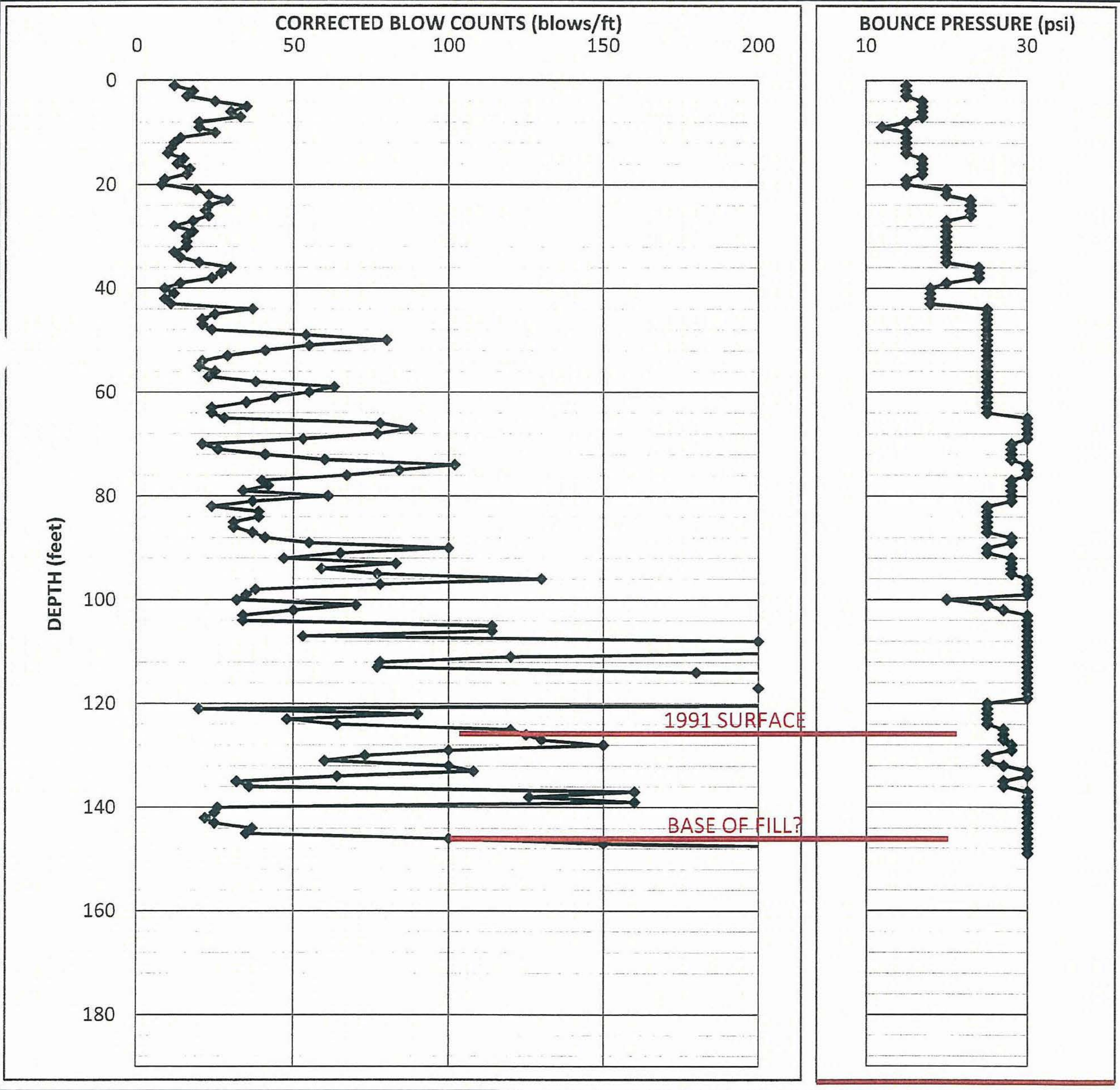
IRVINE GEOTECHNICAL Inc

PROJECT	IC 07034-I NU-WAY
DRILL DATE	10/22/2007
LOG DATE	10/24/2007
LOGGED BY	JAI
DRILL TYPE	BECKER HAMMER - CLOSED BIT
DIAMETER	8 INCHES

SURFACE ELEVATION	404 feet
DRILLING CONTRACTOR	GREAT WEST DRILLING
SURFACE CONDITIONS	Level dirt surface

BORING 6

Page 1 of 1

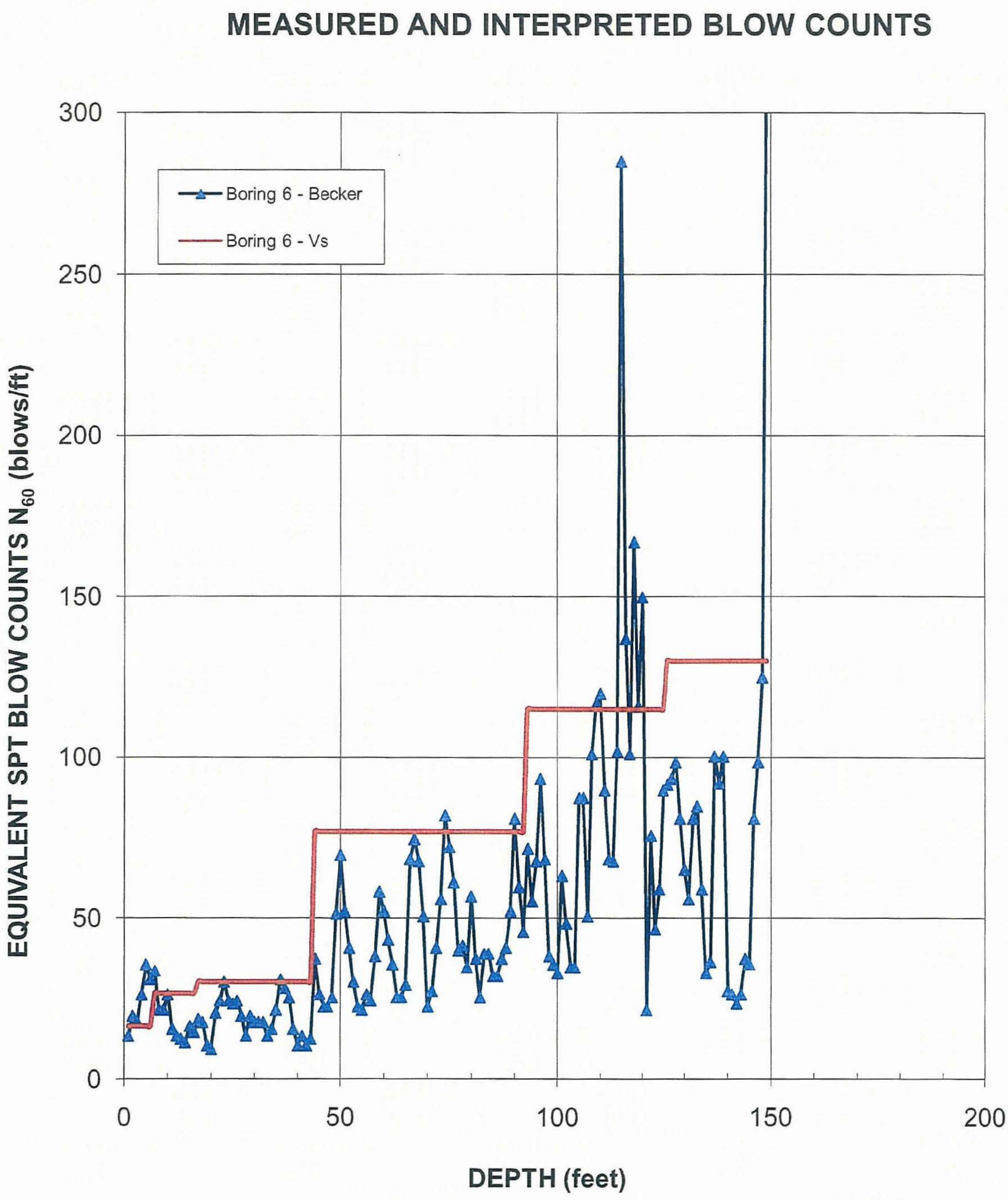


IRVINE GEOTECHNICAL Inc

BLOW COUNT COMPARISON

IC: 07034 CONSULT: JAI
CLIENT: IC 07034-I NU-WAY

GRAPH # 6



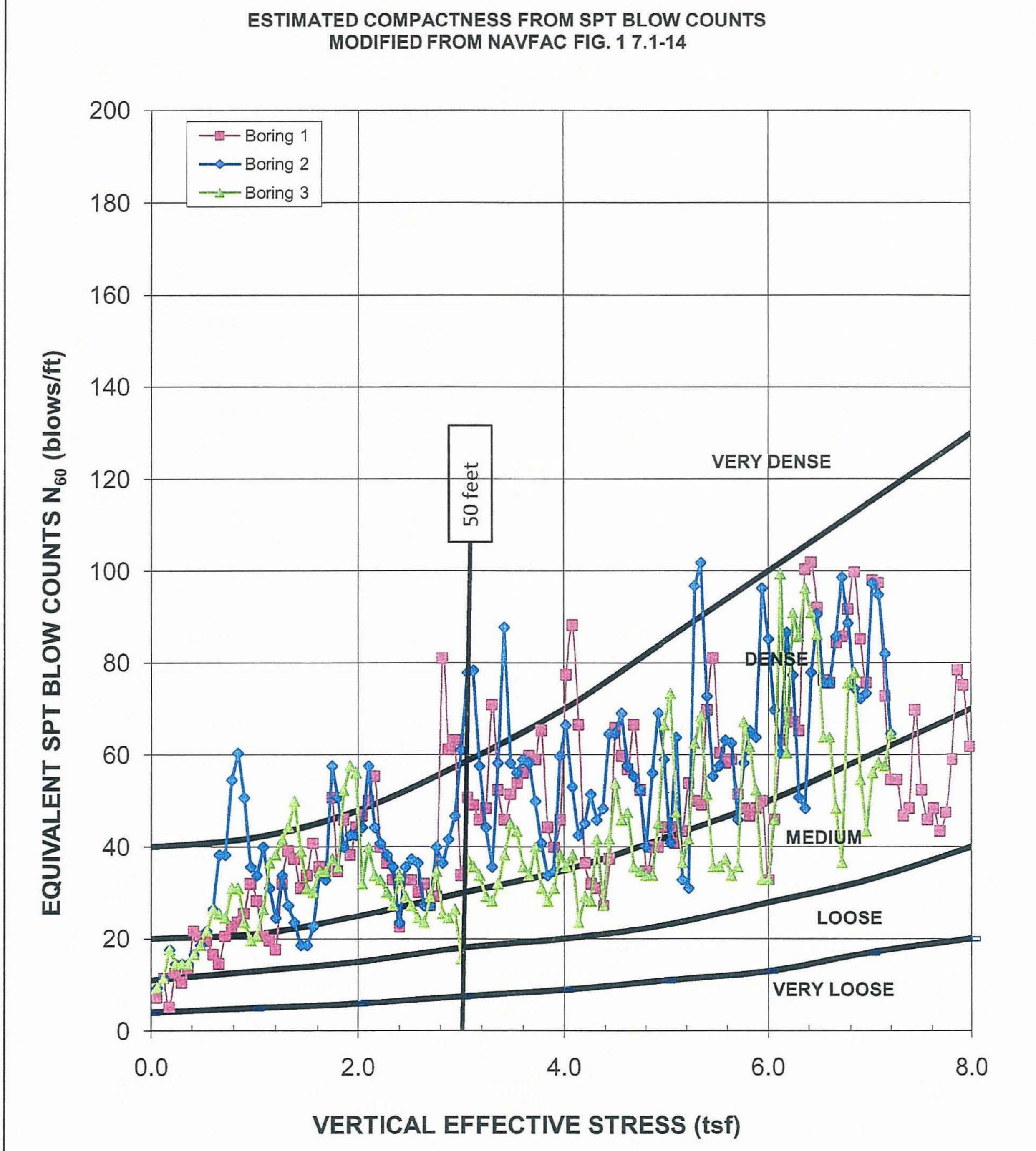
IRVINE GEOTECHNICAL Inc

NAVFAC DENSITY

IC: 07034 CONSULT: JAI

CLIENT: IC 07034-I NU-WAY

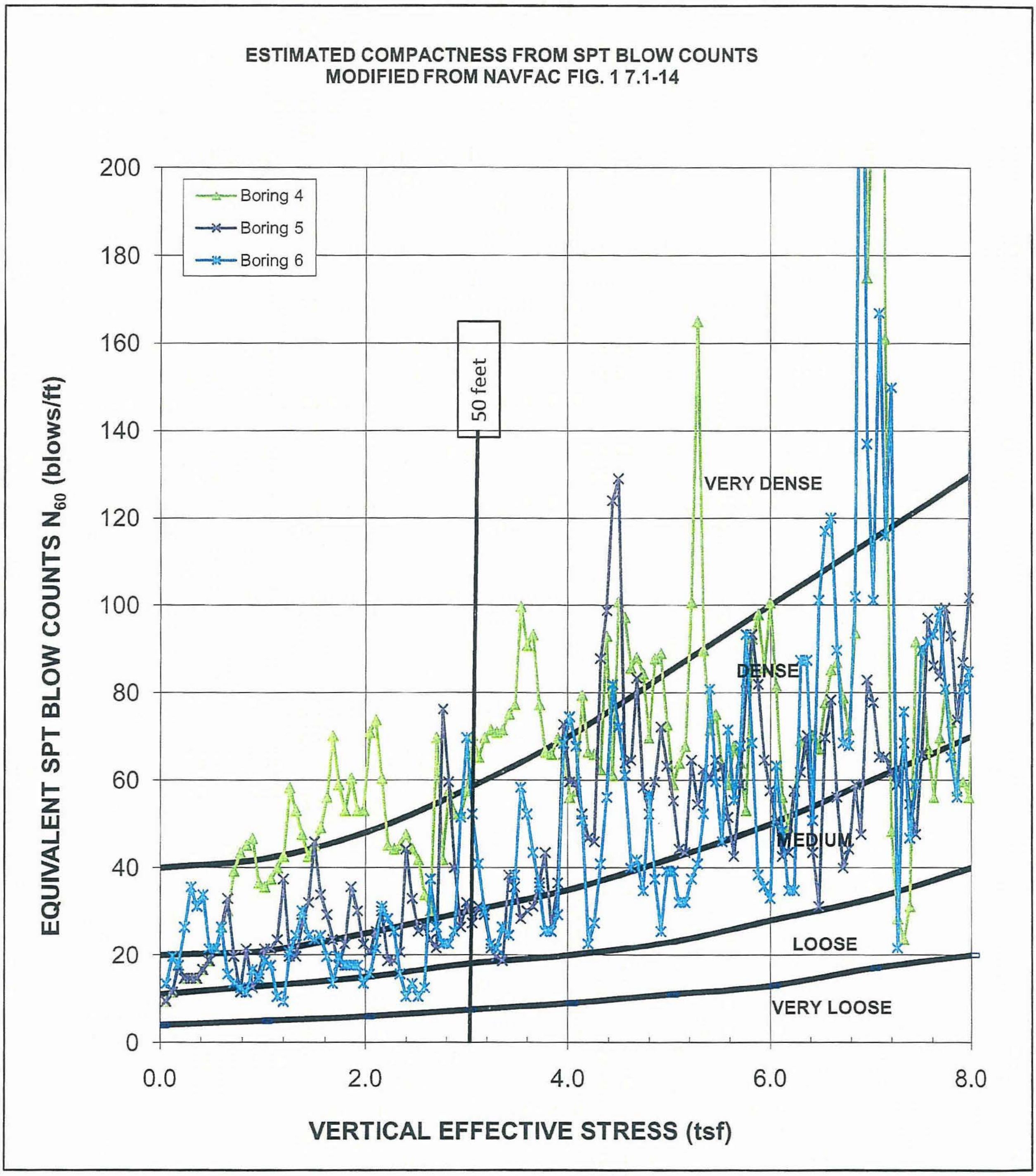
GRAPH # 1 - Borings 1 - 3



IRVINE GEOTECHNICAL Inc

NAVFAC DENSITY

IC: 07034 CONSULT: JAI
CLIENT: IC 07034-I NU-WAY
GRAPH # 1 - Borings 4 - 6



GEOTECHNICAL BORING LOG

SHEET 1 OF 1

PROJECT NAME	Nu-Way				
PROJECT NO	90346-00			BORING DESIG.	B-1
DATE STARTED	09/13/90	DATE FINISHED	09/13/90	STATION	
SUBCONTRACTOR	Layne Environmental	LOGGED BY	ALB	DIAMETER	6"
GROUND WATER ELEV		GW DEPTH (FT)		GSE	282.0
TYPE OF DRILL RIG	Rotary Wash	DRIVE WT (LBS)	140	DROP	12"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0							**ARTIFICIAL FILL (Uncertified) Afu:** @ 0' - 5': Silty Sand, tan-gray, dry, loose, fine to medium-grained, trace pebbles (up to 1-inch diameter), beach type sand.				SM
	280										
5			X	3/5							
	275										
10			X	5/7			@ 10': Same as above, moist, firm	21.0	82.7		
	270										
15			X	6/8			@ 15': Same as at 10', trace rounded cobbles (up to 2-inch diameter).	5.1	95.1		
	265										
20			X	6/10			@ 20': Same as at 15'.	25.1	80.6		
	260										
25			X	7/10			@ 25': Same as at 20', wet to saturated, occasional fine silt and clay laminae (up to 1/16" thick), medium brown, wet, firm.	29.2	93.5		
	255										
30			X	6/10			@ 30': Same as at 20', moderate silt and clay laminae (up to 1/8" thick), medium brown, wet, soft, laminated surfaces.	29.0	90.5		
	250										
35							**ALLUVIUM (Qal):** @ 35': Silty sand, light brown, abundant rounded cobbles Total depth = 40' No groundwater encountered No caving				GM

SAMPLE TYPES:
- C ROCK CORE
- S SPLIT SPOON
- D DRIVE SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE

- GW WHILE DRILLING
- GW HRS.
- B BEDDING PLANE
- J JOINTING
- C CONTACT
- F FAULT
- S SHEAR

ZEISER GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

PROJECT NAME	Nu-Way				
PROJECT NO	90346-00			BORING DESIG.	B-2
DATE STARTED	09/13/90	DATE FINISHED	09/13/90	STATION	
SUBCONTRACTOR	Layne Environmental	LOGGED BY	ALB	DIAMETER	6"
GROUND WATER ELEV		GW DEPTH (FT)		GSE	282.0
TYPE OF DRILL RIG	Rotary Wash	DRIVE WT (LBS)	140	DROP	12"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0							**ARTIFICIAL FILL (Uncertified) Afu:** @ 0' - 5': **Silty sand**, tan-gray, damp, loose to firm, fine to medium grained, beach type sand, trace **silt** laminae (1/16" thick), medium brown, moist, firm.				SM
	280										
5			X	6/7				2.2	92.5		
	275										
10			X	7/8			@ 10': Same as at 5'.	20.4	88.0		
	270										
15			X	9/11			@ 15': Same as at 10'.	19.2	87.3		
	265										
20			X	9/12			@ 20': Same as at 15', damp to moist, loose to firm.	12.5	101.4		
	260										
25			X	10/15			@ 25': Same as at 20'.				
	255										
30			X	11/14				26.9	84.4		
	250										
35											
	245						@ 37': Minor cobbles.				
40											

SAMPLE TYPES:
- C ROCK CORE
- S SPLIT SPOON
- D DRIVE SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE

- GW WHILE DRILLING
- GW HRS.
- B BEDDING PLANE
- J JOINTING
- C CONTACT
- F FAULT
- S SHEAR

ZEISER GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

PROJECT NAME	Nu-Way				
PROJECT NO	90346-00			BORING DESIG.	B-2
DATE STARTED	09/13/90	DATE FINISHED	09/13/90	STATION	
SUBCONTRACTOR	Layne Environmental	LOGGED BY	ALB	DIAMETER	6"
GROUND WATER ELEV		GW DEPTH (FT)		GSE	282.0
TYPE OF DRILL RIG	Rotary Wash	DRIVE WT (LBS)	140	DROP	12"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
40			X	13/12			@ 40': **Silty sand**, tan-gray, wet to saturated, firm, fine to medium grained, beach type sand.	29.5	97.5		
	240										
45											
	235						**ALLUVIUM (Qal):** @ 46': **Silty sand**, light brown, abundant rounded cobbles.				GM
50							Total depth = 50' No groundwater encountered No caving				

SAMPLE TYPES:

C ROCK CORE	B BULK SAMPLE
S SPLIT SPOON	T TUBE SAMPLE
D DRIVE SAMPLE	

GW WHILE DRILLING	
GW HRS.	C CONTACT
B BEDDING PLANE	F FAULT
J JOINTING	S SHEAR

ZEISER
GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

SHEET 1 OF 1

PROJECT NAME	Nu-Way			BORING DESIG.	B-3
PROJECT NO	90346-00				
DATE STARTED	09/18/90	DATE FINISHED	09/18/90	STATION	
SUBCONTRACTOR	Layne Environmental	LOGGED BY	MJH	DIAMETER	6"
GROUND WATER ELEV		GW DEPTH (FT)		GSE	281.0
TYPE OF DRILL RIG	CME 750	DRIVE WT (LBS)	140	DROP	30"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0	280						**ARTIFICIAL FILL (Uncertified) Afu:** @ 0 - 2.5': Sandy Clay, light yellow-brown, dry/dessicated, shrinkage cracks to 8"W x 2.5D.				CL
							@ 2.5': Hard drilling (boulder/cobble) to 4.5'.				GP
5	275		1	12/15			**ALLUVIUM (Qal):** @ 5': Silty sand and sandy clay, light yellow-brown and medium brown, dry to slightly moist, friable to soft. @ 5' - 31': Spoil is almost entirely coarse gravel and small cobbles to 4" diameter, well-rounded.				SM/CL
10	270		2	10/17			@ 10': Gravelly sand, slightly silty, medium orange-brown, slightly moist to moist, loose, cohesionless, fine to medium grained, gravel to 2+" diameter.	2.4	117.3		GP
							@ 13': Hard drilling to 14'.				
15	265		3	38/50-2"			@ 15': 2" X 3" clast blocking sampler, with fine to medium grained,medium brown sand.				
							- Hard drilling.				
20	260		4	32/55			@ 20': Gravelly sand (slightly silty), medium brown, fine to medium grained, moist, loose to medium dense, cohesionless, sub-rounded to well-rounded fine to medium gravel, 1" to 2" diameter.	4.6	128.6		
25	255			48/45			@ 25': No recovery.				
				30/44			@ 26': No recovery.				
							@ 27': Easy drilling to 30'.				
30	250		5	21/34			@ 30': Gravelly sand, slightly silty, medium brown, moist, loose, cohesionless, cobbles and gravel are well-rounded to very well-rounded.	3.9	122.7		

Total depth = 31'
No groundwater encountered
No caving

SAMPLE TYPES:
- [C] ROCK CORE
- [B] BULK SAMPLE
- [S] SPLIT SPOON
- [T] TUBE SAMPLE
- [D] DRIVE SAMPLE

- GW WHILE DRILLING
- GW HRS.
- [B] BEDDING PLANE
- [J] JOINTING
- [C] CONTACT
- [F] FAULT
- [S] SHEAR

ZEISER GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

SHEET 1 OF 1

PROJECT NAME	Nu-Way				
PROJECT NO	90346-00			BORING DESIG.	B-4
DATE STARTED	09/17/90	DATE FINISHED	09/17/90	STATION	
SUBCONTRACTOR	Layne Environmental	LOGGED BY	MJH	DIAMETER	6"
GROUND WATER ELEV		GW DEPTH (FT)		GSE	280.0
TYPE OF DRILL RIG	CME 750	DRIVE WT (LBS)	140	DROP	30"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0	280						**ARTIFICIAL FILL (Uncertified) Afu:** @ 0' - 30.5': Silty clay with sand, medium gray-brown, dry to wet, hard to very soft, friable where dry, plastic where wet, shrinkage cracks to 8"W X 3'D, uppermost 6" is dry, very well layered with fine laminations.				ML/CL
5	275		1	1-12"			@ 5': Clay, medium brown, with silt, wet, soft, plastic to liquid, with sand, medium gray, fine gravel, very moist, medium dense (in tip)	20.7	89.1		CL/ML
10	270		2	3/3			@ 10': Clay with sand, as above at 5': clay is slightly redder in color, sand and clay are interlayered (laminae approximately 1"+ thickness).	21.6	88.9		CL/SC
15	265		3	6/7			@ 15': Sand, medium brown-gray, moist, loose to moderately dense, fine grained, micaceous. @ 17': Perched water.	3.9	97.1		SM
20	260		4	7/9			@ 20': Sand, as above at 15'.	26.5	99.9		
25	255		5	6/16			@ 25': Clay with sand, as above at 10'.	42.4	76.2		CL/SC
30	250		6	30/53			@ 30': Clay, as above at 10', with sandy gravel at tip, wet, fine to medium grained, loose with well-rounded gravel. **ALLUVIUM** @ 30.5': Sandy gravel, fine to medium grained, wet, loose. Total depth = 31 water encountered at 17' No caving	47.5	75.4		GP

SAMPLE TYPES:
- [C] ROCK CORE
- [S] SPLIT SPOON
- [D] DRIVE SAMPLE
- [B] BULK SAMPLE
- [T] TUBE SAMPLE

- GW WHILE DRILLING
- GW HRS.
- [B] BEDDING PLANE
- [J] JOINTING
- [C] CONTACT
- [F] FAULT
- [S] SHEAR

ZEISER GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

SHEET 1 OF 1

PROJECT NAME	Nu-Way				
PROJECT NO	90346-00			BORING DESIG.	B-5
DATE STARTED	09/18/90	DATE FINISHED	09/18/90	STATION	
SUBCONTRACTOR	Layne Environmental	LOGGED BY	MJH	DIAMETER	6"
GROUND WATER ELEV		GW DEPTH (FT)		GSE	281.0
TYPE OF DRILL RIG	CME 750	DRIVE WT (LBS)	140	DROP	30"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0							**ARTIFICIAL FILL (Uncertified) Afu:**				
	280										
5	275		1	1-12"			@ 5': Sandy Clay to Clayey Sand, dark brown, very moist to wet, soft, plastic to liquid consistency.	52.1	69.3		CL/SC
10	270		2	1-12"			@ 10': Clay, with silt, medium brown, wet, soft, plastic.	62.6	66.8		CL/ML
							ALLUVIUM (Qal) @ 12': Harder drilling, gravel in spoil.				GP/SP
15	265		3	75-6"			@ 15': Sandy gravel to gravelly sand, wet, loose to moderately dense, medium to coarse sand with fine to medium gravel, occasional clasts to 3" diameter.	4.9	121.9		
							Total depth = 16' No groundwater encountered No caving				

SAMPLE TYPES:
- C ROCK CORE
- S SPLIT SPOON
- D DRIVE SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE

- GW WHILE DRILLING
- GW HRS.
- B BEDDING PLANE
- J JOINTING
- C CONTACT
- F FAULT
- S SHEAR

ZEISER GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

SHEET 1 OF 1

PROJECT NAME	Nu-Way				
PROJECT NO	90346-00			BORING DESIG.	B-6
DATE STARTED	09/18/90	DATE FINISHED	09/18/90	STATION	
SUBCONTRACTOR	Layne Environmental	LOGGED BY	MJH	DIAMETER	6"
GROUND WATER ELEV		GW DEPTH (FT)		GSE	282.0
TYPE OF DRILL RIG	CME 750	DRIVE WT (LBS)	140	DROP	30"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0							**ARTIFICIAL FILL (Uncertified) Afu:**				
	280						@ 0' - 20': Sand, light yellow-brown, dry to moist, loose, very fine to fine grained, very clean beach-type sand, micaceous.				
5			1	5/6			@ 5': Sand as above.	2.9	90.2		SM
	275										
10			2	2/3			@ 10': Silty sand, medium gray-brown, very moist, loose to medium fine grained, with interlayered clayey silt laminae, 1/10" thick, micaceous.	30.2	91.1		
	270										
15			3	1/2			@ 15': No recovery, trace fine sand in sampler.				
	265										
20			4	3/3			@ 20': Silty clay, medium brown, very moist to wet, soft, plastic.	44.0	78.2		CL
	260										
25			5	3/3			@ 25': Clay, as above at 20', with lenses of fine sand, dark gray, micaceous, clean, loose to medium, very moist.	46.2	75.2		CL/SC
	255						**ALLUVIUM (Qal)**				
							@ 27': Harder drilling.				
30			6	24/28			@ 30': Cobble blocking sampler - no recovery.	6.8	129.2		GP/SP
	250										
35							@ 35': Sandy gravel/gravelly sand, medium orange-brown, very moist, loose to moderately dense, medium to coarse sand with fine to medium well-rounded gravel, clasts to 2" diameter.				
							Total depth = 36' No groundwater encountered No caving				

SAMPLE TYPES:
- [C] ROCK CORE
- [B] BULK SAMPLE
- [S] SPLIT SPOON
- [T] TUBE SAMPLE
- [D] DRIVE SAMPLE

- GW WHILE DRILLING
- GW HRS.
- [B] BEDDING PLANE
- [J] JOINTING
- [C] CONTACT
- [F] FAULT
- [S] SHEAR

ZEISER
GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

SHEET 1 OF 1

PROJECT NAME: Nu-Way
PROJECT NO: 90346-00
DATE STARTED: 09/18/90
SUBCONTRACTOR: Layne Environmental
GROUND WATER ELEV:
TYPE OF DRILL RIG: CME 750

DATE FINISHED: 09/18/90
LOGGED BY: MJH
GW DEPTH (FT):
DRIVE WT (LBS): 140

BORING DESIG.: B-7
STATION:
DIAMETER: 6"
GSE: 288.0
DROP: 30"

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0							**ARTIFICIAL FILL (Uncertified) Afu:** @ 0 - 25': Silty clay to clayey silt, dark gray to black, moist, very plastic, with trace sand.				CL/ML
	285										
5			1	28/50			@ 5': Silty sand, with gravel, dark red-brown, slightly moist to moist, low plasticity, gravel to 1/2", well rounded.	18.0	108.6		
	280										
10			2	4/5			@ 10': Clayey sand to clayey silt, dark gray-green, moist, firm, non-plastic.	25.1	94.0		SC/ML
	275										
15			3	7/9			@ 15': Sandy silt with clay, dark green-gray, moist, firm, non-plastic.	25.3	94.6		ML/CL
	270										
20			4	5/7			@ 20': Sandy silt as above at 15'.	43.3	78.9		
	265										
25			5	17/28			@ 25': Upper 6": sandy silt as above 20'. Lower 6": medium grained sand, medium orange-brown, poorly graded, clean, very moist.	37.1	84.5		SM GP/ML
	260						**ALLUVIUM (Qal)** @ 26': Hard drilling.				
30			6	30/15			@ 30': Gravelly sand, medium to coarse grained, medium orange-brown, very moist, moderately dense, well-rounded clasts, with interbedded clayey sandy silt, very moist, firm, well-bedded.	4.3	117.7		
	255										
35			7	50-4"			@ 35': Cobble blocking sampler tip, trace gravel with coarse sand in sampler, very moist.	21.0	104.3		
							Total depth = 36' No groundwater encountered No caving				

SAMPLE TYPES:
C ROCK CORE
S SPLIT SPOON
D DRIVE SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

GW WHILE DRILLING
GW HRS.
B BEDDING PLANE
J JOINTING
C CONTACT
F FAULT
S SHEAR

ZEISER
GEOTECHNICAL, Inc.

GEOTECHNICAL BORING LOG

PROJECT NAME	Nu-Way			BORING DESIG.	B-8
PROJECT NO	90346-00			STATION	
DATE STARTED	09/19/90	DATE FINISHED	09/19/90	DIAMETER	6"
SUBCONTRACTOR	Layne Environmental	LOGGED BY	MJH	GSE	281.0
GROUND WATER ELEV		GW DEPTH (FT)		DROP	30"
TYPE OF DRILL RIG	CME 750	DRIVE WT (LBS)	140		

DEPTH FEET	ELEV	SAMPLE TYPE	SAMPLE	BLOWS/FT OR SCR/RQD	GRAPHIC LOG	ATTITUDES	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT%	DRY PCF DENSITY	SHEAR STRENGTH PARAMETER	GEOLOGIC NOTES
0							**ARTIFICIAL FILL (Uncertified) Afu:**				GC
	280						@ 0' - 11': Sand with gravelly cobbles and boulders, crushed concrete and brick with sand, silt and trace clay matrix, light gray-tan color, dry to slightly moist, loose, hard drilling.				
5	275										
10	270						@ 11': Sandy, silty and gravelly clay, very moist to wet, dark brown, soft, plastic.				
15				3/4			@ 15': No recovery.	47.2	76.5		CL
	265		1	2/2			@ 16': Silty sandy clay, dark green-brown, very moist to wet, soft, plastic when dried, with trace gravel to 1".				
20	260		2	21/45			@ 20': Gravelly clay, dark brown, very moist to wet, very soft, plastic when dried, gravel to 2", very well-rounded clasts, trace gravel in tip.	34.7	85.0		CL/GC
25	255		3	4/5			@ 25': Clay with fine sand and trace gravel, very moist, soft, plastic.	33.2	86.7		
30	250		4	23/26			@ 30': Gravelly clay, as above at 25'.	18.3	107.7		
							@ 32': Refusal				
							Total depth = 32' No groundwater encountered No caving				

SAMPLE TYPES:
- [C] ROCK CORE
- [S] SPLIT SPOON
- [D] DRIVE SAMPLE
- [B] BULK SAMPLE
- [T] TUBE SAMPLE

- GW WHILE DRILLING
- GW HRS.
- [B] BEDDING PLANE
- [J] JOINTING
- [C] CONTACT
- [F] FAULT
- [S] SHEAR

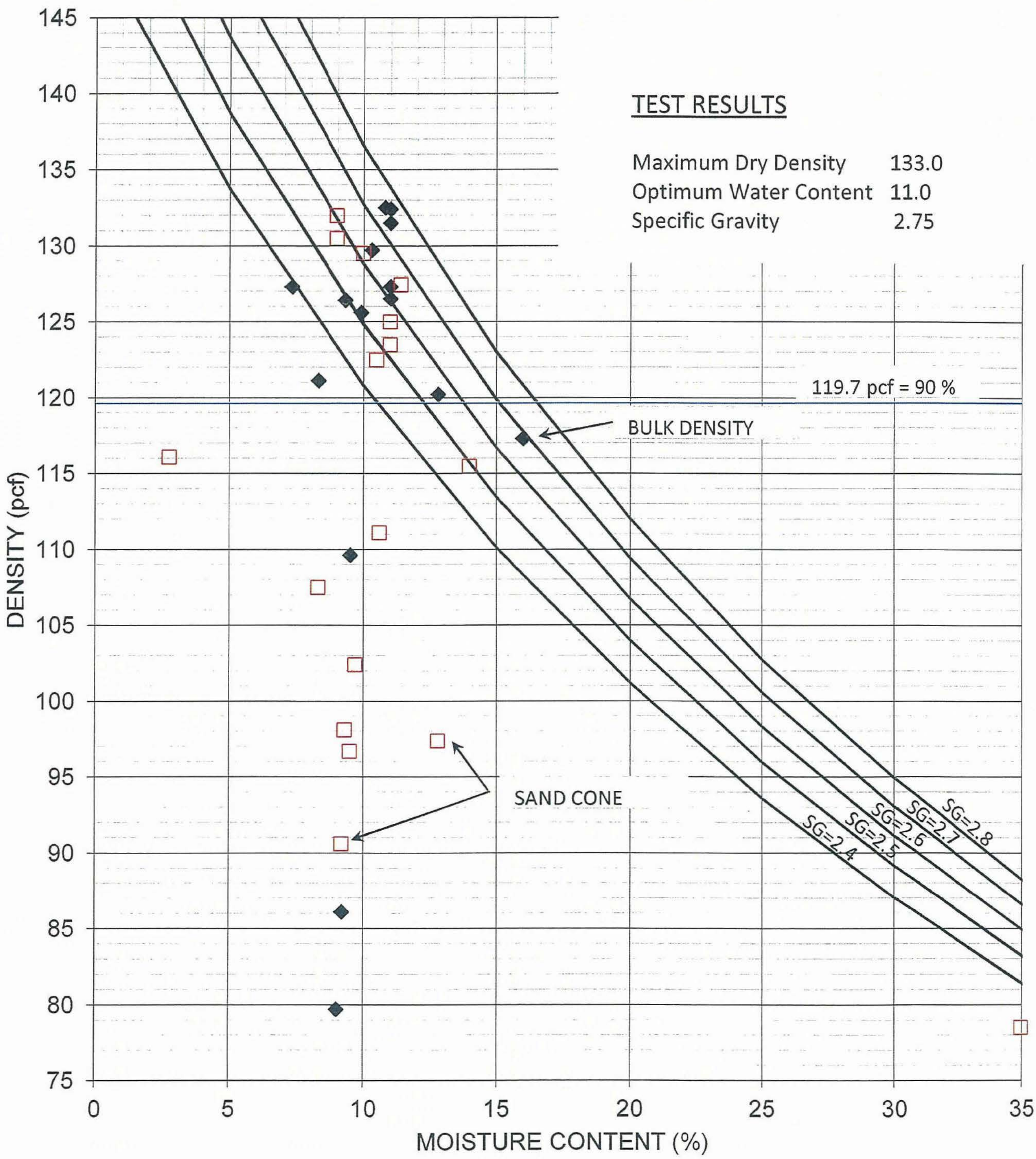
ZEISER GEOTECHNICAL, Inc.

IRVINE
GEOTECHNICAL Inc

MOISTURE-DENSITY RELATIONSHIP

IC: 7034 CONSULTANT: JAI
CLIENT: NU WAY - MNOIAN

EARTH MATERIAL:

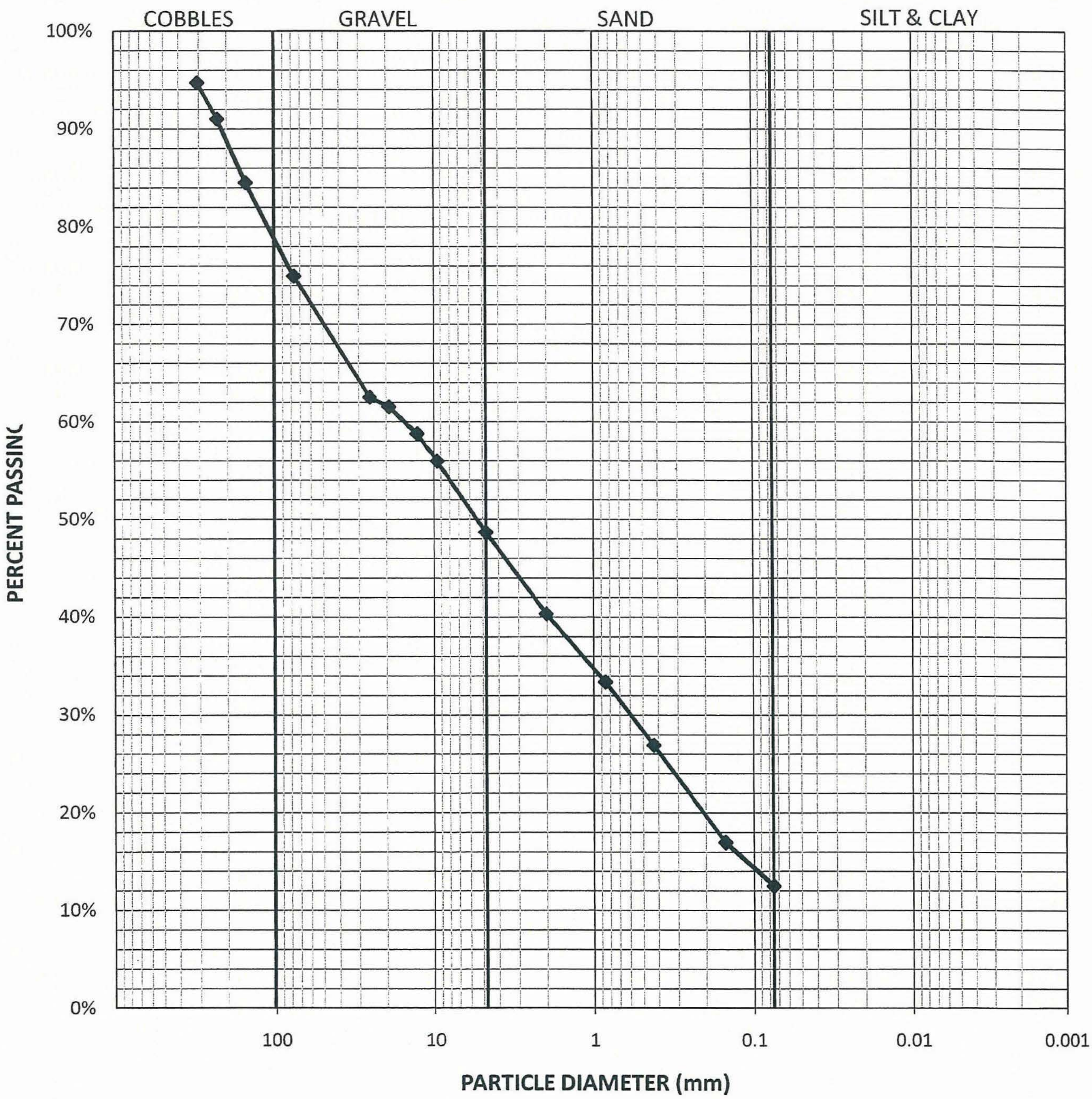


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC: 07034-I CONSULT: JAI
CLIENT NUWAY - MNOIAN
SAMPLE TEST PIT 1 - BENCH 1 PLATE 1

REFERENCE Test Pit 1, Bench 1, Depth = 12 feet (Elev. 397)

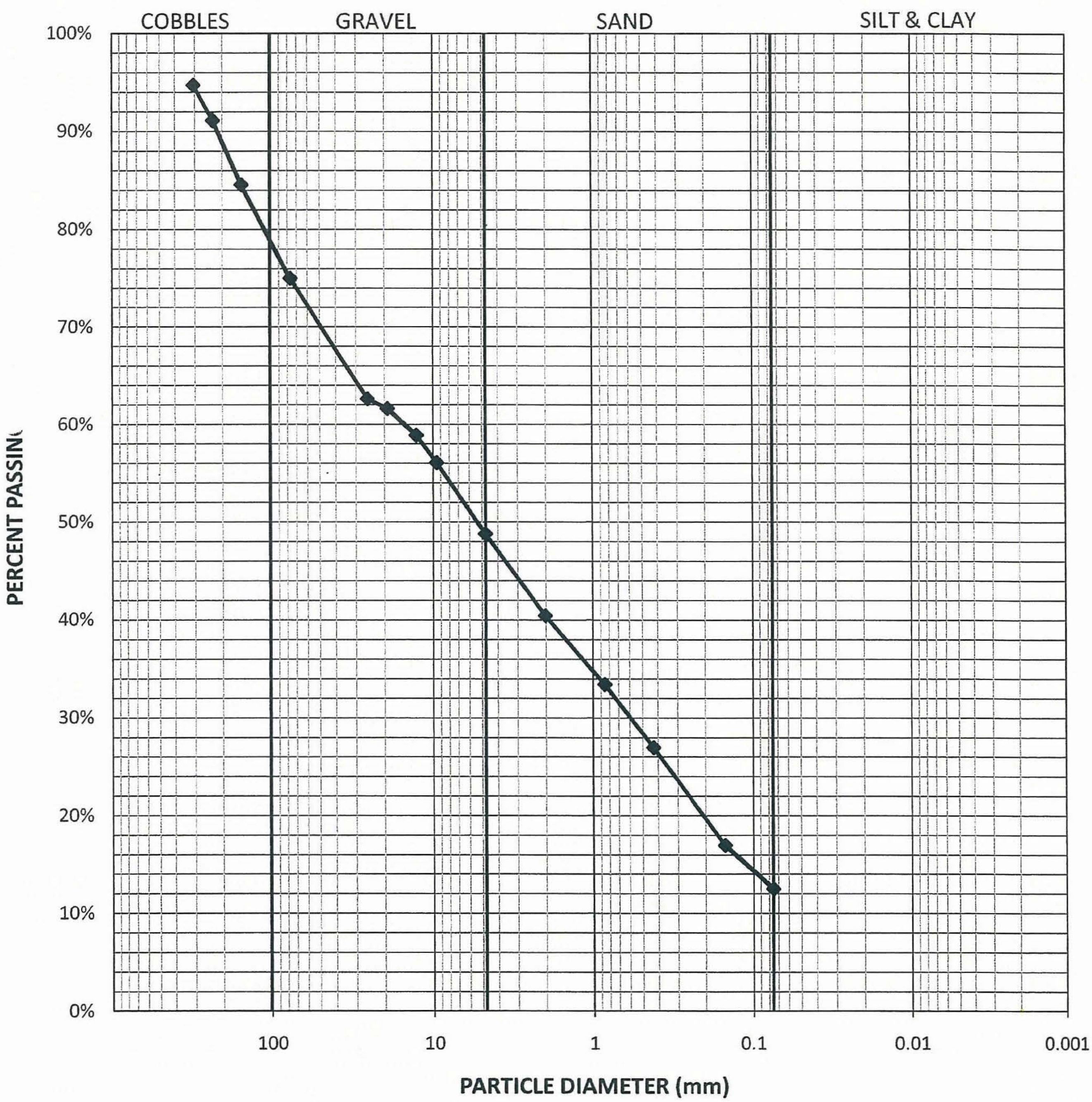


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 1 - BENCH 2		PLATE 2

REFERENCE: Test Pit 1, Bench 2, Depth = 24 feet (Elev. 386)

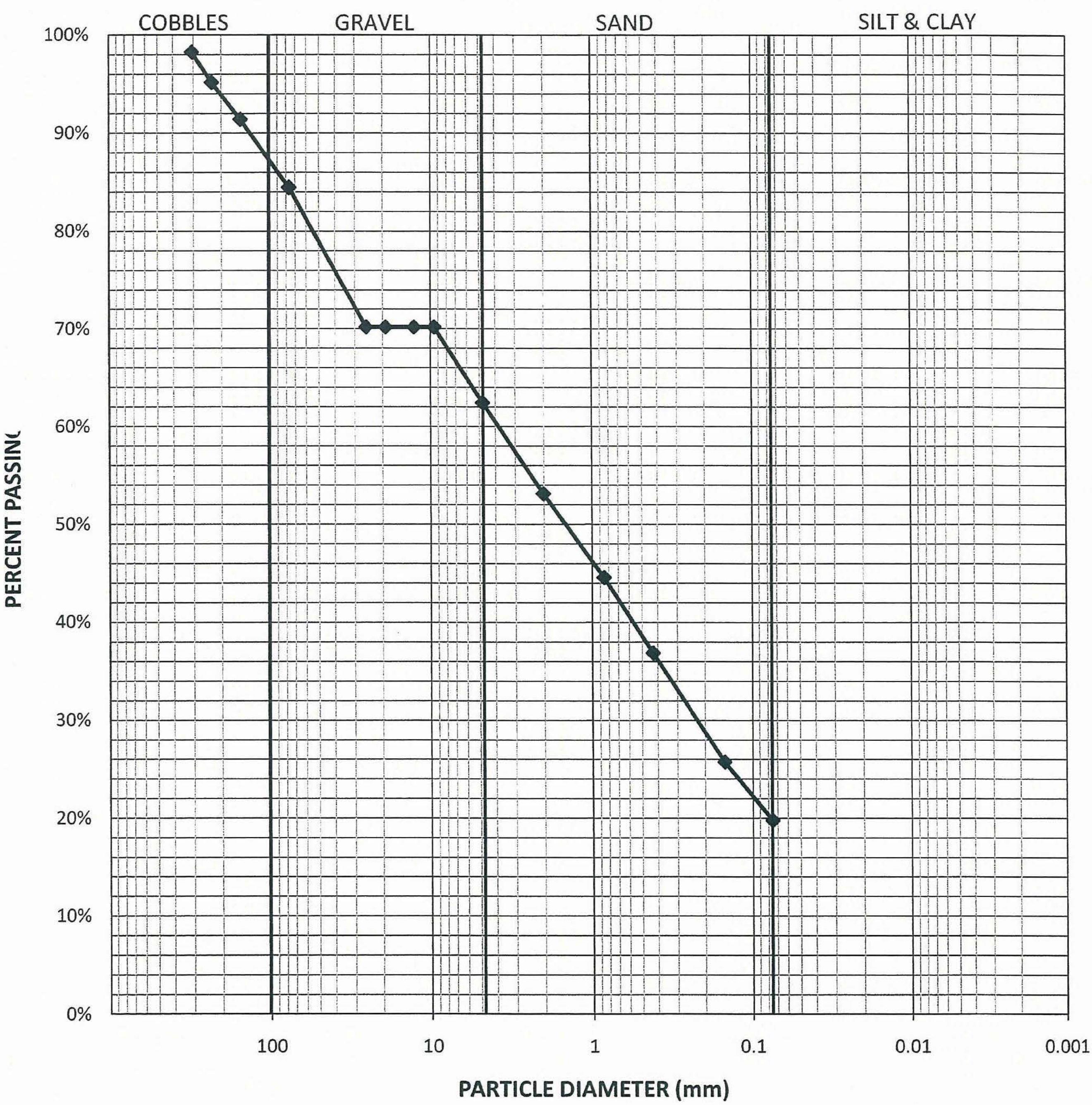


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 1 - BENCH 2		PLATE 3

REFERENCE: Test Pit 1, Bench 2, Depth = 24 feet (Elev. 386) MAD TEST PAD

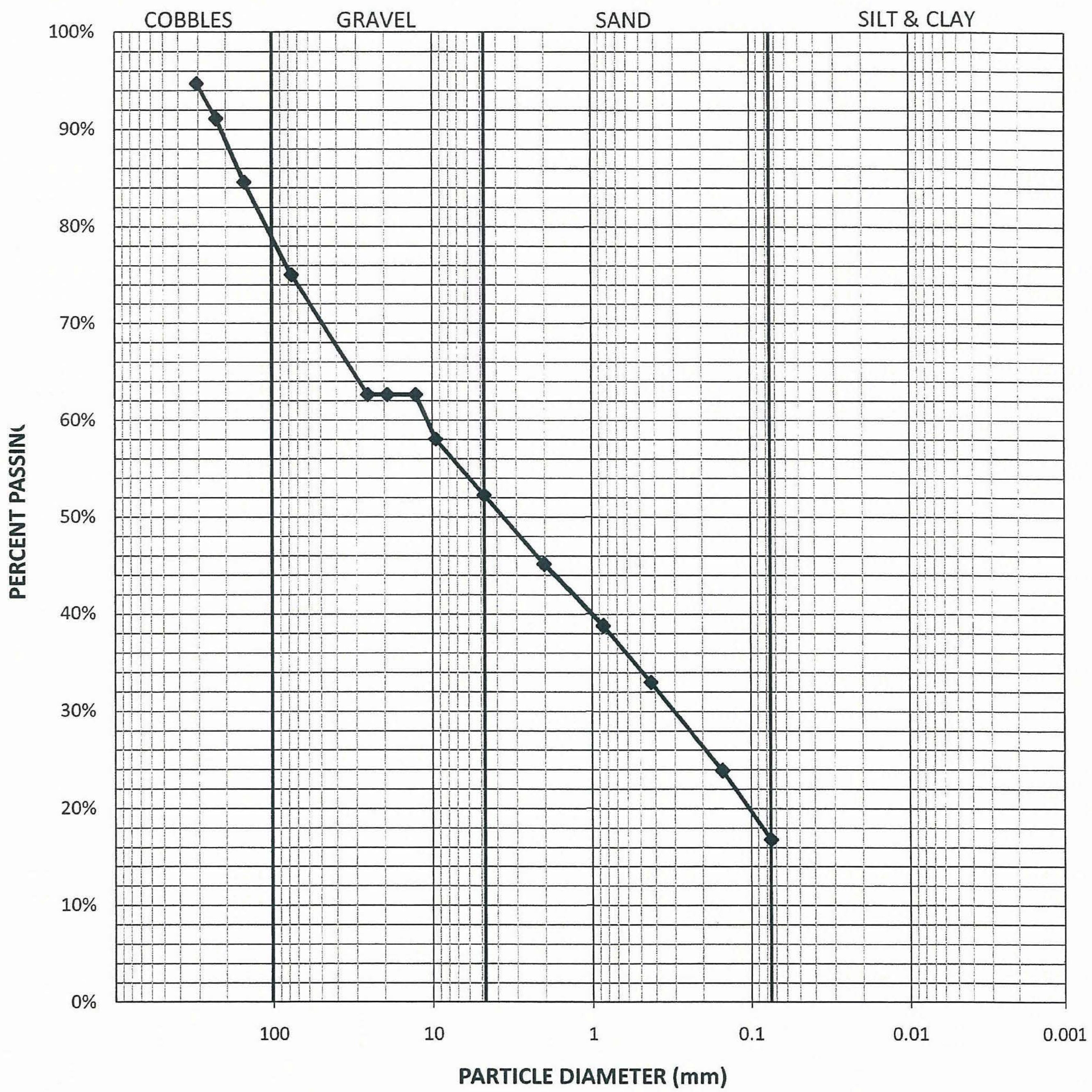


GRAIN SIZE DISTRIBUTION

IRVINE GEOTECHNICAL Inc

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 1 - BENCH 3		PLATE 4

REFERENCE: Test Pit 1, Bench 3, Depth = 32 feet (Elev. 376)

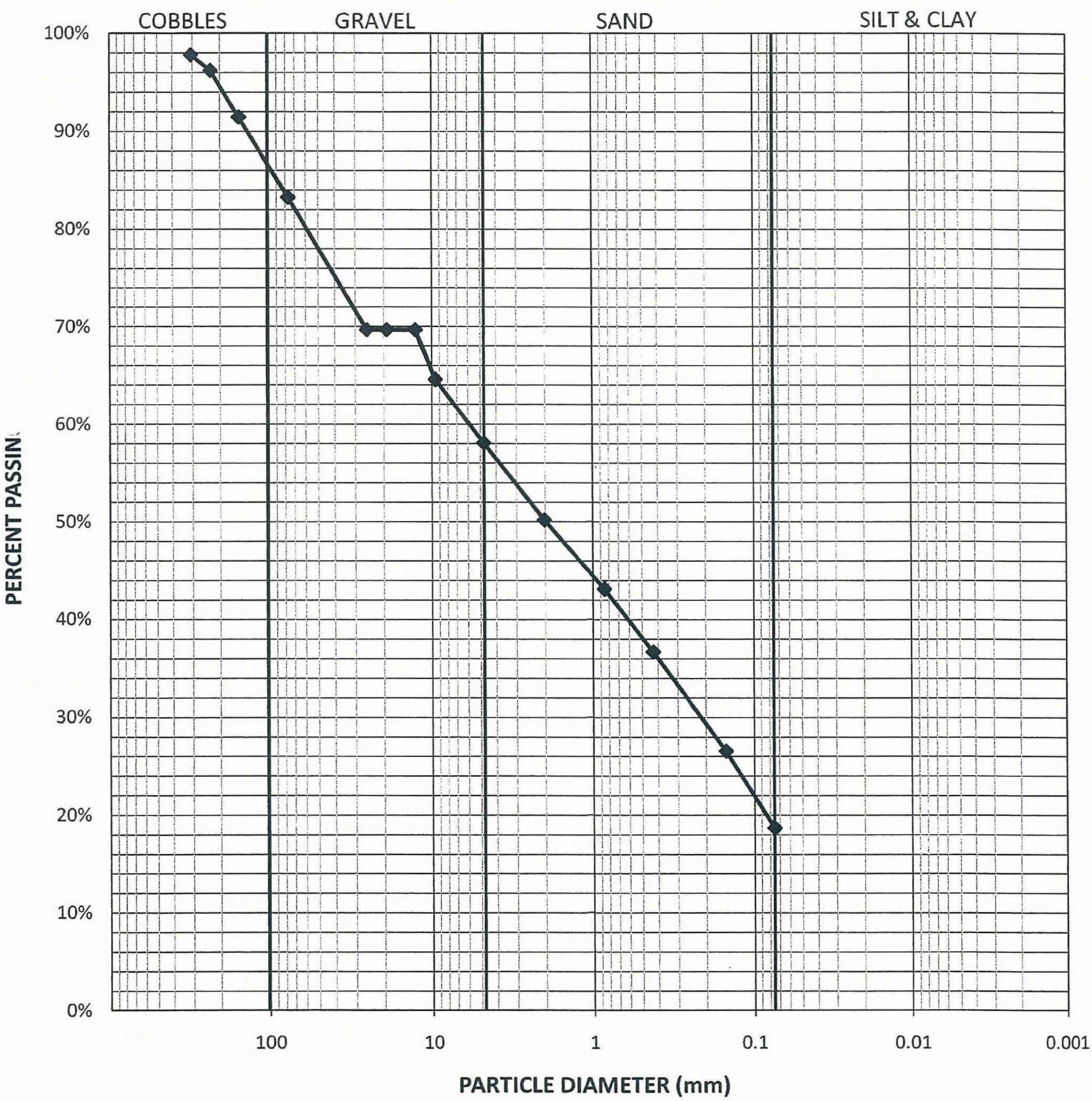


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 1 - BENCH 3		PLATE 5

REFERENCE: Test Pit 1, Bench 3, Depth = 32 feet (Elev. 376) MAD TEST PAD

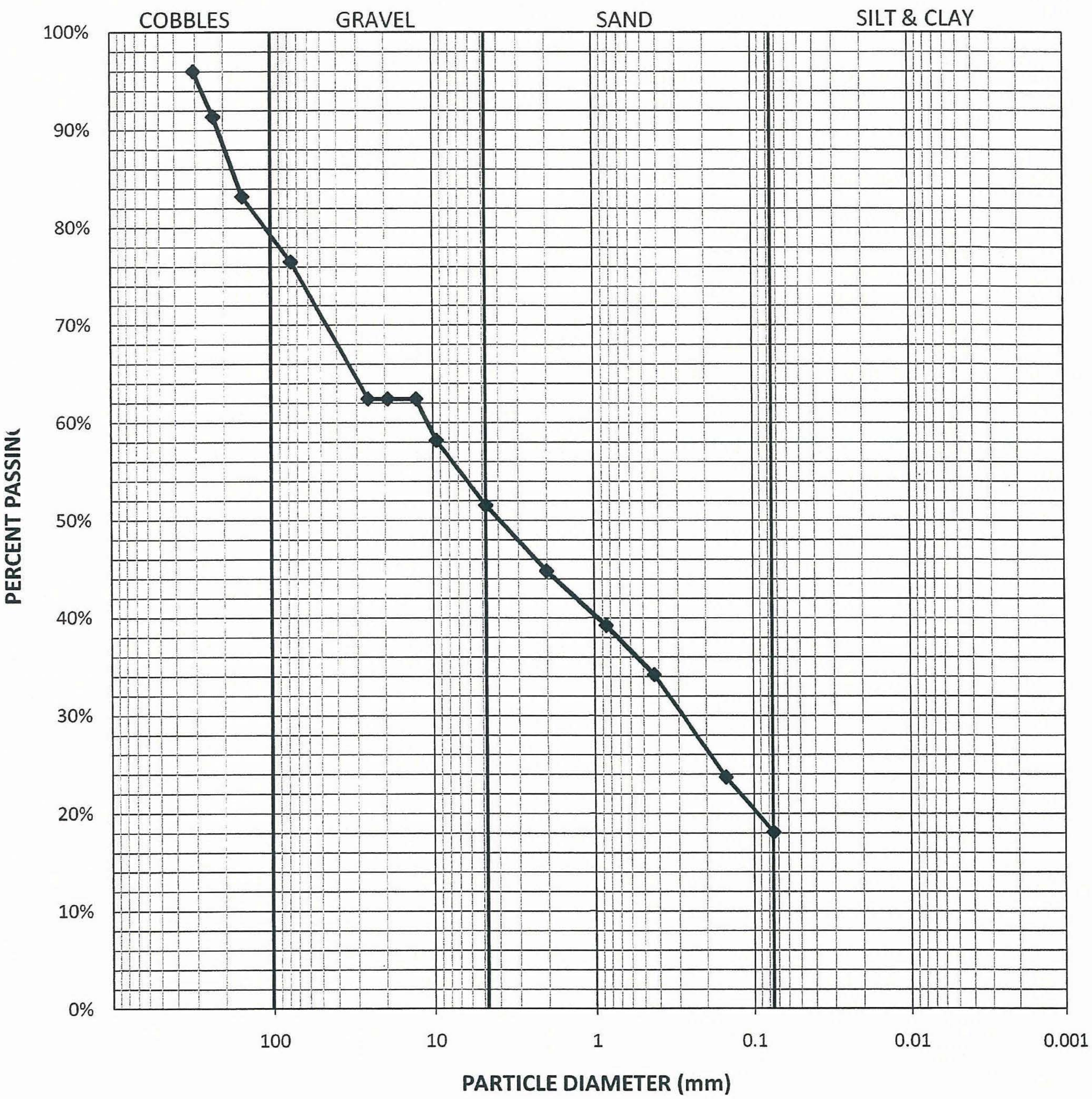


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 1 - BENCH 4		PLATE 6

REFERENCE: Test Pit 1, Bench 4, Depth = 43 feet (Elev. 364)

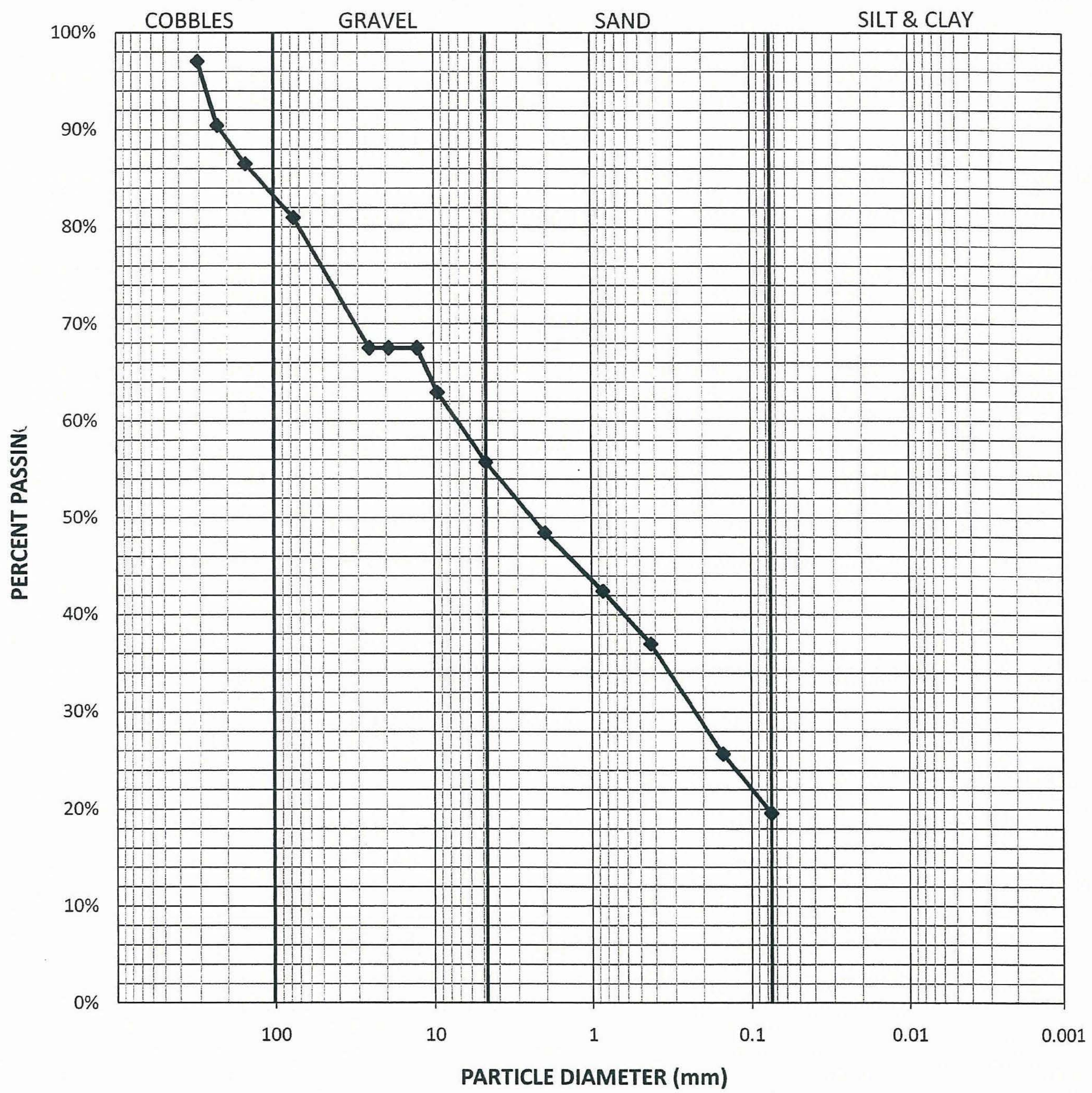


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 1 - BENCH 4		PLATE 7

REFERENCE: Test Pit 1, Bench 4, Depth = 43 feet (Elev. 364) MAD TEST PAD

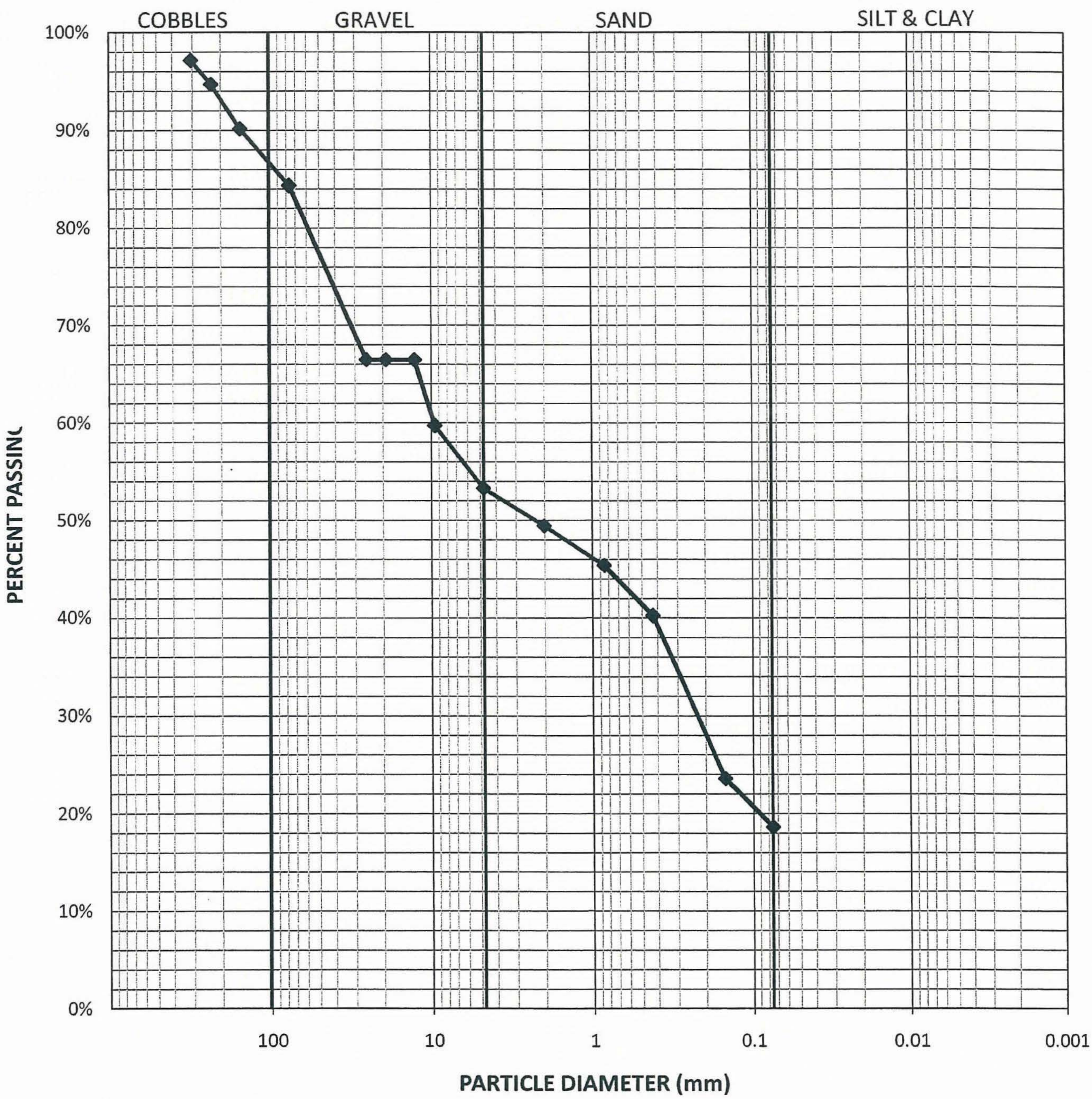


GRAIN SIZE DISTRIBUTION

IRVINE GEOTECHNICAL Inc

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 2 - BENCH 1		PLATE 8

REFERENCE Test Pit 2, Bench 1, Depth = 14 feet (Elev. 391)

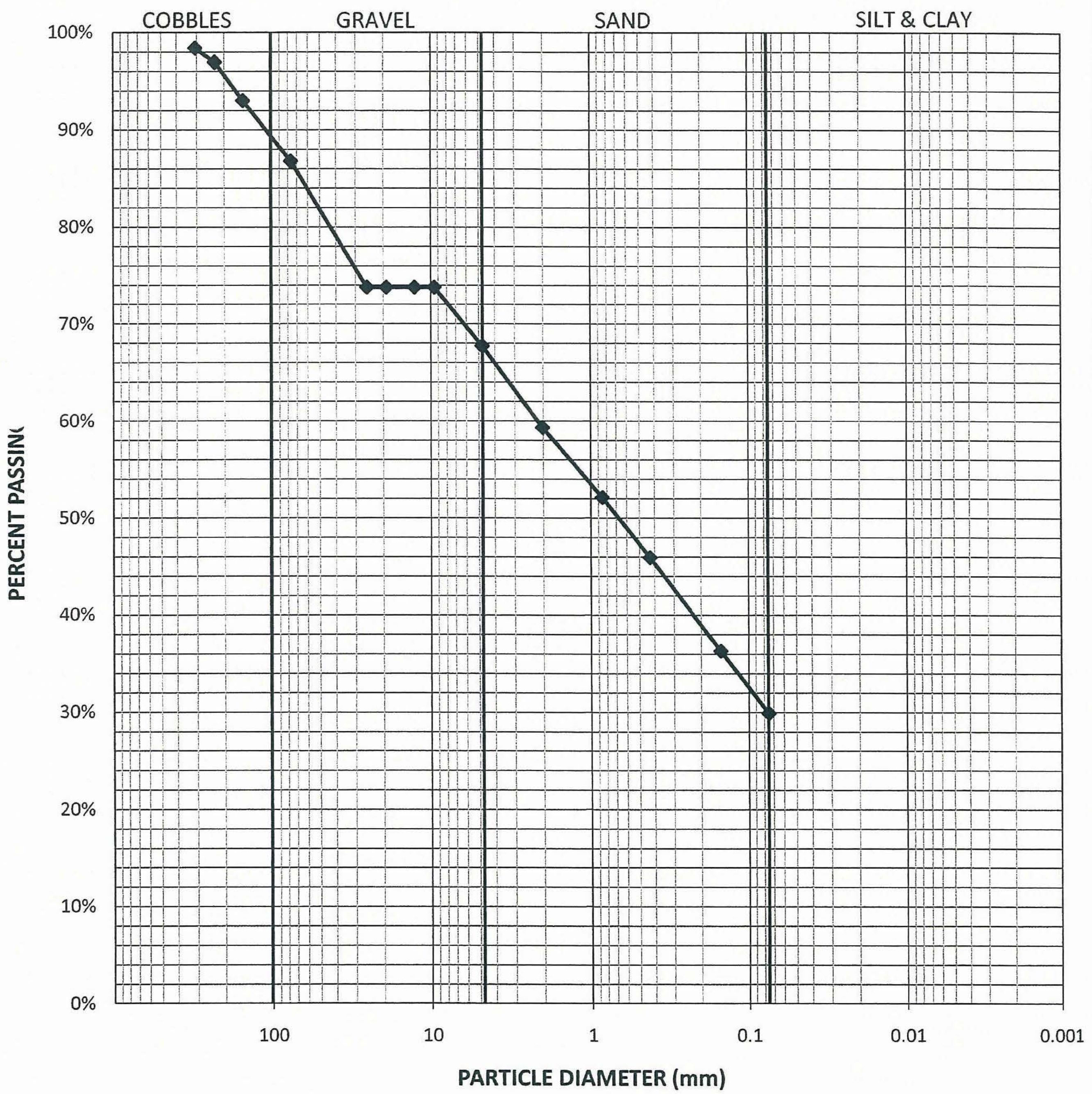


GRAIN SIZE DISTRIBUTION

IRVINE GEOTECHNICAL Inc

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 2 - BENCH 2		PLATE 9

REFERENCE: Test Pit 2, Bench 2, Depth = 24 feet (Elev. 382) MAD TEST PAD

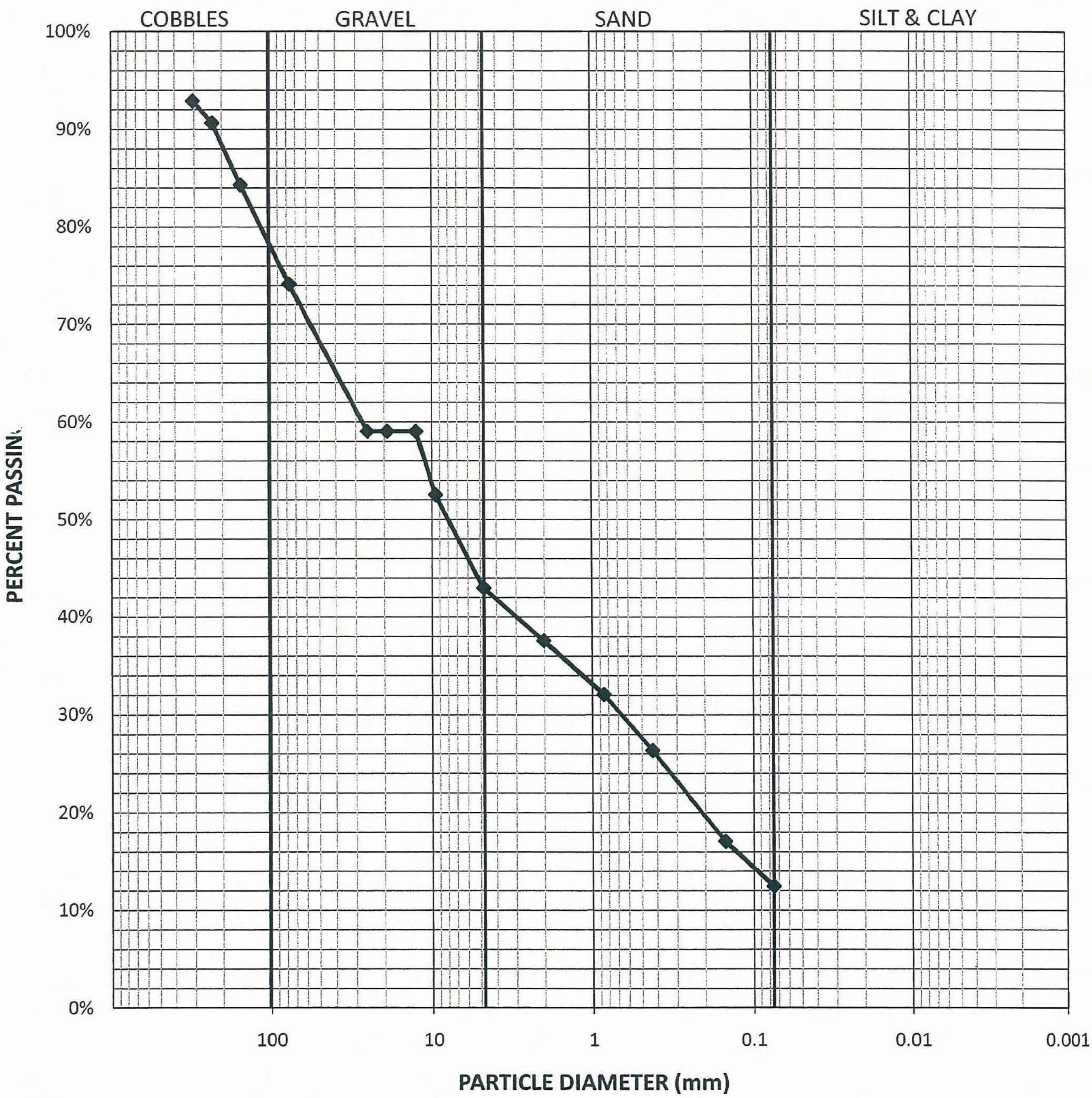


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 2 - BENCH 3		PLATE 10

REFERENCE: Test Pit 2, Bench 3, Depth = 36 feet (Elev. 370)

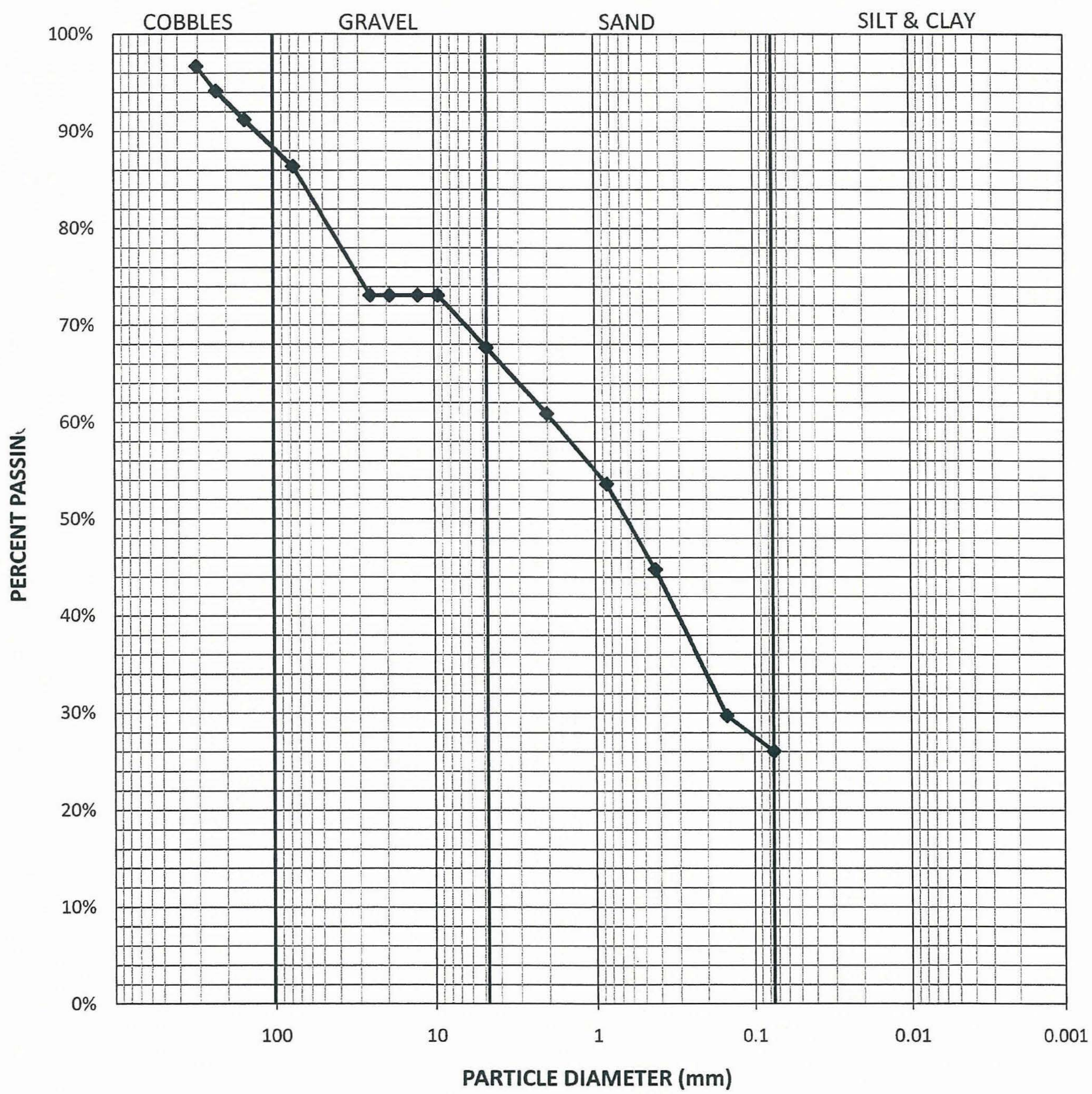


GRAIN SIZE DISTRIBUTION

IRVINE GEOTECHNICAL Inc

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 2 - BENCH 3		PLATE 11

REFERENCE: Test Pit 2, Bench 3, Depth = 36 feet (Elev. 370) MAD TEST PAD

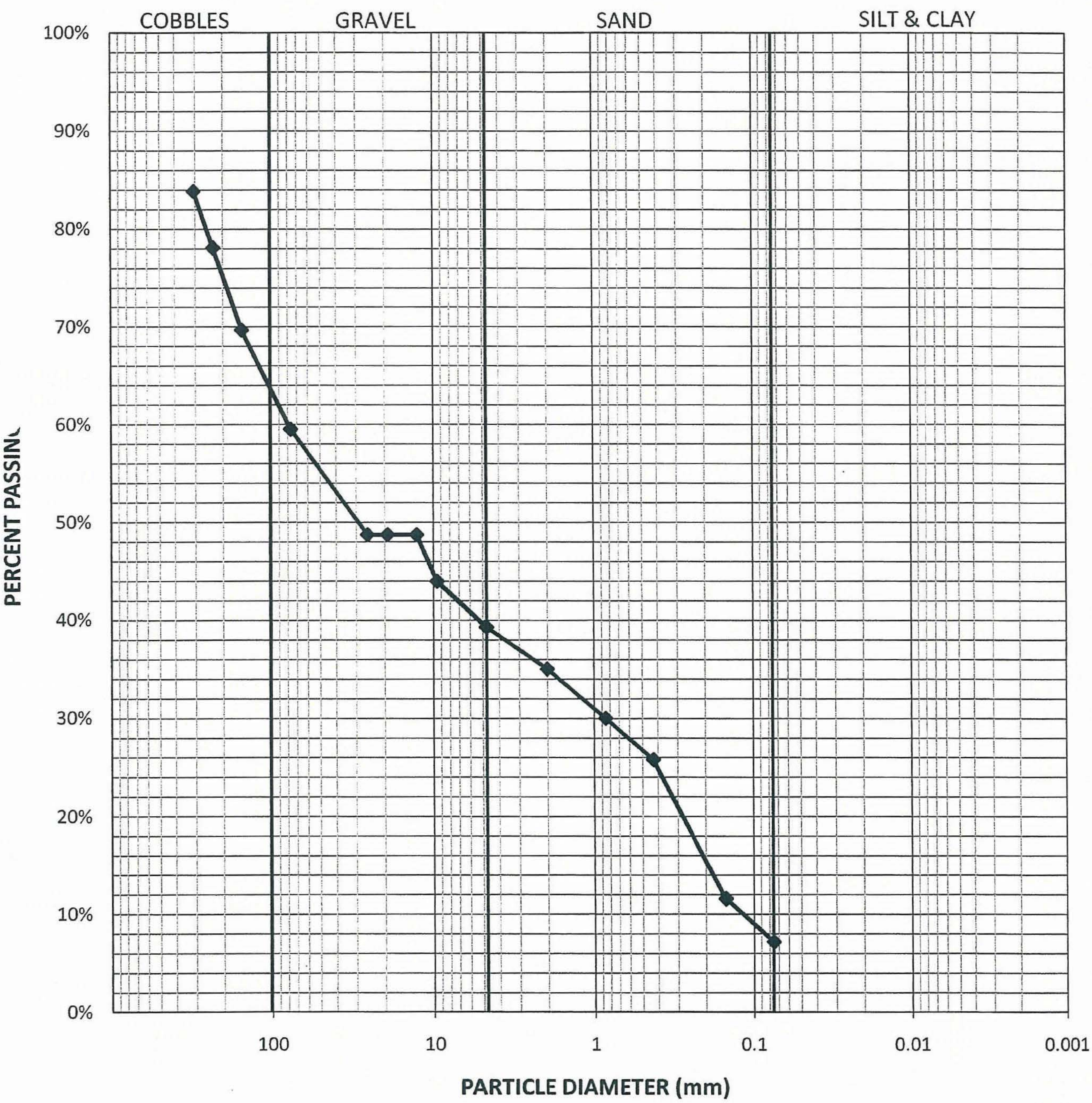


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 2 - BENCH 4		PLATE 12

REFERENCE Test Pit 2, Bench 4, Depth = 50 feet (Elev. 358)

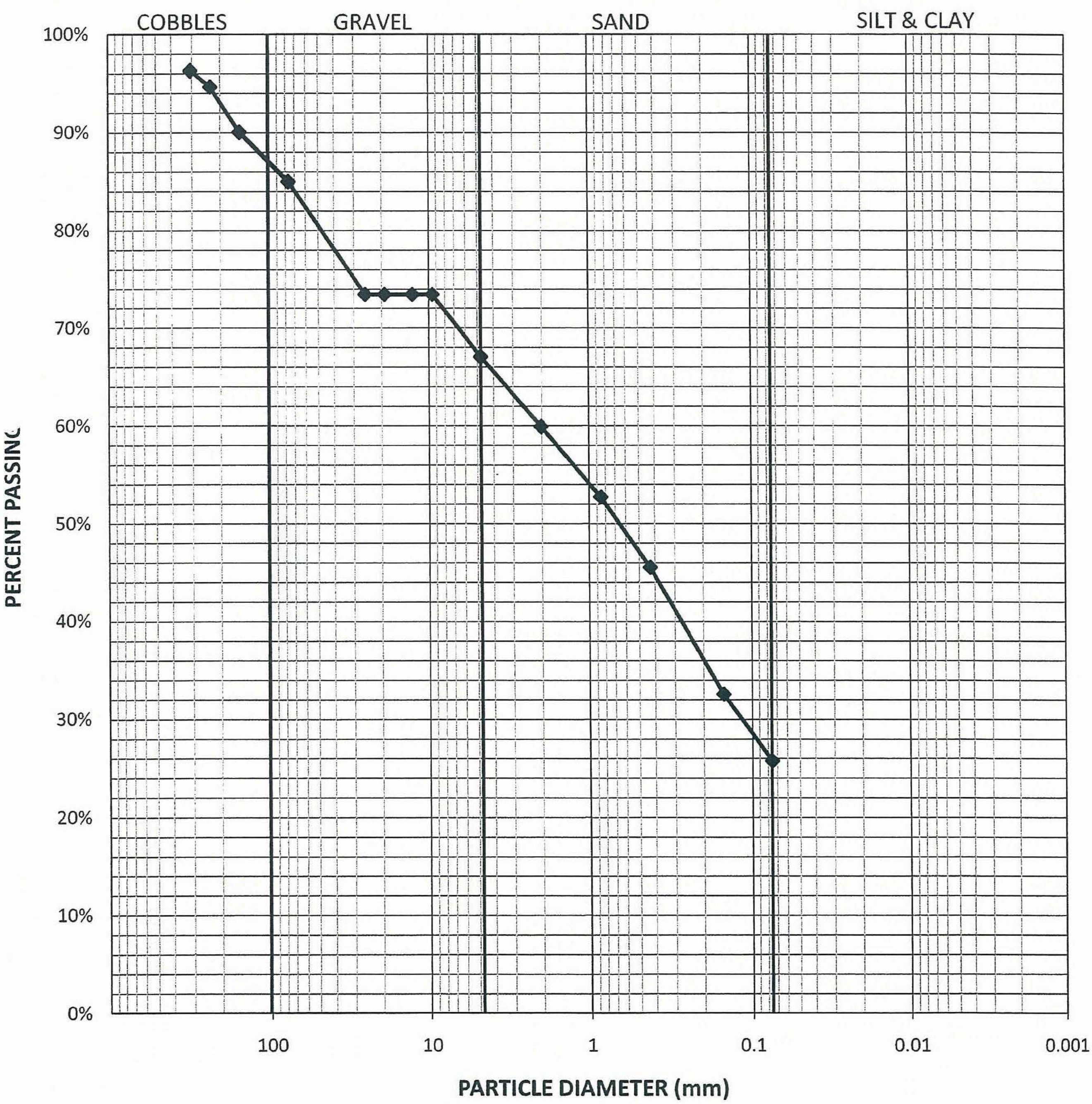


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 2 - BENCH 4		PLATE 13

REFERENCE: Test Pit 2, Bench 4, Depth = 50 feet (Elev. 358) MAD TEST PAD

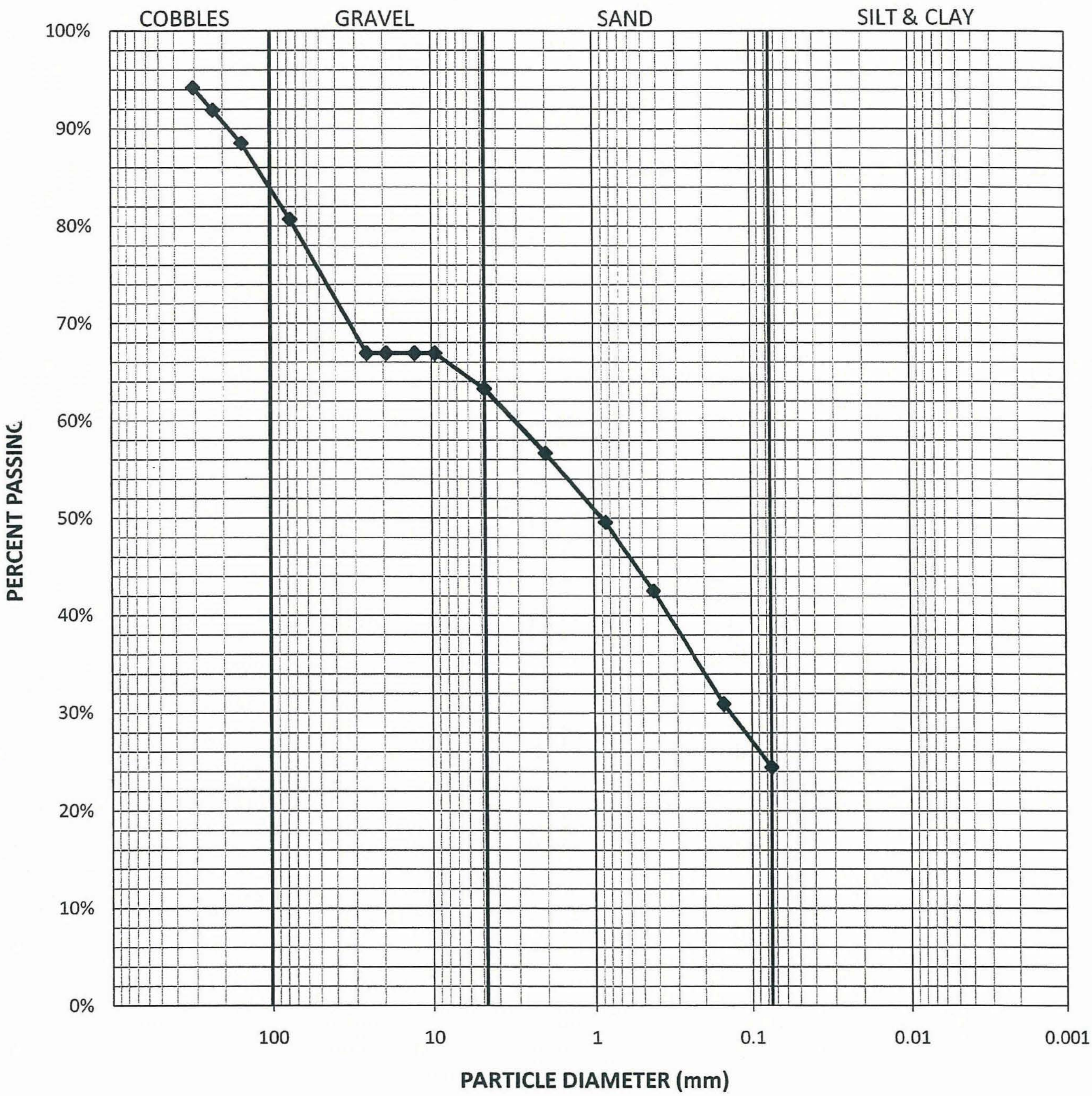


IRVINE GEOTECHNICAL Inc

GRAIN SIZE DISTRIBUTION

IC:	07034-I	CONSULT:	JAI
CLIENT	NUWAY - MNOIAN		
SAMPLE	TEST PIT 2 - BENCH 4		PLATE 14

REFERENCE: Test Pit 2, Bench 4, Depth = 50 feet (Elev. 358) MAD TEST PAD B



IRVINE GEOTECHNICAL Inc

LOG OF BORING

PROJECT IC 07034 Mnoian
DRILL DATE 9/1/2009 - 9/10/2009
LOG DATE 9/1/2009 - 9/10/2009
LOGGED BY MH/JAI
DRILL TYPE Auger
DIAMETER 36 inch

SURFACE ELEVATION 370 feet
DRILLING CONTRACTOR Malcomb Drilling
SURFACE CONDITIONS On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
370	0	**RUBBLE FILL:** Rubble Debris and concrete fragments with a Silty Sand matrix
369	1	
368	2	
367	3	concrete slab fragments, 4 to 6 inches thick and 12 to 36 inches long, reabar protruding into hole, Silty Sand Matrix
366	4	
365	5	abundant rebar, voids and nesting of debris
364	6	
363	7	
362	8	
361	9	
360	10	rebar and trash debris
359	11	
358	12	
357	13	
356	14	wood and metal debris, voids
355	15	
354	16	
353	17	
352	18	
351	19	rubble and concrete fragments, nesting, open voids 6 to 12 inches, caving
350	20	

LOG OF BORING

IRVINE GEOTECHNICAL Inc

PROJECT	IC 07034 Mnoian
DRILL DATE	9/1/2009 - 9/10/2009
LOG DATE	9/1/2009 - 9/10/2009
LOGGED BY	MH/JAI
DRILL TYPE	Auger
DIAMETER	36 inch

SURFACE ELEVATION	370 feet
DRILLING CONTRACTOR	Malcomb Drilling
SURFACE CONDITIONS	On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
350	20	concrete fragments with rebar, nesting, open voids
349	21	metal and trash debris and rebar, concrete fragments 18 to 30 inches in dimension,
348	22	
347	23	
346	24	steel pipe, slab fragments, voids and nesting of debris, rubble and concrete fragments greater than 36 inches in dimension, caving
345	25	
344	26	
343	27	
342	28	bricks, wire, rubble, Silty Sand matrix
341	29	
340	30	rubble and concrete fragments larger than 24 inches, nesting, voids
339	31	
338	32	
337	33	
336	34	conduit with wires
335	35	
334	36	rebar and concrete fragments larger than 48 inches, nesting, voids, caving
333	37	
332	38	Water seeping into boring, water is flowing within a rubble layer and is perched ontop of a Clayey layer, heavy seep on 9/3 becoming a trickle on 9/10 ▽
331	39	
330	40	Sandy Clay, dark brown

IRVINE GEOTECHNICAL Inc

LOG OF BORING

PROJECT IC 07034 Mnoian
DRILL DATE 9/1/2009 - 9/10/2009
LOG DATE 9/1/2009 - 9/10/2009
LOGGED BY MH/JAI
DRILL TYPE Auger
DIAMETER 36 inch

SURFACE ELEVATION 370 feet
DRILLING CONTRACTOR Malcomb Drilling
SURFACE CONDITIONS On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
330	40	concrete fragments with rebar, nesting, open voids
329	41	metal and wood debris and rebar, concrete fragments 18 to greater than 24 inches in dimension, caving, nesting, voids
328	42	
327	43	
326	44	
325	45	
324	46	Clayey Sand matrix, compact, tight, shearing and breaking of rubble fragments
323	47	
322	48	concrete rubble with no soil in matrix, fragments larger than 12 inches, caving, voids
321	49	
320	50	steel pipe, abundant rebar, caving, nesting, abundant voids
319	51	Slab longer than 3 feet,
318	52	
317	53	--
316	54	Concrete rubble fragments in Silty Sand and Clayey Sand matrix, well graded, drastic reduction in rebar and oversize material, dark grey brown
315	55	A/C fragments, rubble lup to 12 inches, weak horizontal layering, no voids,
314	56	
313	57	Horizontal layering, tight, well graded
312	58	
311	59	
310	60	

IRVINE

GEOTECHNICAL Inc

LOG OF BORING

PROJECT	IC 07034 Mnoian
DRILL DATE	9/1/2009 - 9/10/2009
LOG DATE	9/1/2009 - 9/10/2009
LOGGED BY	MH/JAI
DRILL TYPE	Auger
DIAMETER	36 inch

SURFACE ELEVATION	370 feet
DRILLING CONTRACTOR	Malcomb Drilling
SURFACE CONDITIONS	On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
310	60	Silty Sand and Clayey Sand matrix, dark grey, moist, tight, weak horizontal layering, well graded
309	61	
308	62	wire mesh, silty sand with cobbles,
307	63	
306	64	asphalt fragments, mixed with rubble, silty sand and clayey sand matrix, horizontal layering visible, well graded
305	65	
304	66	
303	67	
302	68	
301	69	
300	70	**END Boring at 70 feet**

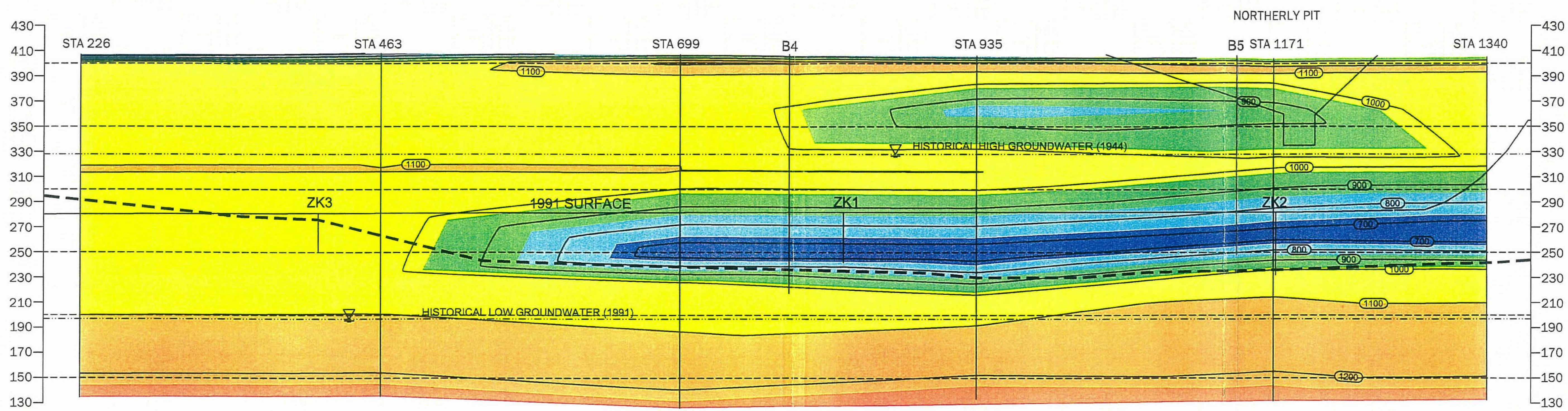
IRVINE GEOTECHNICAL Inc

VELOCITY PROFILE

PROJECT: IC07034 - MNOIAN NU WAY

CONSULTANT: JAI | SCALE: 1" = 80'

S-WAVE VELOCITY SECTION - LINE A NORMALIZED



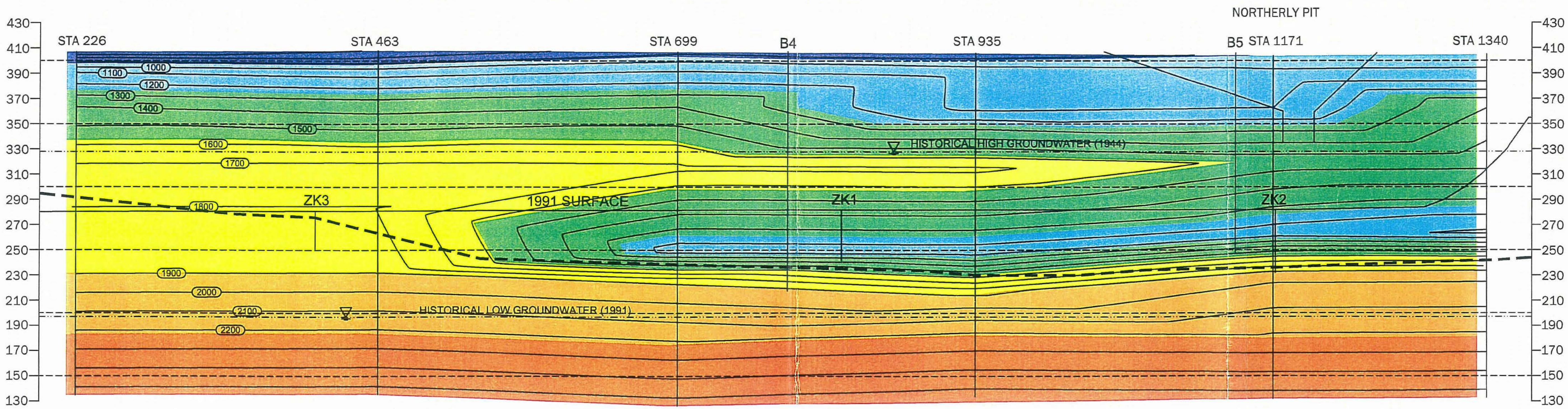
LAYER Vs - ft/s	Color
600-733	
733-855	
855-976	
976-1100	
1100-1219	
1219-3000	

IRVINE
GEOTECHNICAL Inc

VELOCITY PROFILE

PROJECT: IC07034 - MNOIAN NU WAY	
CONSULTANT: JAI	SCALE: 1" = 80'

S-WAVE VELOCITY SECTION - LINE A



LAYER (Vs - ft/s)	COLOR
607 - 929	
929 - 1,252	
1,252 - 1,575	
1,575 - 1,897	
1,897 - 2,220	
2,220 - 3,000	

0916.0114

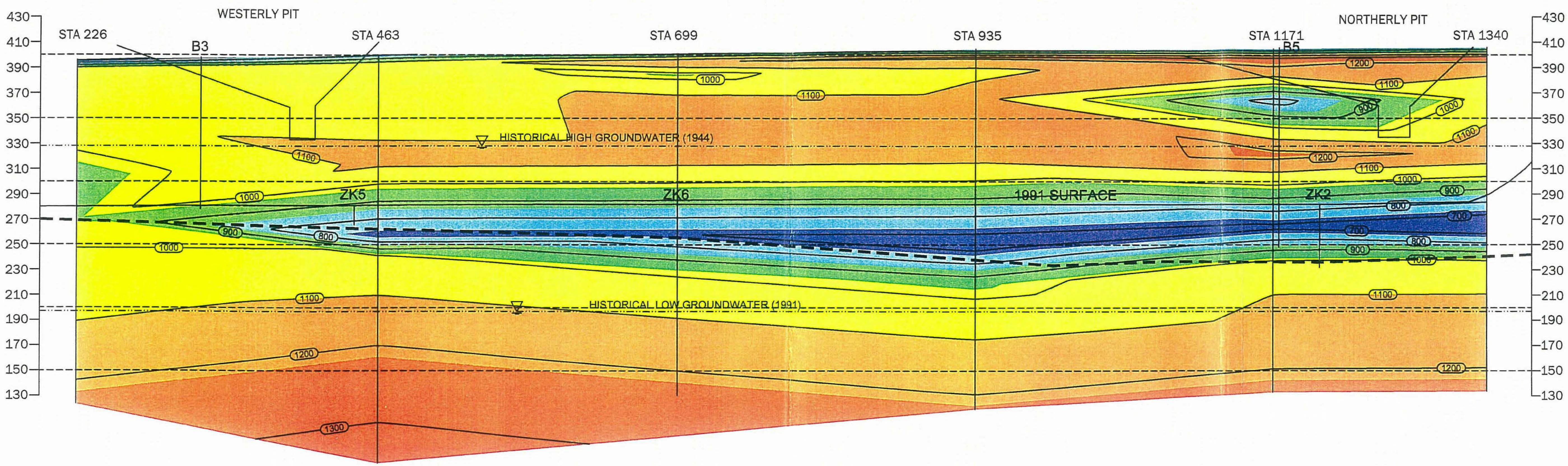
IRVINE GEOTECHNICAL Inc

VELOCITY PROFILE

PROJECT: IC07034 - MNOIAN NU WAY

CONSULTANT: JAI | SCALE: 1" = 80'

S-WAVE VELOCITY SECTION - LINE B NORMALIZED



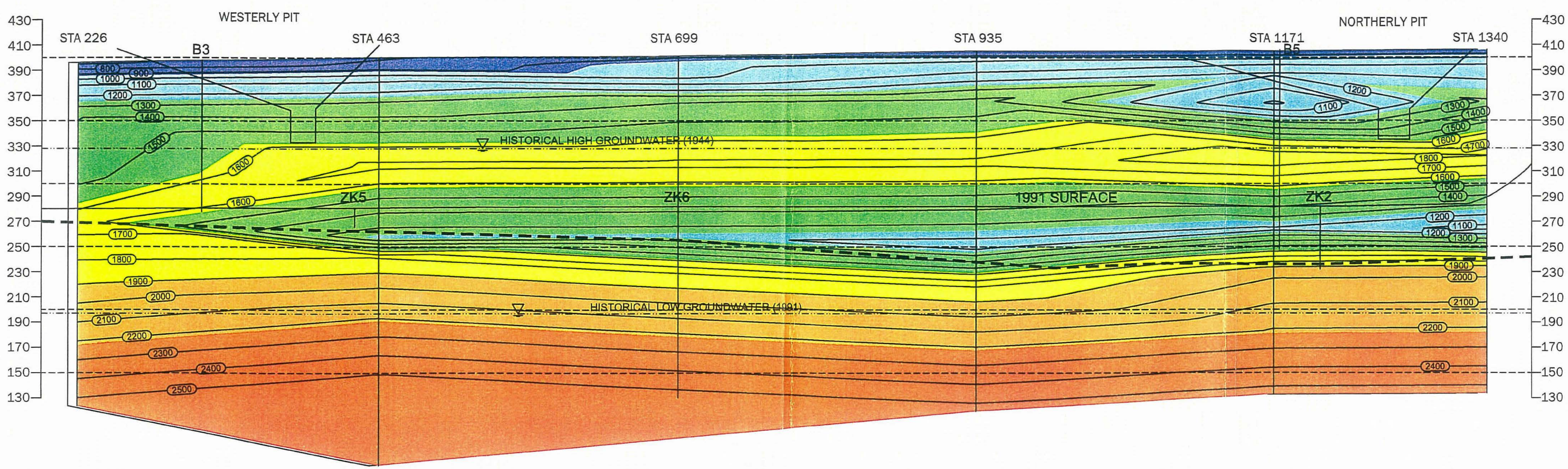
LAYER Vs - ft/s	Color
600-733	
733-855	
855-976	
976-1100	
1100-1219	
1219-3000	

IRVINE GEOTECHNICAL Inc

VELOCITY PROFILE

PROJECT: IC07034 - MNOIAN NU WAY	
CONSULTANT: JAI	SCALE: 1" = 80'

S-WAVE VELOCITY SECTION - LINE B



LAYER (Vs - ft/s)	COLOR
607 - 929	
929 - 1,252	
1,252 - 1,575	
1,575 - 1,897	
1,897 - 2,220	
2,220 - 3,000	

0916.0116

VELOCITY PROFILE

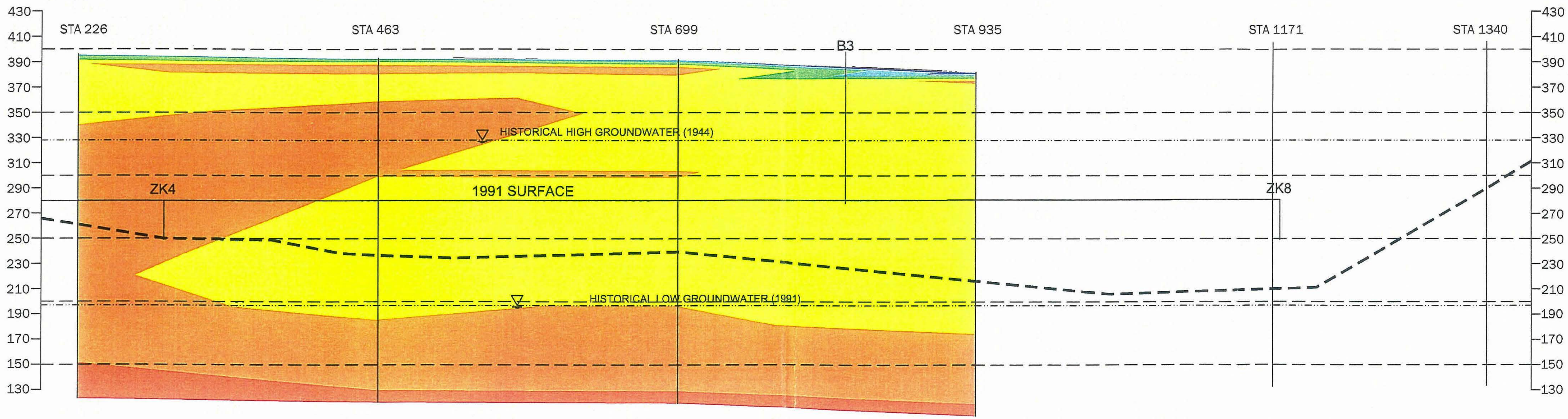
IRVINE GEOTECHNICAL Inc

PROJECT: IC07034 - MNOIAN NU WAY

CONSULTANT: JAI

SCALE: 1" = 80'

S-WAVE VELOCITY SECTION - LINE C NORMALIZED



LAYER Vs - ft/s	Color
600-733	dark blue
733-855	light blue
855-976	green
976-1100	yellow
1100-1219	light orange
1219-3000	orange-red

0916.0117

IRVINE GEOTECHNICAL Inc

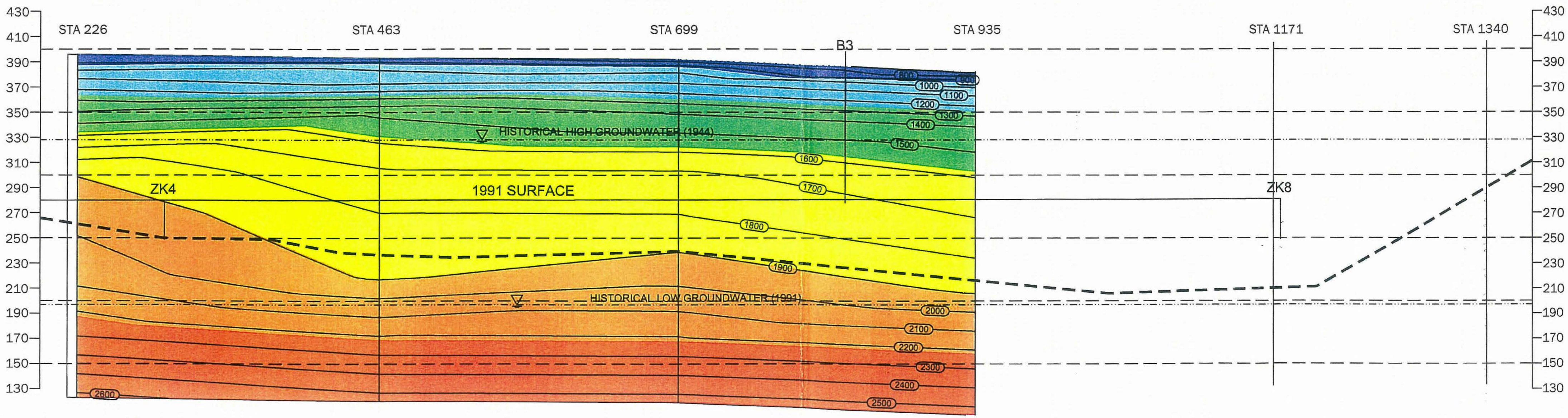
VELOCITY PROFILE

PROJECT: IC07034 - MNOIAN NU WAY

CONSULTANT: JAI

SCALE: 1" = 80'

S-WAVE VELOCITY SECTION - LINE C



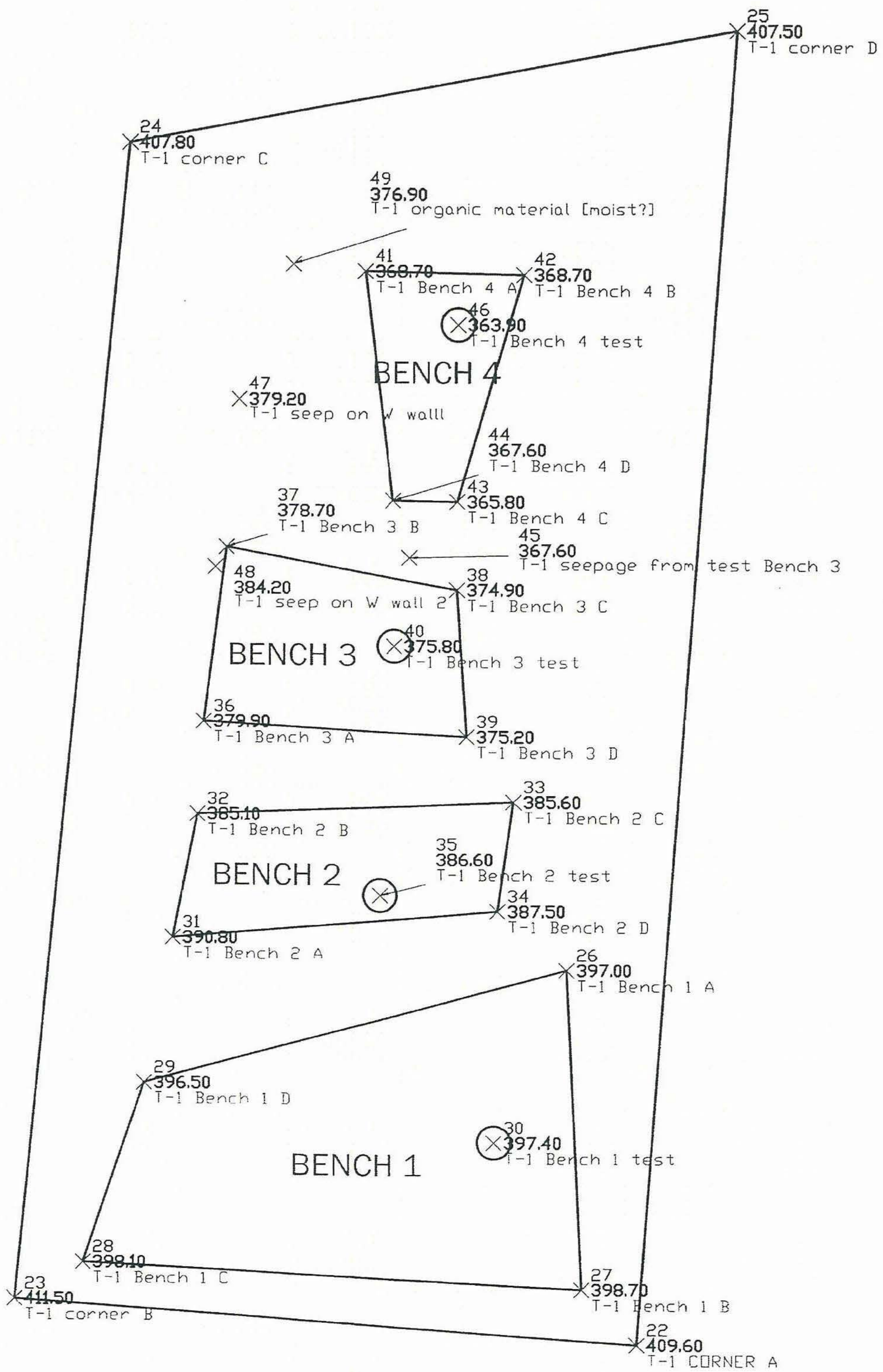
LAYER (Vs - ft/s)	COLOR
607 - 929	
929 - 1,252	
1,252 - 1,575	
1,575 - 1,897	
1,897 - 2,220	
2,220 - 3,000	

IRVINE GEOTECHNICAL Inc

TEST PITS MAP

PROJECT: IC07034 - MNOIAN NU WAY	
CONSULTANT: JAI	SCALE: 1" = 30'

TEST PIT 1



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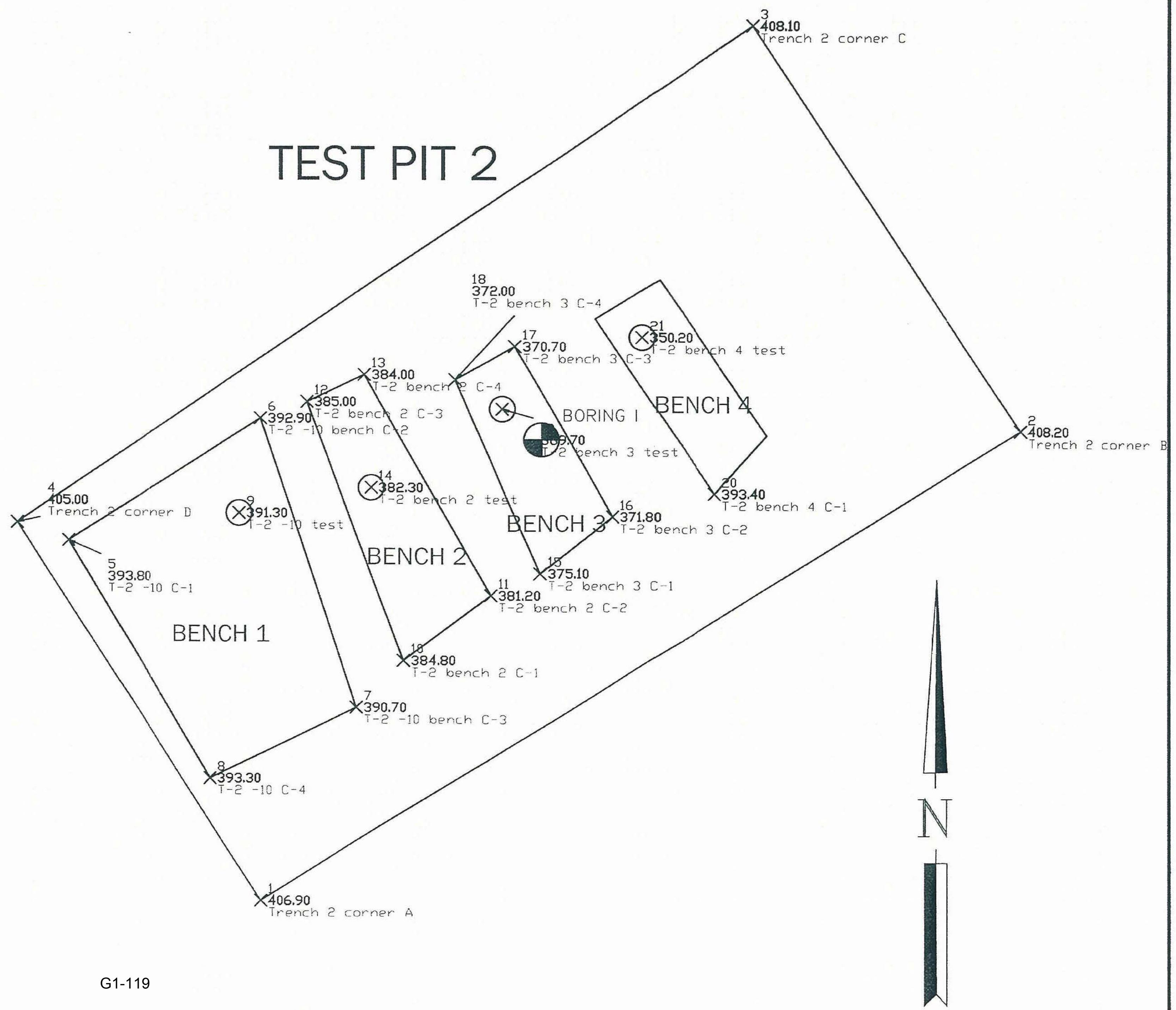

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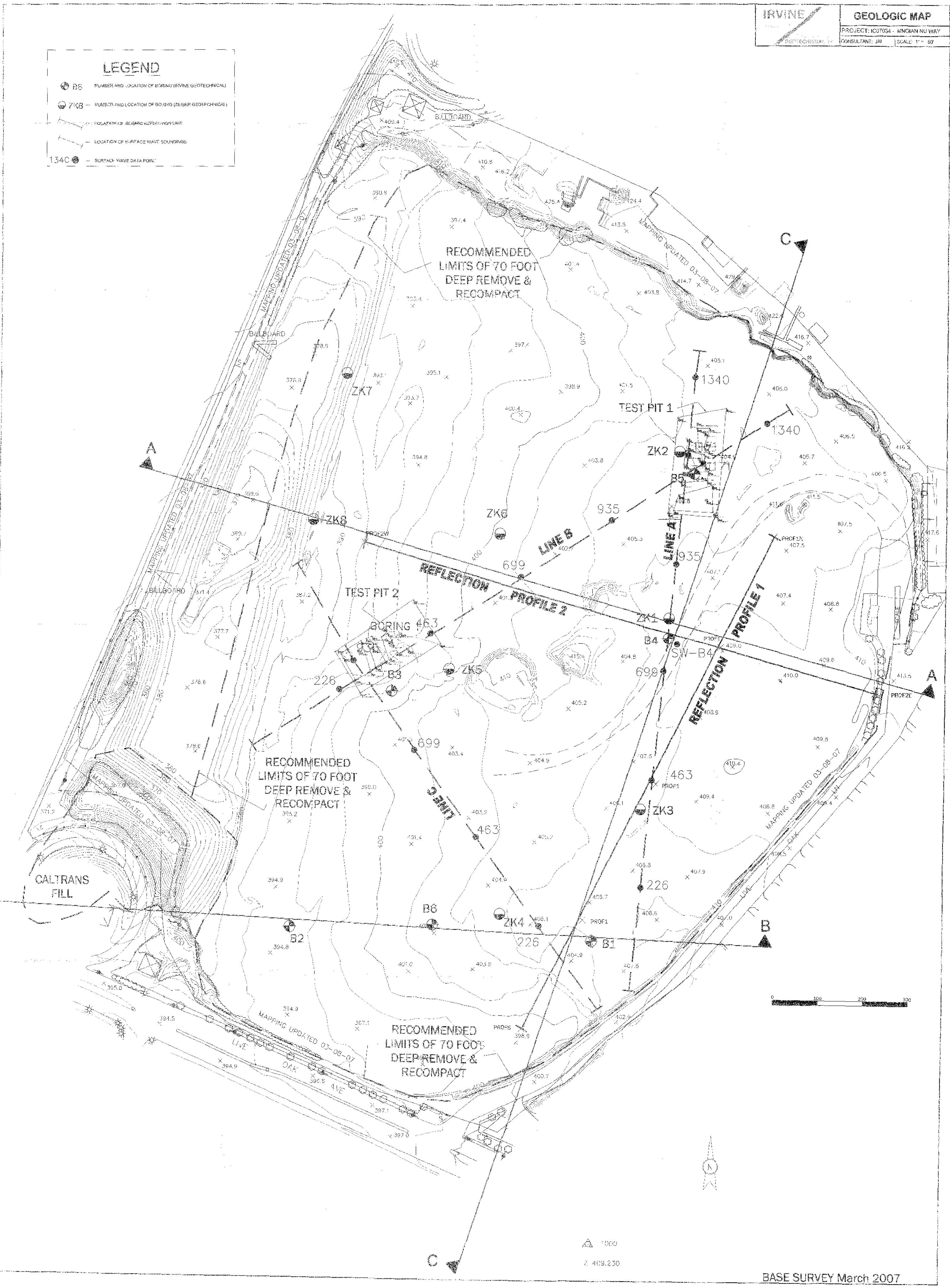
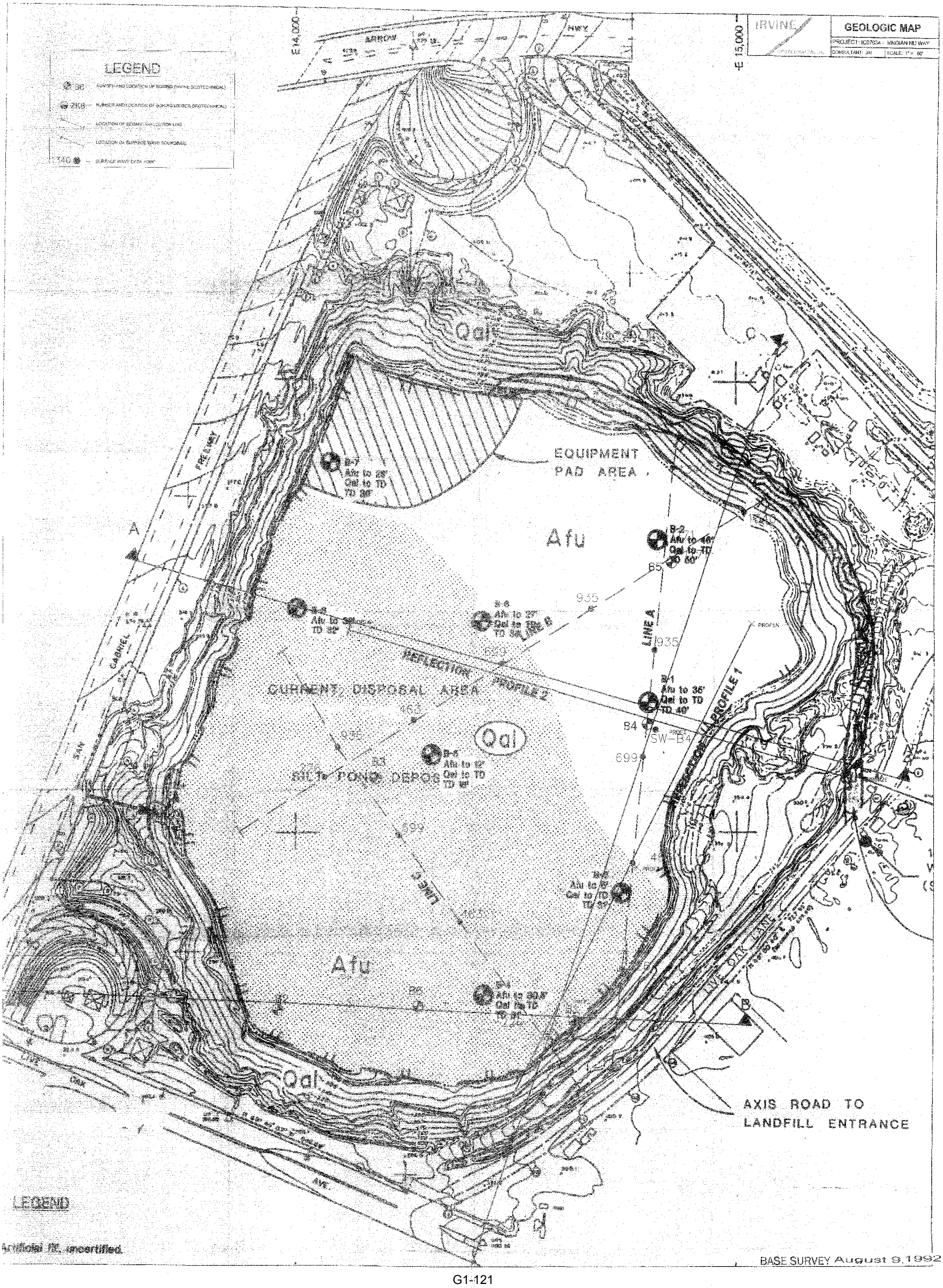
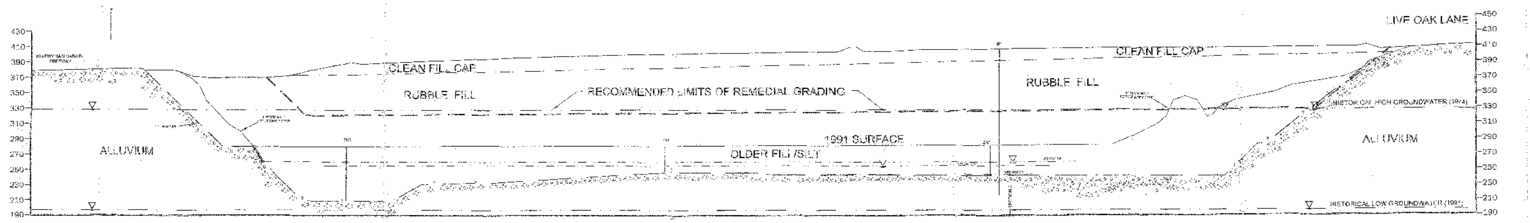


IRVINE
GEOLOGIC MAP
LEGEND
NUMBER AND LOCATION OF BORING (IRVINE GEOTECHNICAL)
NUMBER AND LOCATION OF BORING (ZEISER GEOTECHNICAL)
LOCATION OF SURFACE WAVE SOUNDINGS
SURFACE WAVE DATA POINT
RECOMMENDED LIMITS OF 70 FOOT DEEP REMOVE & RECOMPACT
MAPPING UPDATED 03-08-07
BILLBOARD
TEST PIT 1
TEST PIT 2
BORING
REFLECTION PROFILE 1
REFLECTION PROFILE 2
LINE A
LINE B
LINE C
CALTRANS FILL
ZK1
ZK2
ZK3
ZK4
ZK5
ZK6
ZK7
ZK8
B1
B2
B3
B4
B5
B6
SW-B4
1340
935
699
463
226
LIVE OAK
BASE SURVEY March 2007

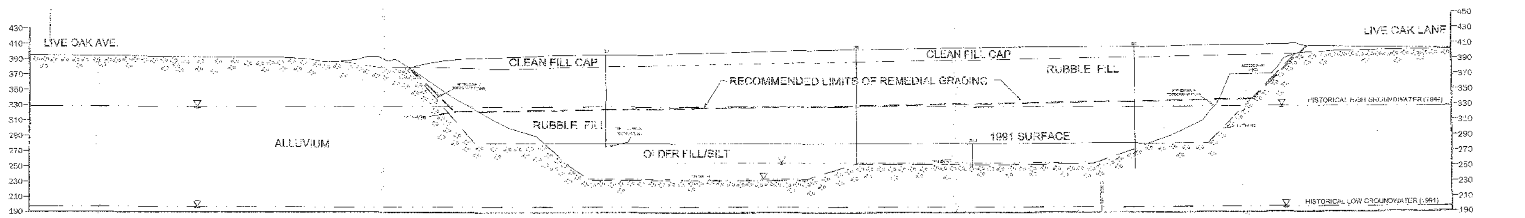


GEOLOGIC MAP
PROJECT: IC07034 - MNOIAN NU WAY
CONSULTANT: JAI
SCALE: 1" = 60'
IRVINE
E 14,000
E 15,000
LEGEND
NUMBER AND LOCATION OF BORING (IRVINE GEOTECHNICAL)
NUMBER AND LOCATION OF BORING (ZEISER GEOTECHNICAL)
LOCATION OF SEISMIC REFLECTION LINE
LOCATION OF SURFACE WAVE SOUNDINGS
340
SURFACE WAVE DATA POINT
ARROW
HWY.
Qal
EQUIPMENT
PAD AREA
Afu
A
C
B
LINE A
REFLECTION PROFILE 2
CURRENT DISPOSAL AREA
SILT POND DEPOS
SW-B4
AXIS ROAD TO
LANDFILL ENTRANCE
LIVE OAK LANE
LEGEND
Artificial fill, uncertified.
BASE SURVEY August 9,1992

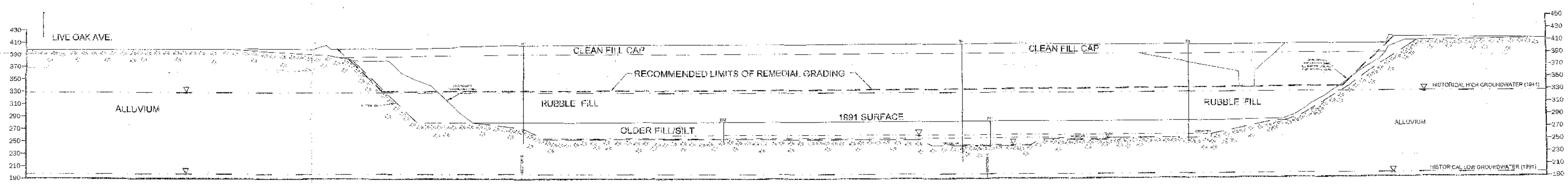
IRVINE GEOTECHNICAL, INC.	SECTIONS A, B & C	
	PROJECT: IC07034	MNOIAN NU WAY
	CONSULTANT: JAI	SCALE: 1" = 80'



SECTION A



SECTION B



SECTION C

APPENDIX I
REPORT BY ADVANCED EARTH SCIENCES

Technical Memorandum on Settlement Analysis Nu-Way Live Oak Pit Irwindale, California

Prepared for:

Mnoian Management, Inc.
P.O. Box 661238
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Prepared by:

Advanced Earth Sciences, Inc.
9307 Research Drive
Irvine, California 92618

Project No.: 10-101
November 2010

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EXECUTIVE SUMMARY

ES.1 PURPOSE AND BACKGROUND

This technical memorandum (TM) has been prepared to document the nature and quality of inert debris fill at the Nu-Way Live Oak Pit (previously known as Owl Rock Quarry) and to present an estimate of potential total and differential settlements of the fill due to static and seismic loading. The pit has been backfilled with up to 180 feet of material including 5 to 40 feet of saturated silt at the bottom of the pit, and inert debris fill consisting predominantly of concrete, brick, and asphalt fragments mixed with varying amounts of soil. The upper 10 to 12 feet consists of soil material compacted in thin lifts under the full-time observations of a geotechnical consultant.

Because the inert debris fill has not been placed and compacted per the City's backfilling requirements included in the 1990 Quarry Rehabilitation Plan nor the 1994 Conditional Use Permit (CUP) issued by the City of Irwindale (City), it is necessary to either demonstrate that the backfilled pit in the current condition can be developed for its intended use (industrial and/or retail development), or to present remedial measures to bring it to a condition suitable for such development, i.e., to make it CUP-compliant equivalent.

For a site to be deemed suitable for proposed development, the City, County and the building code have maximum settlement requirements that need to be met. These call for a maximum differential settlement of 1 inch in 30 feet. A settlement analysis for the backfilled pit under static and seismic loading was performed to determine if this criterion can be met in the current condition, and if not, to determine the depth of removal and replacement (or thickness of engineered fill cap) to bring it to these acceptable settlement standards (to an equivalent of CUP-compliant condition).

ES.2 PIT BACKFILLING HISTORY AND BACKFILL CONDITION

Backfilling began in the late 1980s, and from the late 1980s to 1991 saturated silt was deposited in the bottom of the pit, prior to backfilling with inert debris fill. Zeiser Geotechnical (Zeiser) performed a subsurface investigation of the pit in 1991. They concluded that silt thickness varied from 5 to 40 feet. The total settlement of the silt was estimated at 24 to 36 inches under the weight of proposed 150-foot thick inert debris fill, and 90 percent of this settlement was estimated to occur within one year following completion of backfilling. The pit reached finish grade in 2006.

Backfilling with inert debris fill began in January 1991. During much of 1991, a thick blanket of inert debris fill was placed over the deposited silt to stabilize the pit bottom. Backfilling was observed and tested part time by Zeiser Kling (ZK). From 1991 to 1995, the frequency of ZK site inspection visits was about once a week; from 1995 to 1998/1999 the frequency was once in two weeks to once a month; and from 1999 to 2005 visits were very infrequent, as infrequent as once in 6 months to one visit per year in 2000 and 2001. During 2000 and 2001, up to 500,000 to 700,000 cubic yards of inert debris fill went unobserved between ZK inspection visits.

Agree

Both the 1990 Quarry Rehabilitation Plan and 1994 CUP from the City required the inert debris fill to be placed as engineered fill consisting of minus 12-inch material placed in thin lifts and compacted to 90 percent relative compaction. Oversize (+12-inch material) was to be crushed or placed in windrows surrounded by densified/compacted soil backfill. The specified windrow detail was identical to that adopted by the LA County as standard grading detail for engineered fill sites proposing structure developments. The locations of windrow placement were required to be approved by the City in advance.

Agree

The actual backfilling procedures reported in ZK inspection reports included placement of inert debris fill in thick lifts, typically 5 to 8 feet thick and as much as 10 to 12 feet thick. During the first 5 to 7 years of filling, to about 1998, lifts were reportedly less thick (3 to 4 feet) and the plus 12-inch material was periodically crushed before placement as fill. Crushing operations were reportedly discontinued sometime in early 2000's. The typical placement procedure also included placing a thin 6 to 12-inch cap of soil over each thick lift of inert debris, and compacting the surface. In-place density testing of fill was probably limited to tests on the soil cap or the soil infill material when the proportion of the infill material was significant.

Agree

This backfilling practice resulted in a highly non-uniform fill condition, with nesting of oversize material and open voids or voids partially backfilled with loose infill material. Irvine Geotechnical Inc. (IGI) performed a post-filling investigation in 2008 and exposed the upper 60 to 65 feet of pit face at two locations. Their observations and results of large-scale in-place density tests confirmed the presence of open voids and loosely infilled material. They concluded that the debris fill above approximate elevation of 320 feet (corresponding approximately to the 1998/1999 fill elevation) was in poor condition with open voids or loosely backfilled infill and exhibited a layered pattern with thick lifts of rubble capped with thin layers of soil. These observations mirror the backfilling history that reported excessively thick lifts and inadequate backfilling oversight/ testing by ZK after 1998/1999, and confirm that the inert debris fill down to at least Elevation 320 feet is not CUP-compliant

Agree

Agree

ES.3 SETTLEMENT ANALYSIS

Static and seismic settlement analysis models should reflect the actual fill conditions and placement practices described above. Conventional methods of settlement analysis, particularly for unsaturated materials under seismic loading conditions, are based primarily on laboratory cyclic shear studies on homogeneous sands. There are no industry-accepted standards to predict settlements of inert debris fill that contain significant oversize fragments and significant open voids as is the case at the Nu-Way pit.

Also, there is no database on observed settlements of such debris filled pits in the area or vicinity. Settlement models to be used for predicting seismic and static settlement must take into account the lack of uniformity and control in fill placement operations, and the layered sequence of actual fill placement reported and observed for this site. Due to the above reasons, there will be a significant degree of uncertainty associated with settlement predictions. The approach taken for the settlement analysis was to provide a range of anticipated settlements supported by a rational settlement model, reasonable assumptions and parametric analysis.

The settlement model developed for the Nu-way pit considered the layered nature of much of the debris fills, particularly above the 1998/1999 horizon, consisting of a succession of loose, voided and nested rubble lifts, typically 5 to 8 feet thick and capped by a 6 to 12 inch thick layer of loose to medium dense sands.

Seismically Induced Settlements

Under seismic shaking, mechanisms contributing to settlement include the densification of the loose sand layers between the rubble layers, densification of the rubble layers, and the filling of open voids within the rubble layers due to a combination of sand migration into open voids and collapse of the nested clasts. The cumulative thickness of the sand layers and rubble layers, and the depth interval over which the succession of thin sand layers over thick rubble layers with open voids occur, were estimated at each of the six Becker Borehole locations (drilled by IGI) by evaluating the respective blowcount profiles. The volume of open voids in the rubble layers was calculated based on the results of large-scale density tests in the rubble fill, sand cone density tests in the infill soils and on the particle size distribution tests in the rubble fill materials performed by IGI.

The seismic settlement caused by densification of the sand layers was estimated using the conventional Tokimatsu & Seed (1987) procedure, using the measured Becker hammer blowcounts as input. The seismic settlement caused by densification of the rubble layers was estimated also using the Tokimatsu & Seed procedure, using as input the measured seismic shear wave velocity profile and the measured blowcounts in the rubble fill. Although the applicability of the Tokimatsu & Seed (1987) procedure for the seismic settlement of the rubble layer and the reliability of the predictions are questionable, it was used, nevertheless, to obtain a rough order of magnitude estimate. The third, more dominant, component of seismic settlement is caused by the partial filling of open voids in the rubble fill by a combination of fines migrating from overlying sand layers into the voids and collapse of the nested rubble clasts. The average volume of open voids (as a percentage of total volume) in the rubble layer was estimated to be 6.7 percent. Not all of the open voids will be filled as a result of seismic shaking. The proportion of open voids that get filled will depend on the amplitude, frequency and duration of shaking, but is not known in the absence of specific physical laboratory modeling. As an initial estimate, it was assumed that approximately 20% of the open voids are filled due to seismic shaking. Subsequently, sensitivity analyses were performed by varying this percentage. The total seismic settlements were evaluated at each of the six Becker hammer borehole locations. Based on these assumptions, the total seismic settlement of the in-place debris fill is estimated to range from 5.6 inches to 14.4 inches with an average of 11.2 inches. Considering that the total thickness of the debris fill contributing to the settlements is approximately 110 feet, the average settlement of 11.2 inches is roughly equivalent to 0.85 percent of fill thickness. This estimate compares well with some well documented case histories of settlement of dry compacted sandy fills in southern California which settled approximately 0.6 to 0.9 percent of fill thickness during the 1971 San Fernando and 1994 Northridge earthquakes, under ground accelerations comparable to the design ground accelerations for the Nu-Way pit. However, considering that the rubble fill consists of uncompacted fill with significant voids, the actual settlement could be even higher.

One of the remedial measures for controlling seismic settlement at the site will consist of partial removal and replacement of the existing debris fill with a properly processed and compacted fill cap. With increasing depth of removal and replacement of the existing fill, the remaining fill thickness vulnerable to seismic settlements would decrease, thereby resulting in lower seismic settlement potential. The presence of the cap will also serve to attenuate the total and differential settlement taking place at depth as it manifests at the surface of the fill cap. The non-linear finite difference Computer Program FLAC was used to model the impact of cap thickness on surface manifestation of total and differential settlements. The analysis considered the surface manifestation of differential settlement at a specific location (caused by an isolated large void) and the surface manifestation of randomly varying settlements applied at the base of the fill cap, for increasing thicknesses of fill cap. The results, presented as plots of surficial total and differential settlements versus thickness of fill cap (for a range of assumed values of percentage open voids filled by migration of fines and collapse), show decreasing values of surficial settlement with increasing fill cap thickness. For example, the results show that for a 40-foot thick fill cap the maximum total settlements are on the order of 4 to 10 inches, while the maximum surficial differential settlements range from 1 to 3.5 inches over a 30-foot length. If the fill cap thickness is increased to 70 feet, the total surficial settlements are less than 3.7 inches and differential surficial settlements are less than 1 inch over a 30-foot length, the latter satisfying the regulatory requirements for a developable site.

Static Settlements

Under static loading, the components of settlement include the settlement of the debris fill under its self weight, long-term primary and secondary settlement of the of the silt deposits underlying the debris fill, and settlement caused by filling of open voids due to migration of fines (sands) and collapse as a result of fluctuations in the groundwater levels. Of these, the settlement of the debris fill due to self weight is anticipated to occur during and immediately following completion of filling. The majority of the long-term settlement of the silt deposits due to the debris fill loads is estimated to have been complete over the 4+ years that have elapsed since completion of filling operations.

Historical groundwater data indicate that the groundwater levels fluctuate, with an estimated high groundwater level at elevation 330 feet. The placement history (1991-2006) suggests that the fills placed above approximate elevation 290 feet may not have been subjected to saturation due to groundwater fluctuations during the pit filling period. The estimated total settlement caused by groundwater fluctuations will depend on the percentage of open voids that get filled to due to migration of fines and collapse. Assuming that 20 to 40 percent of the open voids in the rubble fills get filled due to fines migration and collapse caused by groundwater fluctuations during the pit filling period, the resulting total settlements are estimated to range from 3.2 to 6.4 inches. These settlements occur at a depth of 80 feet below the finish ground surface. The corresponding differential settlements at the ground surface are estimated to be less than 1 inch over 30 feet.

Because the same mechanism (migration of sands into open voids and collapse) control both seismic settlement and settlement due to groundwater fluctuations, the two components of settlement are not considered to be cumulative, and the maximum differential settlement due to both components may still be less than 1 inch in 30 feet.

ES.4 CONCLUSIONS AND RECOMMENDATIONS

The evaluation of fill quality and results of settlement analyses suggest that at least the upper 70 feet of existing, predominantly uncontrolled fill should be removed and replaced by a properly processed and compacted engineered fill cap, in order to limit the potential for total settlements and limit the differential settlements to within tolerable limits (less than 1 inch over 30 feet).

Additional evaluation of fill to be left under the cap, i.e. below the 70-foot depth, would probably be required to confirm or otherwise modify cap thickness recommendation and to satisfy the requirements of the LADPW/City of Irwindale for proposed development for light industrial/retail structures.

1.0 INTRODUCTION

1.1 GENERAL

This technical memorandum (TM) by Advanced Earth Sciences, Inc. (AES) has been prepared to document the nature and quality of inert debris fill at the Nu-Way Live Oak Pit (previously known as the Owl Rock Quarry site) and to provide an estimate of potential static and dynamic (seismic) settlements of the inert debris fill, based on the available data. The purpose of this analysis is to evaluate the suitability of the existing fill in its current condition to support a proposed industrial/retail development at the site and to consider suitable remedial options, as appropriate, to bring it to a condition that would permit such a development. The Nu-Way Live Oak Pit is located between Live Oak Avenue and Arrow Highway, east of Route I-605 freeway and west of San Gabriel River in Irwindale, California. The site location is shown on Figure 1-1.

1.2 PROJECT BACKGROUND AND SITE CONDITIONS

Based on our review of previous investigations performed by Irvine Geotechnical, Inc. (IGI) in 2007 and 2008, the site occupies a total area of about 65 acres. Finished elevations of the backfilled pit range from about 408 to 410 feet along the eastern portions of the property to approximately 375 feet along the western edge of the pad and within the Southern California Edison (SCE) easement. The estimated maximum depth of the fill appears to be approximately 180 to 185 feet below ground surface (bgs).

According to the descriptions of the fill materials provided by previous investigators, IGI (2008a, 2008b), Zeiser Geotechnical [Zeiser] (1991) and Zeiser Kling [ZK] (1991 through 2006), the types of inert debris materials disposed in the pit included *"earth materials including gravel, sand and silt size soils, construction materials including brick, concrete, some concrete with rebar, asphalt, minor amounts of wallboard and porcelain toilet fixtures."* In the southwestern part of the pit, saturated silt from quarry operations was reportedly disposed between 1988 and 1991. According to Zeiser (1991), the silt thickness varied from 5 feet to 40 feet. Although considered compressible under load, Zeiser (1991) concluded that 90 percent of this settlement will be complete within 1 year after completion of grading. The inert debris fill in the pit, estimated at approximately 11 million cu. yds., was placed between 1991 and 2006. The upper 10 to 12 feet of the pit were backfilled with a clean, well compacted soil cap.

1.3 SCOPE OF ANALYSIS

The scope of work for our analysis included:

1. Review data on backfilling operations of the pit including daily field reports and in situ density test results performed by ZK (1991-2006) and by Hushmand Associates, Inc. (HAI) in 2006 and 2007.

2. Review the placement, testing and reporting requirements for inert debris fill as contained in the Conditional Use Permit (CUP) issued by the City of Irwindale in December 1994 and in the Quarry Rehabilitation Plan agreement dated January 1990, between the Owner and the City of Irwindale.

3. Provide an opinion on the compliance or non-compliance of actual filling operations with the requirements of the CUP and Quarry Rehabilitation Plan agreements, on the basis of data reported by Zeiser, ZK and IGI investigations.

4. Perform a settlement evaluation of the fill in its current condition based on the material properties as interpreted from the results of ZK inspection and test reports and from IGI investigations.

5. Evaluate the suitability of the existing fill to support the proposed development and, if not suitable, evaluate remedial options in an attempt to bring it to a condition considered acceptable for proposed development.

The results of these evaluations are discussed in the following sections.

2.0 AVAILABLE DATA

2.1 GENERAL

The reports reviewed for our current evaluation are listed in Section 6.0 – References. In summary, the principal documents/reports reviewed for the current study included:

1. Pre-Filling Investigation by Zeiser performed in 1990 (February 1991 report)

2. Backfilling Guidelines and Regulations in 1994 CUP and 1990 Agreement re. Quarry Rehabilitation Plan

3. Field Reports of Backfilling Monitoring and Testing by ZK from 1991 through 2006, including periodic progress reports

4. Test data obtained by HAI (2006 and 2007) for upper soil cap

5. Post-Filling Subsurface Explorations by IGI, Phases 1, 2 and 3 (2008a, 2008b, 2010), including logs of two large test pits prepared by GeoLogic Associates (GLA)

2.2 PRE-FILLING INVESTIGATION (Zeiser Geotechnical, 1991)

Zeiser conducted a subsurface investigation for the site in 1990 and presented their findings and geotechnical recommendations for inert debris fill placement for the pit in their report dated February 4, 1991 (Zeiser, 1991). Their key conclusions/recommendations included the following:

1. *The subsoil encountered in the exploration borings consisted of uncertified artificial fill 5 to 40 feet in thickness. The settlement is expected to range from 24 inches to 36 inches and may require approximately 1 year after completion of grading (±200 feet of fill) to achieve 90 percent of total settlement.*

2. *The groundwater was recorded at El. 203. Groundwater was not anticipated to be a problem for fill placement and also, liquefaction was not likely.*

3. *Fill materials shall be placed in near-horizontal layers not exceeding 6 inches in compacted thickness (consultant may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness).*

4. *Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fills, unless the location, materials and disposal methods are specifically approved by the consultant. Oversized disposal operations should be such that nesting of oversized materials does not occur, and such that the oversize material is completely surrounded by compacted or densified fill.*

5. *Field tests to check the fill moisture and degree of compaction should be performed by the consultant. The location and frequency of tests should be at the consultant's discretion. In general, the tests will be taken at an interval not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of embankment.*

The Zeiser report also specified that all earthwork and grading operations should be performed in accordance with all applicable City of Irwindale and Los Angeles County requirements. The General Earthwork Specifications included in Appendix D of their report included typical Windrow Rock Disposal Detail and other Grading Details exhibits that are directly obtained from the Los Angeles County Grading Code details, for engineered fill sites graded for future development.

2.3 BACKFILLING GUIDELINES AND REGULATIONS

2.3.1 Agreement between City of Irwindale and Nu-Way dated January 25, 1990

This agreement made pursuant to City's Quarry Rehabilitation Plan goal to find useful purposes for the City's many abandoned quarry pits, permitted Nu-Way to fill the Owl Rock Pit (Nu-Way Live Oak) with imported inert fill in accordance with the geotechnical consultant's recommendations. The Agreement required Nu-Way to comply with the following:

- *Provide, place and compact to 90% density clean earth and solid fill material (e.g., broken A.C. and concrete). No organic materials will be imported to the site.*

- *Provide proper supervision and control of the project at all times.*

- *Constantly monitor the loads to assure that no hazardous, or toxic materials or solvents are brought to the site. NU-WAY will provide certification on a monthly basis that only clean, inert materials were imported to the site the previous month.*

- *The area under the transmission lines (Southern California Edison Company) is to be compacted under special and unique circumstances. Inasmuch as no buildings may be allowed, the recommendations of the geotechnical consultants will be accepted in terms of compaction and certification.*

2.3.2 Conditional Use Permit (CUP) dated December 15, 1994

In Resolution No. 94.55-1381, the City Council of the City of Irwindale certified a Final EIR and approved a CUP for the Nu-Way Live Oak Inert Landfill. The CUP findings concluded that *"the proposed use and development are consistent with the City's General Plan, applicable specific plans and are permitted within the zone in accordance with the City's zoning ordinance."*

The key excerpts of the CUP, as they relate to the quality of the inert fill, placement and compaction requirements, and reporting requirements and frequencies, and as defined in Exhibit A-II ONSITE DEVELOPMENT CONDITIONS of the CUP, are provided below:

- *Applicant shall provide, place and compact to 90% density clean earth and inorganic solid fill materials (e.g., broken concrete and A.C.). No organic materials will be imported to the site.*

- *The Applicant will be responsible for monitoring the materials for discharge at the site to insure that such materials meet the discharge specifications presented in the project EIR regarding the California Regional Water Quality Control Board waste discharge requirements. Any materials found in the pit that do not meet the approved discharge specifications shall be removed from the site.*

- *Oversize materials shall be crushed prior to placement as fill, or set in windrows that meet the conditions presented in the Project Geotechnical Report by Zeiser Consultants dated October 15, 1990 which has been modified by the City Council to read as follows:*

  "Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fills, unless the location, materials and disposal methods are specifically approved by the City. The City shall receive notice of the intent to windrow oversize materials as approved, 48 hours prior to commencing such operations. The applicant/operator shall furnish periodic reports on windrow disposal, to the City which information will be provided to the City Council. Oversized disposal operations shall be such that nesting of oversized material does not occur, and such that the oversize material is completely surrounded by compacted or densified fill (Figure 1). Oversize material shall not be placed within 10 feet vertically of finish grade or construction, unless specifically approved by the City." (see Figure 1 at end of Exhibit A.)

- *The area under the transmission lines (Southern California Edison Company) is to be compacted under special and unique circumstances. Inasmuch as no buildings may be allowed, the recommendations of the geotechnical consultants will be accepted in terms of compaction and certification.*

- *The Applicant shall, provide to the City of Irwindale, for approval by the City Engineer, geotechnical progress reports on each annual anniversary of the date of approval of the Resolution, or whenever five feet fill is placed, which ever comes first. This report shall be prepared by the Project geotechnical consultant and shall include certification of quality of material, compaction of material, test results, and map of test locations.*

2.3.3 Interpretation of Intent of CUP and January 1990 Agreements

Although the site backfill material is permitted to be inert debris fill (including concrete, brick and other inert fragments), the backfilling specifications in the 1990 Agreement and 1994 CUP are typical of engineered fill materials called for in the County Grading Codes for grading the sites for structural developments. This is confirmed by:

- Compaction Standard – minimum 90 percent.

- No oversize (+12 inches) in fill unless crushed or placed in windrows. The windrow detail in Figure 1 of Exhibit A of the CUP is exactly the same as in the Los Angeles County Grading Code.

- Reporting Requirements – geotechnical progress reports prepared annually or for every 5 vertical feet of fill, whichever comes first, providing test locations and test results on compaction of materials.

The only difference between conventional engineered fill and the inert debris backfill permitted for the subject site is that the source of the coarse fragments may be construction debris material, i.e., broken concrete, bricks, etc., as opposed to soil borrow materials that include gravel, cobbles and boulders. The CUP and Quarry Rehabilitation Plan Agreement mandate that the pit be backfilled with engineered fill, placed and compacted in layers with no nesting of coarse fragments. Material greater than 12-inch size was to be placed in windrows surrounded by soil and the voids in the oversize material backfilled with material with a sand equivalent (SE) > 30 and densified, all in accordance with Los Angeles County's conventional grading requirements.

2.4 PIT BACKFILLING HISTORY

2.4.1 Fill Volumes and Inspection/Testing Frequencies

The pit was backfilled with inert debris fill between 1991 and 2006. Our observations and opinions concerning backfilling materials, filling procedures, and frequency of fill monitoring and testing are based on the following data made available to and reviewed by AES:

- Volume estimates of materials received at the pit between 1991 and 2000 from Operator's records and from as-placed material quantity takeoff from aerial photography between 2000 and 2006

- ZK daily reports of field observations and testing of backfills from January 23, 1991 through July 21, 2007

- HAI daily reports and test results for backfill in the upper 12- to 15-foot thick soil cap between January and December 2006

These data are illustrated on the calendar (Figure 2-1) from Year 1991 through 2006 and include: dates of ZK and HAI site inspections, number of in place density tests taken during their inspection visits and reported comments on excessive lift thicknesses observed by ZK technicians (marked in orange with lift thickness indicated within the box). The data are also summarized in Table 2-1, on a yearly basis. From the total number of site visits and number of tests taken by ZK technicians in a calendar year, an average frequency of inspection and testing, expressed as 1 visit per "x" cu. yds. or 1 test taken every "y" cu. yds., has been derived and is presented in Table 2-1. The frequencies are then graphically illustrated in Figure 2-2. These figures (Figures 2-1 and 2-2) and Table 2-1 clearly illustrate the following trends of fill monitoring and testing:

- Between 1991 and 1993, the average frequency of field inspection visits is about once per week and the fill density test frequency varies from 1 test per 4,300 cu. yds. in 1991 to about 1 test per 2,600 cu. yds. in 1992 and 1993. The lower frequency of testing per fill volume in 1991 may be related to a large volume of initial filling comprising thick rock/debris blanket over the existing uncertified fill (including saturated silts) to stabilize the pit bottom. This blanket fill apparently did not have to be and could not be tested.

- In 1994, the frequency of field visits averaged about 1 visit every 2 weeks. However, testing frequency by volume remained consistent as in 1992 and 1993 and averaged approximately 1 test per 2,800 cu. yds.

- From 1995 through 1998, the frequency of field visits decreased to an average of 1 visit per month and fill testing frequency progressively decreased, ranging from 1 test per 5,800 cu. yds. in 1995 to about 1 test per 9,500 cu. yds. in 1996 and 1997, and to about 1 test per 14,000 cu. yds. in 1998. The frequency by number of inspection visits ranged from about 1 visit for every 25,000 cu. yds. in 1995 to about 1 visit per 55,000 cu. yds. in 1998.

- Between 1999 and 2002, the frequency of visits and testing deteriorated to almost "no supervision/oversight" level, with 4 visits in 1999, representing 1 visit per approximately 175,000 cu. yds., 1 visit in Year 2000 when over 700,000 cu. yds. of fill was placed, 5 visits in

2001 representing 1 visit for approximately 156,000 cu. yds. of fill, and 6 visits in 2002 representing 1 visit per approximately 100,000 cu. yds. of fill placement.

- In 2003 and 2004, the frequency of visits average 1 per month to 1 every 2 months representing 50,000 to 100,000 cu. yds. of fill placed between visits.

- In 2005, visits increased to an average of 1 visit per week representing 1 visit every 25,000 cu. yds. However, excessive fill lift thicknesses ranging between 8 feet and 12 feet were consistently reported throughout the year.

- In 2006, much of the fill placement activity was for the upper 10- to 15-foot thick soil cap that received full-time supervision and testing by HAI with an average test frequency of 1 test per 1,400 cu. yds.

It should be noted that the number of tests (844 tests) for the upper 10- to 15-foot thick soil cap, representing about 1 million cu. yds. of fill, was about 5 times the total tests taken over a 7-year period from 1998 through 2004, when over 5 million cu. yds. of fill was placed. It is clear that about 5 million cu. yds. of debris fill representing roughly the upper 70- to 80-foot thickness received little to no supervision and received minimal testing. This trend, combined with placement methods/fill characteristics discussed below, raises serious concerns with regard to its suitability to support any development of industrial/retail structures within allowable settlement tolerances.

2.4.2 Inert Debris Fill Characteristics and Lift Thicknesses

In general, the inert debris fill consisted primarily of concrete with abundant rebar, floor tile, cement and asphalt shingles, bricks, soil and crushed glass. In the early stages of fill placement drywall was reportedly accepted as backfill material and the drywall areas were moisture conditioned to break up the material and mix with the soil.

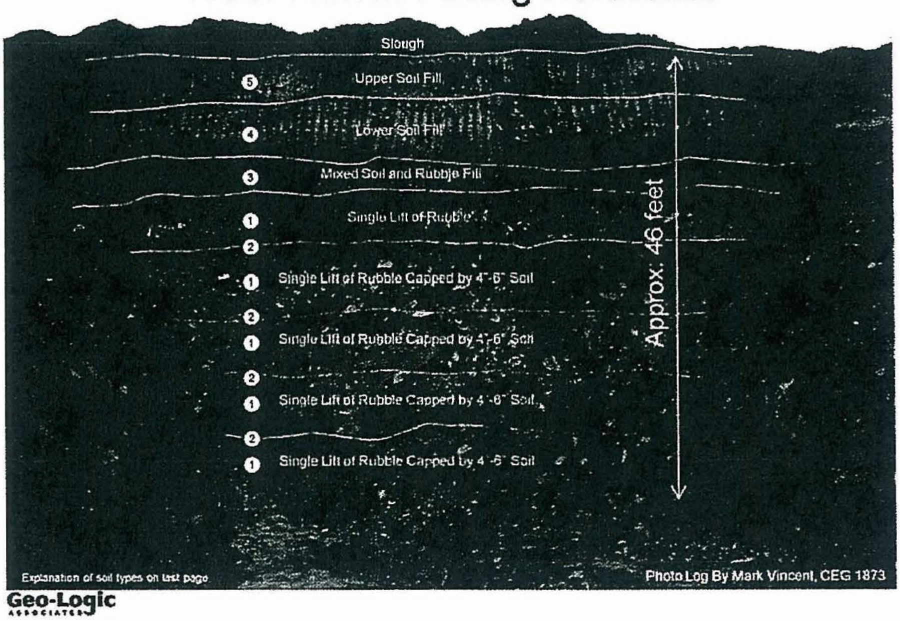
The rubble-soil mixture ratio reportedly varied through the pit backfilling history depending upon composition of incoming loads. According to IGI's Phase 2 Investigation report (IGI, 2008b), significant (approximately 15 to 20 percent) oversize, larger than 12 inches was present in the fill. Of the oversize fraction, steel-reinforced concrete fragments, foundation elements, columns and other construction demolition debris with dimensions ranging from 24 inches to more than 20 feet were present in the fill.

During the earlier parts of backfill placement, a mobile crusher was reportedly used to crush material larger than 12 inches prior to placement in the fill. However, the regularity of crusher operations is not well documented. The ZK daily field reports indicate that the crusher was working during most of their field visits through 1990s. It should be noted though that the frequency of their field visits was less than once per month from 1997 onwards. There is no record of crusher working in any of their reports beyond 2002.

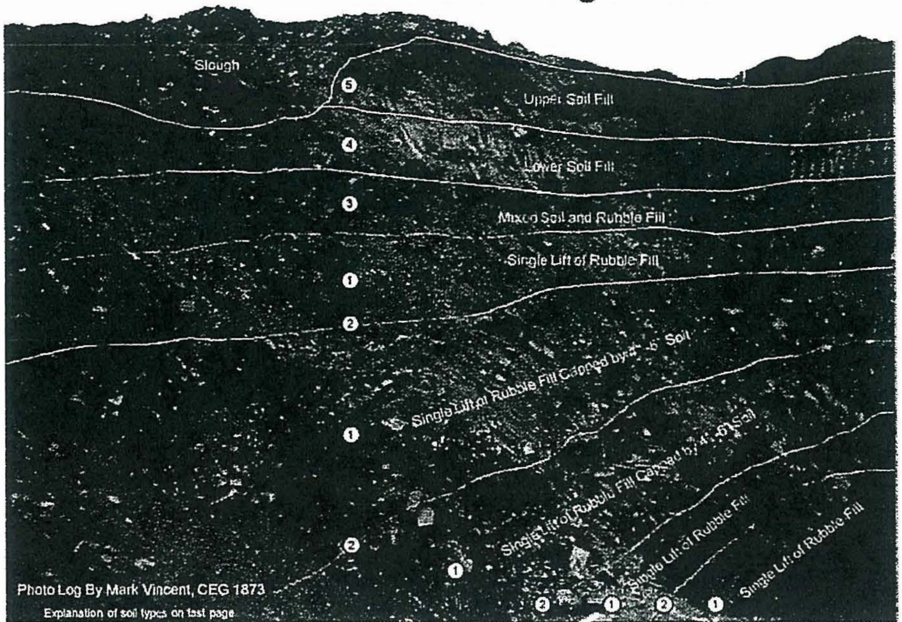
The patterns of fill placement and lift thicknesses were always of concern, particularly after 1993. The fill lifts were reportedly 3 to 4 feet thick during earlier stages of fill placement and were routinely in excess of 5 feet during the late 1990s through 2005. Also, the oversize material and thick lifts were placed in "blanket type" pattern, with 5- to 8-foot thick lift of debris fill placed by end dumping and topped with a 6- to 12-inch thick soil layer/blanket. The testing performed by ZK was always in the soil fill material or matrix, either within the soil blanket or in the bulk fill layer where the soil component was significant. On numerous occasions, the ZK daily reports indicate that the fill was not suitable for testing, i.e., oversize fraction was too excessive for any of the conventional field density test methods (nuclear or sand cone) to be of any meaningful value.

The lift thicknesses from late 1990s through 2005 were frequently greater than 7 to 8 feet and the oversize material was never placed in windrows, as called for in the CUP. A typical description of rubble fill and soil blanket layers, as provided by GLA and as illustrated in a mapped log of the test excavations performed by IGI in the in-place debris fill, is provided in the following photographs. This type of placement pattern was identified to a depth of at least 65 to 70 feet bgs, the maximum depth of the pit excavated during IGI investigations.

West Trench Facing Northeast



West Trench Facing North



Geo-Logic Associates

Legend:

1. **Rubble Fill – consisting primarily of concrete with abundant protruding rebar, floor tile, cement and asphalt shingles, bricks, soil, crushed glass, and diatomaceous earth filter packing. Many blocks of concrete have extreme aspect ratios (ratio of the length to width) with some maximum dimensions in excess of four feet. Most of the blocks are nested with some loose, dry, dark gray, organic rich, clayey soil backfill around the blocks. Extensive rubble zones are voided and have no soil fill. These rubble fill lifts or layers appear to be formed by end dumping of debris from trucks into layers four to five feet thick. The lifts of layers were then capped by a four to six inch thick layer of silty sand to sandy silt. Areas of seepage observed in the excavations consist of what appears to be diatomaceous earth filter packing. The material was most likely placed in the fill while saturated and have remained wet due to a capillary break resulting from being surrounded by concrete blocks with little or no soil backfill between blocks.**

2. **Soil – Tan to brown silty sand to sandy silt with rare dark gray sandy clay containing only minor amounts of fine to medium gravel. The layers are found sandwiched between single, thick lifts of rubble fill.**

3. **Soil with Rubble – Tan to brown silty sand to sandy silt with abundant coarse gravel to boulder size clasts of concrete and asphalt.**

4. **Lower Soil Fill – Tan to gray sandy silt to sandy clay contains minor amounts of fine to medium gravel composed mostly of rock with minor amounts of concrete. Well compacted and slightly moist during excavation.**

5. **Upper Soil Fill – Tan silty sand to sandy silt contains minor amounts of fine to medium gravel composed mostly of rock. Well compacted and slightly moist to dry during excavation.**

This placement pattern and lift thicknesses cast a serious doubt on validity of ZK's field test data which consistently report relative compaction of greater than 90 percent, since that compaction refers only to soil layers or soil rich matrix, and is not representative of the relative compaction of rubble fill.

Presence of Voids

Because of the large pieces of concrete/rubble and placement in thick layers, "nesting" of large fragments and presence of voids in the fill were a common phenomenon. These have frequently been reported in ZK field reports and were observed in the test trenches excavated during IGI's Phase 2 investigation. The CUP had clearly required that oversize material be either crushed or placed in windrows, surrounded by soil and that the voids between the oversize fragments be filled with granular material and densified by flooding. This was not followed in filling practice.

2.4.3 Soil Fill Cap

HAI assumed the responsibility for full-time monitoring of soil cap placement and testing for the upper 12- to 15-foot thick cap of fill to finish grade. The results of their observation and testing between January 2006 and April 2007 indicate that the soil cap consisted of silty and clayey sand with gravel and was compacted to an average relative compaction of about 94 percent based on ASTM D1557. From their description of fill materials and from photographs and logs of test pits, the fill cap is typically free of oversize and in a dense, well compacted state.

2.5 POST-FILLING INVESTIGATIONS

Following a review of the ZK summary reports listed in Table 2-1, and based on field inspection of filling operations conducted by the City of Irwindale (City) staff and their geotechnical consultants, the City expressed concern on the fill quality and placement methods. They first conveyed their concern in their letter dated May 7, 2004 and again in their letter dated January 25, 2005. Excerpts from the latter (January 25, 2005 letter from Kwok Tam to Scott Jenkins of WMI) are provided below:

> *The City's geotechnical consultant, GLA & Associates, has reviewed the observation reports (dated September 5, 1991 to July 31, 2003), prepared by Zeiser Kling Consultants for Waste Management, concerning backfilling operations at the Nu-Way Landfill. The reports are generalized, infrequent, and do not include data representing fill density testing. The descriptive commentary on general conditions and fill placement are, in every case, after the fact that a significant thickness of fill was placed unobserved. Apparently, backfilling was accomplished in the absence of fill specifications or a quality control plan. No information is presented which could enable an assessment of long-term fill performance. Although they may meet the regulatory standards of an inert landfill, the reports are well below the standard of practice for grading on geotechnical projects. We conclude,*

therefore, that these reports are not sufficient to demonstrate that the backfilled pit will be suitable for supporting structures.

In order to facilitate reclamation of the site such that building permits could be issued, the City of Irwindale is requesting the following information:

1. *A geotechnical evaluation of the backfilling completed to date and its ability to support the proposed end use. This evaluation should include a quantitative assessment of the settlement potential in the existing backfill.*

2. *If the suitability of the existing backfill cannot be adequately demonstrated, then provide a methodology for improving the backfill so that it would be acceptable.*

3. *Provide a geotechnical demonstration that the finished site will be suitable for supporting the proposed buildings, especially that the potential settlements will be tolerable.*

No action to their letter to demonstrate suitability of the site for supporting structures was taken until the site was graded to finish grade in 2007. In late 2007, the Owner initiated investigation by IGI, which was conducted in three phases discussed below.

- Phase 1 Investigation (IGI report dated March 31, 2008) – conducted from October 2007 to February 2008 and included six Becker hammer borings through fill, and geophysical surveys including two seismic reflection lines, one downhole seismic shear wave survey and three surface wave velocity surveys.

- Phase 2 Investigation (IGI report dated December 8, 2008) – conducted in August 2008 and included excavation of two large test pits to a maximum depth of 70 to 75 feet bgs and conducting large size in situ ring density tests in debris fill and sand cone density tests in fine grained matrix.

- Phase 3 Investigation (IGI report dated April 27, 2010) – drilling and video logging of a 3-foot diameter boring drilled below Bench 3 (approximately drill pad El. 370 feet) of Test Pit 2 to a depth of 70 feet (bottom of hole approximate El. 300 feet).

The Phase 1 investigation included evaluation of fill characteristics by indirect methods (Becker hammer blowcounts and shear wave velocity), while Phase 2 and Phase 3 investigations results directly reflect the nature of inert debris fill materials by direct physical observation of fill materials. Phase 2 and Phase 3 results were, therefore, predominantly used for interpretation of fill characteristics for site suitability evaluation.

The results of Phase 2 investigations which included open excavation and logging of the exposed faces to a depth of about 65 feet bgs are best illustrated in photographs included in Appendix A. IGI's description of the exposed fill conditions is highlighted below:

> *"It is clear from visual observations of Test Pits 1 and 2 that the fill exposed in the upper 50 or more feet was not processed and the deposits were essentially buried in piles dumped from trucks. Fill lifts are clearly visible in the walls of the test pits. In general, the tops of the rubble lifts are identified by a level to gently sloping soil caps. Lifts thicknesses are visible that vary from a few feet thick to more than 8 feet thick. The lifts also reveal little processing and spreading. Individual piles of debris that were apparently "end-dumped" in the landfill are surrounded by other end-dumped piles and in turn buried by additional lifts. Nesting and voids were observed and common. Nesting and bridging were primarily observed adjacent to very large oversize fragments and where oversize fragments were concentrated. Nesting and bridging also occurred where layers of drywall covered concrete fragments."*

Large-scale ring density tests indicated relative compaction values ranging from 88 to 95 percent at six of the 8 locations, and 60 to 64 percent at the other two locations. The high relative compaction values at six locations are not indicative of "well compacted" material but are rather misleading and influenced by a very large percent of irreducible concrete fragments. The interstitial pore spaces/voids were filled with "loose" matrix materials. When sandy material was tested by small-scale tests (6-inch diameter sand cone), the relative compaction of the matrix was almost always less than 90 percent (with the exception of one of eight tests that showed 94 percent relative compaction), with values ranging from 74 to 88 percent. These observations, combined with very fast rate of water percolation and fines migration observed in the ring density test holes, confirmed that the voids were either open or loosely filled.

IGI's Phase 3 investigation that included drilling and video logging of a 70-foot deep, 3-foot diameter bucket auger hole from the bench of Pit 2 (surface El. approximately El. 370 feet) revealed that the material to a depth of 52 feet below bench elevation (to approximate El. 318 feet) *"was very difficult to drill due to caving, oversize concrete fragments and rebar. The fill below 52 feet was uniform, mostly devoid of oversize rubble fragments and easy to drill."* This elevation roughly coincides with the pit backfill surface at Year 1998/99 filling. It is thus interpreted that the inert debris fill placed above the 1998/99 fill elevation, i.e., above approximate El. 310-320, may represent highly uncontrolled fill conditions. These include presence of excessive large size concrete fragments, and presence of open voids/loosely backfilled voids, caused by excessively large lift thicknesses, and "blanket type" filling process comprising 8- to 10-foot thick layers of end-dumped debris fill capped with a thin lift of soil matrix. The behavior of this fill under seismic loading, groundwater fluctuations and potential fines migration will likely be "at best" unpredictable and settlements difficult to quantify. This is discussed in detail in Section 3.0 of this report.

2.6 SUMMARY OF OVERALL FILL QUALITY AND EVALUATION OF CUP COMPLIANCE

The 1990 Quarry Rehabilitation Plan and the 1994 CUP laid out geotechnical requirements for backfilling Nu-Way Live Oak inert debris pit with engineered fill such that the backfilled site will be suitable for future development. Based on the preceding discussions, our comments on the actual material quality, filling methods, and frequency of testing and reporting versus the City requirements as laid out in the 1990 Agreement and 1994 CUP are summarized below. The ZK list of progress reports from 1991 through 2006 are provided in Table 2-2.

Parameter	CUP or 1990 Agreement Requirement	CUP Compliant? (Yes/No)	Remarks/Reason
Material Quality and Placement Methods	1. Clean earth and inorganic solid fill material, no organic material	Not Sure	Frequent reference to disposal of organic materials, plasterboard, partially treated sludge (later reclassified as water treatment solids) in daily reports
	2. Oversize material (+12-inch) to be crushed or set in windrows	No	Crusher operation intermittent, no crushing after 2003, oversize material placed in horizontal thick blanket fills (5' to 8' thickness), not in windrows
	• City to be notified 48 hours prior to windrow disposal, to approve locations and disposal methods	No	Some windrow disposal discussed in field reports in early placement (1991-1993) but City not notified
	• 90 percent compaction	No (particularly after 1994)	Only matrix material tested for in situ density, not appropriate for oversize fragments
	• Oversize disposal such that nesting does not occur and oversize is completely surrounded by compacted or densified fill	No	Frequent mention of nesting and "voids" in the fill in ZK reports and in test pits excavated by IGI

Parameter	CUP or 1990 Agreement Requirement	CUP Compliant? (Yes/No)	Remarks/Reason
Supervision and Fill Control	1. Provide proper supervision and control of project at all times (1990 Reclamation Plan)	No	Supervision too infrequent, (particularly after 1998), to properly control material quality
	2. Monitor the material to ensure that such materials meet the discharge specifications as per RWQCB's waste discharge requirements	Not Sure	Do not know if daily inspection records of incoming loads were kept
Geotechnical Progress Reports	1. Annual reports or whenever 5 feet of fill is placed, whichever comes first	No	Initially through 1994, ZK reports submitted on quarterly basis. After 1995, frequency of reports 1 in 2 years or 3 years (at frequency of fill placement of about 1 to 2 million cu. yds. or after 30 to 40 vertical feet of fill placement). See Table 2-2.

From overall CUP and Quarry Rehabilitation Plan compliance standpoint, the fill placement from 1991 through 1994, that was inspected and monitored approximately at one visit per week and tested at an average frequency of 1 test per 3,000 cu. yds. (Figure 2-2), may be considered as regulatory compliant and engineered fill. The fill placed between 1994 and 1998 had monitoring frequency of approximately 1 visit per month and an average test frequency of about 1 test per 10,000 cu. yds. The supervision is considered generally inadequate and fill quality perhaps "marginal." However, some benefit of doubt may be granted due to the reported crusher operation and based on IGI's observation of downhole logging of 3-foot diameter bucket auger boring which indicated that fill below El. 318 feet was relatively easy to drill with no large fragments. The fill placed above approximate El. 320 feet, i.e., in upper 70 to 80 feet of the pit (with the exception of the upper soil cap) is considered unsuitable to function as engineered fill. This interpretation is based on:

- Actual visual observation of fill in two test pits to a depth of about 70 feet;
- Infrequent to virtually no monitoring beyond 1999;
- Placement pattern in 8- to 10-foot thick "blankets"; and
- Frequent occurrence of open voids or voids with loosely backfilled soil matrix.

2.7 HISTORIC GROUNDWATER LEVELS

Historic groundwater levels observed in the groundwater monitoring wells and piezometers in the vicinity of the site suggest that the groundwater levels fluctuate at the site and vicinity. The historical groundwater high of 330 feet msl at the site (80 feet below finish grade) reportedly occurred in 1944 (IGI, 2008a). Groundwater data from a well 4218C located northwest of the site indicates that fluctuations are common, the groundwater levels fluctuating between El. 200 and 320 feet msl.

3.0 SETTLEMENT ANALYSIS

3.1 GENERAL

This section presents the static and seismic settlement analysis of the fill based on detailed evaluation of existing fill characteristics as interpreted from the review of available data described in Section 2.0. The results of the settlement analyses are utilized to arrive at remedial recommendations to render the site suitable for proposed development.

There is a significant degree of uncertainty associated with the settlement estimates at the Nu-Way pit due to the following reasons:

- Uncertainty related to subsurface profile and material characteristics due to the non uniform, uncontrolled nature of fill placement and limited subsurface information available
- Uncertainty regarding the various mechanisms of static and seismic settlement within rubble fills and the lack of an industry-accepted settlement calculation procedure for such fills
- Lack of case histories in such fills with documented performance under seismic loading
- Lack of settlement monitoring information for this site or other gravel pits in the vicinity

Due to these reasons, a reliable quantitative estimate of settlement is difficult to provide. The approach taken herein is to provide a range of anticipated settlements, supported by a detailed evaluation of fill conditions, field test data and placement practices at the site, a rational settlement model, reasonable assumptions and parametric analyses.

3.2 CHARACTERISTICS OF THE FILL

3.2.1 General

Based on historic evidence, reports of fill placement and results of previous investigations described in Section 2.0 of this report, the fill may be divided into four general vertical zones for purposes of analysis. These zones comprise, from top to bottom:

Zone 1: Compacted soil fill cap extending from the surface to a depth of 10 to 15 feet. The material consists predominantly of well compacted sandy materials with minor amounts of gravel. This fill was reportedly placed in 2006 under the observation and testing of HAI (2006).

Zone 2: Predominantly rubble fill extending down from the bottom of the soil fill cap to approximate elevation of 310 to 320 feet msl, representing the fill horizon around 1998/1999. This fill was placed from 1998/1999 to 2005. Observations in deep test pits and borehole excavated by IGI (2008b, 2010) indicate that the fill in this zone appeared to be unprocessed and loosely dumped, with lift thicknesses in the range of 5 to 8 feet. Fill placement was very infrequently observed and tested by ZK, and fill placement records suggest that the fill in this depth zone was placed with little or no control with respect to material processing, lift thickness, maximum particle size or compaction.

Zone 3: Rubble fill extending down from the bottom of Zone 2 (approximate El. 310 to 320 feet msl) to approximate El. 285 feet msl, representing the approximate fill horizon in 1991. Records suggest that this fill, placed from 1991 to 1998/1999, to approximate El. 310 to 320 feet was placed under more controlled conditions than Zone 2. Field observation and testing was performed by ZK at more frequent intervals than in Zone 2. Records indicate that the lift thicknesses were on the order of 3 to 4 feet, and that a crusher was used intermittently to process some of the fill materials.

Zone 4: This zone extends down from approximate El. 285 feet msl (representing the 1991 horizon) to the bottom of the pit. The fill consist of saturated silt deposits comprising sands, silts and clays mixed with varying amounts of uncontrolled rubble fill. The fill thickness is estimated to range from 5 to 40 feet (Zeiser, 1991).

The elevations of the horizons separating each of the above zones are very approximate. Also, the demarcation between Zone 2 and Zone 3 is not well defined, and is interpreted from discontinuous and infrequent fill observations by ZK.

Zone 1 representing compacted engineered soil fill is not considered vulnerable to settlement. The Zone 4 materials consisting predominantly of saturated silts are considered vulnerable to long term consolidation and creep settlement under the fill loads. However, much of this settlement has already occurred during the 15-year filling period and another 4 years since fill was completed to finish grade. The characteristics of the Zone 4 soils were previously tested and evaluated by Zeiser (1991). The focus of the current evaluation is mostly on the rubble fills of Zones 2 and 3.

3.2.2 Rubble Fill Characteristics

<u>**Stratigraphy**</u>

Two large test pits excavated to maximum depth of 75 feet (IGI, 2008b) indicated that the rubble fill in Zone 2 consists of individual lifts of rubble fills (typically about 5 to more than 8 feet thick), each capped by a thin (4 to 6 inches) layer of soil. This pattern appears to be repeated over the full depth of the face exposed in both test pits. The layered nature of the fill is clearly visible in the photographic test pit logs for each test pit produced by GLA (Appendix A).

The rubble lifts consisted primarily of concrete with abundant rebar, floor tile, cement and asphalt shingles, bricks, soil, crushed glass and debris. Approximately 15 to 20 percent of the rubble fill was estimated to be oversize (larger than 12 inches). Concrete blocks with dimensions ranging from 2 feet to more than 20 feet were found. Nesting and voids were common. Most of the blocks were found to be nested with loose dry infill. Extensive rubble zones were reported to be voided with no soil fill. It appeared that the rubble layers were formed by end dumping debris from trucks into layers 4 to 5 feet thick. The lifts revealed little processing or spreading. Each individual lift then appears to have been capped by a thin layer of silty sand or sandy silt (hereafter referred to as "sand layers" for convenience).

The layered nature of the rubble fill appears to be confirmed by the blowcount patterns from the six Becker Hammer borings performed through the fill by IGI (2008a). The corrected Becker Hammer blowcounts, and equivalent SPT blowcounts corrected for overburden pressure (Appendix B) show the presence of periodic thin layers (no more than a foot thick) with relatively low blow counts, indicating a relatively loose to medium dense sand layer, at regular intervals within the profile. The intervals generally range from 5 to 15 feet in Zone 2. Considering that some of the sand layers sandwiched between successive layers of rubble fill may be too thin to be reflected in the blowcount data (which represent blows measured per foot of driving), the repetitive pattern of the low blowcounts appear to match the layers of sand seen in the test pit photo-logs. The layered nature of the rubble fill/sand layer is very prevalent within Zone 2, and is less prevalent in Zone 3.

In-between the successive thin layers of sand, the blowcounts fluctuate significantly within the rubble, typically between equivalent $N_{1(60)}$ values of 15 and 30, and periodically spiking up to higher values, sometimes in excess of 50 or 60. The spikes are interpreted to represent the presence of oversize materials, as the 8-inch diameter closed end Becker Hammer bits displace or break through the larger clasts.

The layered nature of the rubble fill (5 to 8-foot thick voided rubble zones interbedded with thin sand layers at regular intervals) as observed in the test pits and interpreted from the blowcounts, would appear to influence the mechanisms of settlement, particularly under seismic loading or fluctuations of the water level. This is discussed in more detail in subsequent sections.

Grain Size Distribution

Bulk gradation tests performed on eight large samples obtained from various depths from the two IGI (2008b) test pits gave the following results. All of the samples obtained were from the upper 45 feet of fill, within Zone 2.

Material Size	Range (%)	Average (%)
Boulders (>12")	3 to 23	11
Cobbles (>3")	10 to 25	18
Gravels >¾"	6 to 20	14
Finer than ¾"	44 to 66	57

The bulk gradation tests were performed on samples excavated from the 6-foot diameter by 4- to 5-foot deep bulk density test pits, and do not contain the representative amount of the larger clasts. Therefore, the above gradation ranges are considered to underestimate the large cobble and boulder size fraction. IGI confirmed (verbal communication) that they could not include representative amounts of oversize in the bulk gradation sample.

A rough estimate of the oversize content was also made from the Becker hammer blowcount data profiles. As discussed above, the blowcount spikes likely represent the presence of materials larger than the size of the Becker Hammer bit. The relative fraction of the plus 9-inch clasts in the fill (by volume) was roughly estimated from the six Becker Hammer blowcount profiles to be an average of 24 percent (Appendix B). By weight, the plus 9 inch fraction is estimated to be roughly 31 percent.

Another estimate of oversize content by volume was obtained by overlaying a scaled grid over a scaled photograph of the test pit face (obtained by GLA), and measuring the relative area occupied by the visible oversize clasts. This method indicated the plus 9 inch clasts to be approximately 18 percent by volume. The corresponding fraction by weight is estimated to be 23 percent.

The Becker Hammer interpretations and oversize estimate from photographs indicate that the plus 9-inch fraction is roughly 23 to 31 percent by weight. By comparison the laboratory grain size distribution data indicates a plus 9 inch fraction of about 15 percent. Thus the laboratory gradation curves reported by IGI, likely underestimate the oversize clasts by approximately 12 percent.

Relative Compaction

The relative compaction of the rubble fill was evaluated by IGI (2008b) by performing a total of eight large diameter (6-foot diameter) ring density tests (ASTM D5030-04), at various depths within the two test pits. The tests, four in each pit, were performed on benches at depths ranging from 13 to 55 feet below finish grade. All of the tests were within Zone 2. The bulk density values are summarized in Table 3-1. The measured bulk densities (moist densities) range from 79.7 pcf to 133.6 pcf, with an average value of 117 pcf. The corresponding dry densities were also calculated by IGI, by subtracting the weight of water contained in the minus ¾-inch fraction from the total weight of the material. The plus ¾-inch fraction consisting of concrete fragments, brick and steel were not considered to contain appreciable moisture. The corresponding dry density values range from 76.6 pcf to 126.4 pcf, with an average value of 110.4 pcf.

Of the 8 tests, four had dry densities between 120 and 126 pcf, two had dry densities between 110 to 120 pcf, and two had very low dry densities of 77 and 86 pcf respectively. In comparison, when these materials were compacted in multiple, maximum achievable density (MAD) test pads, the dry density values ranged from 127 to 133 pcf. Field notes recorded during the in situ field density testing suggest the presence of a large volume of voids in the rubble fill. Water percolation tests performed in the field density test holes supported this observation. The water from the ring density tests reportedly completely drained or substantially dropped in a matter of minutes at most of the test locations, when the visqueen sheet lining the bottom of the pit was perforated and removed. In contrast, the water in the MAD Test Pad pits reportedly remained undrained for several days, indicating a low permeability and significant reduction in voids when the materials excavated from the density test pits are properly compacted.

A total of 8 sand cone tests performed in exposed layers of the relatively finer grained (sandy) materials showed relative compaction values (with respect to ASTM D1557) ranging from 72.5 to 94 percent, with an average of about 81 percent (IGI, 2008a). The sand cone tests could have been located either within the sand layers capping rubble lifts or within relatively finer grained fill materials (soil rubble mix with relatively low oversize content). The tests indicate that regardless of whether they are sand layers between rubble layers or infill materials within the rubble fill, the infill material is relatively loose.

3.3 SETTLEMENT MODEL AND METHODOLOGY

The idealized soil profile used for settlement estimates is illustrated in Figure 3-1. Because Zone 2 and, to a lesser extent, Zone 3, were placed in an uncontrolled manner with little or no control of maximum particle size, processing, lift thickness or compaction, the fill is anticipated to experience significant static and seismic settlements. Also, because of the non-homogeneity of the fill, the differential settlements are anticipated to be a significant proportion of the total settlements.

3.3.1 Methodology for Seismic Settlement

The typical approach to seismic settlement of granular unsaturated fills consists of calculating the contractive volumetric strains in the material induced by seismic shaking and integrating it over the thickness of fill. The current state of practice consists of the following steps (Tokimatsu and Seed, 1987):

- Estimate cyclic shear stresses induced by the design earthquake. This is a function of the maximum horizontal ground acceleration.

- Estimate corresponding induced cyclic shear strains (γ). In order to calculate the induced shear strain, the shear modulus G should be known. G is strain dependent (decreases with increasing cyclic shear strain) and can be calculated based on the maximum shear modulus (G_{max}), i.e. the shear modulus corresponding to very low values of shear strain, and the relationship between

G/G_{max} and γ. The value of G_{max} can be estimated from the $(N_1)_{60}$ values (based on established correlations between G_{max} and $(N_1)_{60}$) or calculated from the shear wave velocity. The relationship between G/G_{max} and γ is obtained from experimental data available in the literature for different types of soils

- Calculate induced volumetric strain (ε_v). The induced volumetric strain is a function of the magnitude of the cyclic shear strain γ, and the number of significant cycles (which is a function of earthquake magnitude). This relationship can be established based on laboratory cyclic shear tests on the material. Relationship between γ and ε_v for sands have been established based on laboratory tests on sands compacted to different values of relative density (Silver and Seed, 1971; Pyke et al, 1975). The relative density of sand can in turn be correlated to the $(N_1)_{60}$ values.

- Integrate the calculated volumetric strain over the fill thickness to calculate total settlement.

The Tokimatsu and Seed (1987) procedure and the simplified version of the procedure implemented by Pradel (1998) are typically used to estimate the seismic settlements of loose sands and granular fills under unsaturated conditions. The seismic settlement estimates are based on the $(N_1)_{60}$ blowcounts in the deposit and the laboratory charts developed for sands. The validity of this approach for unsaturated sandy fills has been demonstrated by comparing predictions to observed settlements during earthquakes in Southern California (Pradel, 1998; Tokimatsu and Seed, 1987; Stewart et al, 2001).

The limitation of the Tokimatsu and Seed (1987) approach is that the charts used in the calculations are based on laboratory tests on uniformly compacted clean sands, and on correlations between relative density and SPT $(N_1)_{60}$ values for sands. The method is, therefore, considered inappropriate for non-uniform, voided and nested rubble fills containing significant proportion of oversize clasts. Also the method considers only the contractive volumetric strains induced by shear strains, and does not consider other potential seismic settlement mechanisms such as migration of fines into open voids and collapse of nested structure that may be prevalent in voided and nested rubble fills.

As described previously, the rubble fills at the Nu-Way Pit consist of a succession of loose, voided and nested rubble lifts 5 to 8 feet thick capped by an approximately 6- to 12-inch layer of sand. The voids between the clasts in the rubble fill consist of finer infill materials and open voids. The sand layers capping the rubble lifts, as well as the infill materials within the rubble fill are relatively loose (average relative compaction of 81 percent, based on sand cone density tests). During seismic shaking there will be a strong tendency for the loose sand in each of the sand layers to be shaken into the underlying open voids in the rubble layer (Figure 3-1). A simple visual illustration of this mechanism can be demonstrated by filling the lower portion of a glass jar with gravels and cobbles of various sizes to give a nested structure, capping it with a layer of sand, and shaking the jar a few times to simulate earthquake shaking. The sand will tend to migrate and occupy the underlying voids, resulting in a net reduction in thickness of the sand layer. The seismic shaking will also cause the loose infill materials within the rubble to settle, creating more open voids and increasing the connectivity between the open voids, thereby further encouraging the migration of sands from the overlying layer. As a result of the sand migrating into the underlying rubble fill, there will be a reduction in thickness of the sand layer, which will cause the overlying layers to settle. The cumulative effect of the reduction in layer thickness of each sand layer will translate into an area-wide settlement of the fill. Another potential mechanism during seismic shaking will be the collapse of the nested structure of the oversize clasts. The collapse may occur where oversize clasts are in point to point contact or separated by loose sands which are displaced by the seismic shaking. The collapse mechanism, along with the migration of sands into the open voids, will result in a net reduction in the open voids within the rubble fills, which would then translate into overall settlement of the fill.

This settlement model that represents the actual stratigraphy of the rubble deposit based on the reported placement history and mapped condition of the upper 50 to 60 feet of rubble fill, is considered a better representation of the actual site conditions to calculate settlements rather than any other models that assume uniform, homogeneous fill characteristics.

Based on the above discussion, the seismic settlement of the rubble fill is estimated to consist of the following components:

1. *Densification of sand layers in-between the rubble layers* – The cumulative thickness of the sand layers, in comparison to overall thickness of rubble fill, is generally relatively small; therefore, the contribution to total seismic settlement will be small. The magnitude of settlement can be calculated using the Tokimatsu and Seed (1987) approach, by considering the equivalent $(N_1)_{60}$ values measured in the sand layers.

2. *Densification/settlement of the infill soil within the rubble layers* – This will not directly cause a settlement of the entire deposit. Instead, the densification of the infill soil will create additional open voids within rubble fill, which will then become available to be filled by migration of sands from the overlying sand layer, or cause collapse of the rubble fill skeleton. Since the pre-existing free voids in the rubble layer far exceed the additional voids that may be created as a result of the densification of the infill, this component was neglected in the settlement estimates.

3. *Densification of the rubble fill* – The rubble fill mass acting as a "homogeneous fill" will tend to densify and settle due to the effect of cyclic loading, i.e. the contractive volumetric strain mechanism modeled by the Tokimatsu and Seed (1987) approach. The Tokimatsu and Seed charts cannot be reliably used for this material, because the charts were developed for sands. By obtaining the G_{max} of the material from the average shear wave velocity profile for the fill, and assuming that the charts developed for sands are applicable to the rubble fills, a rough estimate of

the settlement contribution from such densification may be made. The contribution to total settlement from this component is estimated to be small.

4. *Filling of open voids within the rubble fill* - There are two mechanisms that will contribute towards the filling of open voids in the rubble fill:

    i. Migration of sands from the sand layer into the open voids of the underlying rubble layer – This process will be repeated at each sand layer/rubble layer contact.

    ii. Collapse of the nested structure prevalent within the rubble fill

These two processes together will cause a net reduction in the open voids, which will translate into settlement of the fill. The volume of open voids in the rubble layer can be calculated based on the dry density of the rubble layer (from the ring density tests), the dry density of the infill soil (from sand cone tests), the relative proportions of infill materials to oversize clasts and their respective values of specific gravity. Not all of the available open voids will be filled due to migration of sand from the overlying sand layer or collapse of the nested structure. The proportion of open voids that get filled will depend on the amplitude, frequency and duration of shaking, as well as proportion of open voids that are easily accessible to the fines (i.e. not blocked by infill materials or isolated by nesting clasts). In the absence of physical laboratory modeling, there is no reliable means of estimating the proportion of open voids that will get filled due to a given level of shaking. Therefore, a range of values will be estimated to provide an order of magnitude quantification of the settlement.

3.3.2 Methodology for Static Settlement

The static settlement of the rubble fill will consist of the following components:

1. *Static settlement under the self weight of the fill* – In granular fills the majority of the settlement occurs during and immediately after filling, and is therefore not an issue for long-term settlements.

2. *Creep settlement* – In granular fills (Zones 1, 2 and 3) creep settlement is not significant

3. *Long-term primary and secondary consolidation settlement of fine grained layers (Zone 4)* – An estimate of the long term settlement of the saturated silt fills was made by Zeiser (1991), based on laboratory consolidation tests. They estimated a total primary settlement of 24 to 36 inches under the fill load, 90 percent of which would occur within 1 year after completion of grading. Since the total fill has been in place for over 4 years since finish grading, the primary consolidation

settlement of the silt deposits can be considered to be substantially complete. Any remedial grading measures (removal and replacement of the overlying fills) would produce reloading and re-trigger settlements in this deposit. However, the silt deposit would behave as overconsolidated soils, hence the primary settlements would be an order of magnitude lower and occur rapidly during reloading. Long-term secondary settlements will also be lower. The magnitude and rate of secondary settlement and the corresponding waiting periods prior to structure construction can be estimated based on settlement monitoring of the replaced fill.

4. *Migration (washing) of fines into voids and potential collapse of the nested structure due to water table fluctuations and surface water infiltration* – Future water table fluctuations could result in overlying sands and fines being washed into the open voids of the underlying rubble layers. The potential for this occurrence was clearly observed in the water percolation tests performed by IGI (2008b) in the ring density test pits. The potential for settlement under this mechanism will be significant, where the pattern of interlayered sand and rubble layers is present in the groundwater fluctuation zone. Groundwater fluctuations could also cause hydroconsolidation of the infill materials resulting in collapse of the nested structure. Since the groundwater fluctuation zone is very deep (the high water level is estimated to be approximately 80 feet below ground surface), the surface manifestation of such settlement will be relatively small.

   There could be additional localized settlement due to surface water infiltration causing piping. However, since a significant thickness of the upper part of the rubble fill will be replaced with a properly compacted fill and surface drainage will be provided as part of the site development, the potential for piping will be significantly reduced.

3.4 SEISMIC SETTLEMENT

Based on the 2007 California Building Code, the design ground motions for the site is estimated to be a peak ground acceleration of 0.53g. The corresponding moment magnitude was estimated at 6.7 based on a deaggregation analysis performed using the USGS interactive website.

As discussed in Section 3.2.1 above, the seismic settlements include a combination of the following:

- Densification of the sand layers under seismic shaking
- Densification of the rubble fill layers under seismic shaking
- Settlement caused by filling of open voids due to fines migration from overlying sand layers and collapse of the nested structure

Seismic settlement estimates were made at each of the six Becker Hammer boring locations, B1 through B6. The blowcount profiles used in the interpretation of the subsurface stratigraphy at each of these locations are included in Appendix B.

3.4.1 Settlement of Sand Layers

The densification of the thin sand layers at each Becker Hammer location was estimated using the Tokimatsu & Seed (1987) procedure as implemented by Pradel (1998). Only the layers where the equivalent $(N_1)_{60}$ values were less than 15, indicating lose to medium dense sands, were considered for this settlement estimate. Only the layers above El. 285 feet (above the 1991 horizon) were considered. The cumulative thickness of the loose to medium dense sand layers (encountered by the Becker Hammer) ranged from zero at B4 to approximately 38 feet at B6. The G_{max} value required for the analysis was estimated from the $(N_1)_{60}$ values. The shear wave velocity profiles obtained from downhole and surface seismic surveys (IGI, 2008a) could not be used for this purpose because they did not have sufficient resolution to identify presence of the thin sand layers. The calculated seismic settlements, as tabulated in Table 3-2, range from zero to 2.3 inches.

3.4.2 Settlement of the Rubble Layers

The densification (volumetric contraction) of the rubble layers at each Becker Hammer location was also estimated using the Tokimatsu & Seed (1987) procedure as implemented by Pradel (1998). As discussed in Section 3.2.1, although the applicability of this method and the reliability of the predictions are questionable for rubble fills, it was used, nevertheless, to obtain rough order of magnitude estimates. The sand layers considered in Section 3.4.1 above were excluded from the analysis. Only the rubble layers within Zones 2 and 3 were included. The cumulative thickness of rubble fill layers estimated from borings B1 through B6 ranged from 72 feet to 110 feet. The G_{max} value required for the analysis was estimated from the shear velocity profile obtained from the downhole seismic survey performed in Borehole B4 (IGI, 2008a). The calculated seismic settlements, as tabulated in Table 3-2, range from 0.3 to 0.5 inches.

3.4.3 Settlement Caused by Filling of Open Voids (Cumulative Settlement of the Sand Layers Due to Fines Migration + Collapse of Nested Structure)

This analysis involved calculating the available average open voids (as a percentage of the total volume) in the rubble layers and then calculating the settlements assuming that a certain percentage of the open voids get filled by a combination of fines migration and collapse. The total voids in the rubble fill have two components: voids contained within the infill soils $(V_v)_s$, and free (open) voids $(V_v)_f$,. For purposes of this analysis, materials finer than ¾ inch were considered infill soils, while the plus ¾ inch fraction comprised the clasts of the rubble fill.

Knowing the void ratio of the infill soil component, the void ratio of the entire rubble fill, and the ratio of the volume of soil (minus ¾-inch fraction) to volume of clasts (plus ¾-inch fraction), the volume of open voids can be calculated (Figure 3-2). The average void ratio of the infill soil was estimated from the sand cone tests reported by IGI (2008b). The void ratio was calculated using the average sand cone dry density values and assuming a specific gravity (Gs) value of 2.65 for the material. The void ratio of the entire rubble fill was calculated by considering average dry density of the rubble fill as determined from the 8 large scale ring density tests reported by IGI (2008b). The volume ratios of infill soil to rubble clasts were obtained knowing the ratio by weight of the minus ¾-inch fraction to plus ¾-inch fraction from the gradation tests discussed in Section 3.1.2.2, and assuming Gs values of 2.65 for the minus ¾ inch fraction and 2.4 for the rubble (that includes predominantly concrete clasts).

Based on the above evaluations, the average volume of open voids (as a percentage of the total volume) in the rubble layer is estimated to be 6.7 percent (Table 3-1). Due to the pervasive nature of open voids, the settlement due to fines intrusion into the open voids below and collapse, will be an area-wide phenomenon. However, due to the variability of the open void volume, the settlement will be highly non-uniform.

The settlement due to filling of open voids will be prevalent where the fill pattern of thick loosely dumped rubble layers interlayered with thin sand layers exists. The vertical interval over which the interlayering occurs was estimated based on the blowcount profile in each boring (Appendix B) and is listed in Table 3-2. The cumulative thickness of the affected vertical interval ranges from 52 feet at B5 to 82 feet in B3. Not all of the available open voids within this depth interval will be filled due to sand migration or collapse. As indicated in Section 3.2.1, the proportion of open voids that get filled will depend on the amplitude, frequency and duration of shaking, as well as the proportion of voids that are accessible to migrating fines, but is not known in the absence of specific physical laboratory testing. If the percentage of open voids that get filled due to the design ground motions was assumed to be 'p' percent, the corresponding settlement (due to fines migration and collapse) is estimated as:

$$\text{Settlement} = (p/100) \times 0.067 \times T$$

where 'T' is the cumulative thickness of the affected interval, and 0.067 (6.7%) is the calculated average fraction of open voids (as a fraction of the total volume).

In the case of B4 (Appendix B), even though the layering appears to be prevalent over a depth interval of 80 feet, the sand layers appear to be significantly denser, based on blowcounts. Since dense sand will not migrate as easily as loose to medium dense sands into the open voids below, the percentage of open voids that get filled was assumed to be 0.4 x p for B4, based on a review of the blowcount profile.

As an initial estimate it was assumed that 20 percent of the open voids get filled due to the design earthquake, i.e. p = 20 percent. The corresponding settlements due to filling of open voids are then calculated to range from 5.2 inches at B4 to 13.2 inches at B3 (Table 3-2).

3.4.4 Total Seismic Settlements

The total seismic settlements estimated at each borehole location (assuming that 20 percent) of the open voids are filled, ranges from 5.6 inches to 14.4 inches with an average of 11.2 inches (Table 3-2). The total thickness of rubble fill (above the 1991 horizon, i.e. above Zone 4) is approximately 110 feet. The total seismic settlement of 11.2 inches is roughly equivalent to 0.85 percent of fill thickness.

The above estimate was compared to some well documented case histories related to the settlement of dry granular fills in Southern California, during the 1971 San Fernando earthquake and the 1994 Northridge earthquakes. During the Magnitude 6.6 San Fernando earthquake, a 40-foot deep compacted sandy fill at the Jensen filtration plant reportedly experienced a settlement of approximately 4 inches (0.8 percent of fill thickness) under an estimated maximum ground acceleration of 0.45g (Pike et al, 1975). Stewart et al (2001) report numerous cases of settlement of compacted dry fills that resulted in widespread damage to foundations from the 1994 Northridge earthquake. Pradel (1998) reports a case history from the same earthquake where 3.2 inches of differential settlement occurred in a granular compacted fill over a 30-foot differential fill thickness under an estimated maximum ground acceleration of 0.5g (approximately 0.9 percent of fill thickness). Stewart et al (2004) report 3.9 inches of settlement in 54 feet of fill (approximately 0.6 percent of fill thickness) and 6.7 inches of settlement in 61 feet of fill underlain by 22 feet of dry alluvium (approximately 0.7% of combined fill + alluvium thickness) at a well documented site with sandy clay/silty sand fill during the 1994 Northridge earthquake. These documented case histories indicate that seismically induced settlements of 0.6 to 0.9 percent of fill thickness are not unusual in dry sandy fills, for ground accelerations comparable to the design ground accelerations for the Nu-Way Pit. These observations of seismic settlement as a percentage of the fill thickness, compares well with the above estimate for the rubble fill. However, considering that the rubble fill consists of uncompacted fill with significant voids, the resulting settlement could possibly be higher than the estimate above.

One of the likely remedial measures for the substandard fills at the Nu-Way Pit would consist of partial removal of the Zone 2 and possibly Zone 3 fills and replacement with a properly processed and compacted fill cap. With increased removal depth, the remaining fill thickness vulnerable to seismic settlement would decrease, thereby resulting in lower seismic settlement potential. Figure 3-3 provides the results of seismic settlement estimates for increasing levels of removal and replacement. The results are tabulated and plotted in terms of total seismic settlement versus thickness of removal and replacement (compacted fill cap thickness). The compacted fill cap is assumed to experience no seismic settlement. The currently existing condition is represented by an existing compacted soil cap thickness of approximately 10 feet. The plot in Figure 3-3 shows the calculated settlement versus thickness of removal and replacement at each of the six borehole locations, and the average curve for the site.

Under current conditions the site is expected to have total seismic settlements ranging from 5.6 to 14.4 inches with an average of 11.2 inches. If, for instance, the upper 70 feet are removed and replaced, the estimated seismic settlements would range from 0.1 to 3.9 inches with an average of 1.8 inches.

The above estimates (Figure 3-3) were made assuming that the percentage of free voids that would be filled by fines migration and collapse (p) is 20 percent. To quantify the uncertainty associated with this assumption and evaluate the sensitivity, the value of p was varied from 0 to 30 percent (0, 10, 20 and 30 percent). The average settlement curves for various values of p are presented in Figure 3-4. The plots indicate that the seismic settlements are significantly impacted by the assumed value of p. The corresponding settlements (assume 70 feet of removal and replacement) for 10 percent and 30 percent filled voids are estimated at 1 inch and 2.6 inches, respectively (Figure 3-4).

3.5 STATIC SETTLEMENTS

As discussed in Section 3.2.2, the static settlement of the rubble fill under self weight takes place during and immediately following fill placement, and the long term settlement of the underlying silt deposits (when retriggered) can be estimated (projected) based on settlement monitoring data from settlement plates/monuments. Based on projected settlements, the waiting periods for start of foundation construction can be estimated and accommodated in scheduling start of structure construction.

The remaining component of static settlement consists of settlement due to groundwater level fluctuations. Historical groundwater data suggest that the groundwater levels fluctuate at the site and vicinity. The historical groundwater high of 330 feet at the site reportedly occurred in 1944. Groundwater data from a well 4218C located northwest of the site indicates that fluctuations are common, the groundwater levels fluctuating between El. 240 and 320 msl. The data suggest that the fill placed above approximate El. 290 feet may not have been subjected to saturation due to groundwater fluctuations. The zone vulnerable to settlement due to groundwater fluctuations is therefore the 40-foot thick zone between El. 290 feet and 330 feet. Roughly the lower half of this zone is within Zone 3. Assuming that Zone 3 does not have substantial free voids, the bulk of the settlement due to saturation would result from the upper half of this 40-foot interval within Zone 2. This zone contains interlayered sand and rubble layers and the potential for fines being washed into the underlying voids is high.

The estimated total settlement caused by groundwater fluctuations will depend on the percentage of open voids that get filled due to migration of fines and collapse (p). This is dependent on the relative proportion of the open voids that are accessible to the fines or vulnerable to collapse. Some of the open voids will be blocked by in-fill materials or isolated by nesting clasts and will therefore be inaccessible to fines migration or not vulnerable to collapse. The estimated settlements for different percentages of open voids filled are provided below.

% Open Voids Filled due to Groundwater Fluctuations (p)	Settlement (inches)
20%	3.2
30%	4.8
40%	6.4

This settlement occurs at a depth of 80 feet below the ground surface.

3.6 SURFACE MANIFESTATION OF SETTLEMENT

The static and seismic settlements estimated above occur at various depths below the finished fill surface. The surface expression of the settlement will include total and differential settlement. The magnitude of the total and differential settlements manifesting at the surface will depend on the depth at which the settlement occurs and the thickness of compacted fill cap overlying the horizon at which the settlement takes place.

The Computer Program FLAC (Fast Lagrangian Analysis of Continua, Itasca, 2008) was used to model the impact of cap thickness on surface manifestation of the underlying settlements. The program uses finite difference numerical techniques to model the non-linear stress – deformation patterns within soils. A detailed description of the FLAC analysis, including sample input files, selected output files and results of sensitivity analyses is presented in Appendix C.

A typical cross section (Cross Section C-C' from the IGI report, 2008a) was analyzed for this purpose. The soil cap, consisting of engineered fill generated from removal and recompaction of excavated materials was modeled as a non-linear elastic – perfectly plastic material with a Mohr-Coulomb yield criterion.

The initial shear modulus (G_{max}) of the soil cap was calculated from the shear wave velocity measured from the single downhole geophysical seismic velocity survey performed in Borehole B-4 (Terra Physics, 2008 and IGI, 2008a). The shear wave velocity of 880 feet/second measured in the upper portion of the existing fill was selected. The shear modulus was degraded as a function of the shear strain based on the G/Gmax backbone curve proposed for sands by Seed & Idriss (1970).

The 2-dimensional cross section was first initialized under gravity load to calculate and apply the in situ stresses. Surface manifestation of settlement was simulated by considering the soil cap and applying the calculated displacements of the existing fills left in place below the fill cap as vertical nodal displacements along the interface between the fill cap and existing fill. The intent of the analyses was to apply a differential settlement at the base of the fill cap and evaluate the corresponding differential settlement at the surface for various thicknesses of fill cap.

In the FLAC model, the nodal displacements are incrementally applied as a "velocity" (displacement per time step) for the numerical analysis. The incremental displacement for each time step should be small enough such that it does not cause numerical instability in the finite difference analysis. Parametric studies were performed to obtain the optimum region (mesh) size and displacement application rates for the numerical model (Appendix C).

The surface manifestation of settlements occurring at depth was modeled in two different ways:

a) ***Specific Differential Settlement Applied at a Single Location (isolated differential settlement)*** – In the first approach, a specific differential settlement was applied at a single location at the base of the fill cap and the corresponding response at the surface was modeled._ Since the rubble fills are highly non-uniform, differential settlements could be significant. The differential settlement due to seismic action was selected to be half of the total settlement, occurring over a relatively short horizontal distance of about 30 feet. The 30-foot distance was selected to correspond with two equipment widths. The assumption being that the fills were built in cells, each approximately 30 feet wide (approximately 2 equipment widths), and that adjacent cells could experience differential settlements of up to 50 percent of the total. The differential settlement at the base of the fill cap was estimated to be half of the estimated total average settlement (Figure 3-4) occurring in the underlying rubble fill left in place. The differential settlement was applied as a uniformly increasing displacement from zero to the estimated magnitude of differential settlement over a horizontal distance of 30 feet.

   Surface manifestation of this differential settlement was evaluated for three different thicknesses of fill cap: 40, 60 and 80 feet, respectively. The resulting surficial differential settlements corresponding to different thicknesses of fill cap are illustrated in Figure 3-5. The figure also provides the range of estimated differential settlement for different percentages (p) of open voids filled (due to fines intrusion + collapse). The results show for instance, that with a 40-foot fill cap, the estimated maximum differential settlement at the surface (over a horizontal length of 30 feet), corresponding to p = 20%, is approximately 1.8 inches. If p is varied over a range from 10% to 30%, the corresponding surficial differential settlements range from approximately 1 inch to 2.6 inches. If the fill cap thickness is increased to 60 feet, the corresponding surficial differential settlements are reduced to approximately 1 inch or less (Figure 3-5).

b) ***Randomly Varying Settlement Applied at the Base of the Fill Cap*** – In this approach, the settlement of the rubble fill underlying the fill cap was assumed to vary randomly between the maximum and minimum values calculated (Figure 3-3). For example for the case of the 40-foot fill cap, the total calculated settlements for the materials left in place below the cap (corresponding to p = 20%), ranged from 3.5 inches (at B4) to 9.2 inches (at B3). The settlement at the basal nodes was specified to vary randomly between the maximum and minimum values. This was accomplished by the use of a random number generator, whereby the specified settlement at any node was calculated as:

> Specified nodal settlement = $\rho_{min} + r\ .\ (\rho_{max} - \rho_{min})$ inches, where r is a random number between 0 and 1, and ρ_{min}.= 3.5 inches and ρ_{max}= 9.2 inches.

The model was then run to calculate the corresponding surficial settlements. For the 40-foot thick cap for the given basal settlements ranging from 3.5 to 9.2 inches, the corresponding surficial settlements ranged from zero to a maximum of approximately 7.2 inches (zero occurring at the edge of the pit in contact with native deposit). The corresponding differential settlements over any 30-foot horizontal interval ranged from < 1 inch to 2.4 inches. .

This calculation was repeated for cap thicknesses of 40, 60 and 80 feet, and for varying values of p (10% to 30%). The results are summarized in Figure 3-6. The figure shows the maximum calculated values of total surficial settlement and corresponding differential settlements at the surface. The figure indicates that with a 40-foot thick fill cap the maximum surficial differential settlements range from approximately 1.3 to 3.6 inches (for p ranging from 10% to 30%). When the fill cap thickness is increased to 70 feet, the maximum total settlements are less than 3.7 inches and maximum differential surficial settlements are less than 1 inch.

<u>Surface Manifestation of Static Settlement</u>

Total settlements ranging from 3.2 to 6.4 inches were estimated to occur due to groundwater fluctuations (assuming values of p ranging from 20 to 40 percent). As in the case of seismic settlements, differential settlements at the saturated horizon are estimated to be half the total settlement, i.e. 1.6 to 3.2 inches over a horizontal distance of 30 feet. At the surface, approximately 80 feet above the highest groundwater level (Elevation 320 feet), the corresponding differential settlements are expected to be less than 1 inch over 30 feet.

It should be noted that if the groundwater fluctuations occur prior to the design earthquake, the open voids within the groundwater fluctuation zones will be partially filled and will be less vulnerable (available) for seismic settlements. Similarly if a large magnitude earthquake occurs prior to the groundwater fluctuations, the volume of open voids available for fines intrusion due to saturation will be less. Thus it is unlikely that the settlements from seismic loading and from groundwater fluctuations will be cumulative

4.0 CONCLUSIONS AND RECOMMENDED REMEDIAL MEASURES

4.1 CONCLUSIONS

The estimated total seismic settlements of the in-place fill corresponding to various depths of removal and replacement are provided in Figure 3-4. The magnitude of settlement depends on the percentage of open voids in the rubble fill that get filled due to the design seismic event. In the absence of physical laboratory modeling, it is not possible to reliably estimate this percentage. Therefore, settlement estimates for a range of percent open voids filled are provided.

The surface manifestation analysis with fill cap thicknesses of 40, 60 and 80 feet suggests that differential settlements taking place at depth are significantly attenuated at the surface of the fill. The level of attenuation increases with increasing thickness of the fill cap. Figures 3-5 and 3-6 show the range of differential surficial settlements anticipated for different thicknesses of fill cap. The results suggest that the upper 70 feet of fill should be removed and replaced, to limit the maximum surficial differential settlements to less than 1 inch over 30 feet.

Several inches of total settlements could occur at depths below 80 feet from the surface due to fluctuations in the groundwater table. The corresponding differential settlement at the ground surface is estimated to be less than 1 inch over 30 feet. However, the settlement due to seismic shaking and due to groundwater fluctuations is not anticipated to be cumulative

The differential settlement criterion currently used by the LA County Department of Public Works (LADPW) is maximum 1 inch over a horizontal distance of 30 feet for static or seismic loading. Our analysis suggests that a 70-foot thick fill cap will be required to meet the LADPW criteria.

4.2 RECOMMENDED REMEDIAL MEASURES

The settlement analyses presented in Section 3.0, and the evaluation of fill quality as discussed in Section 2.0, suggest that approximately the upper 70 feet of existing, predominantly uncontrolled fill should be removed and replaced as a properly processed and compacted engineered fill cap, in order to limit the potential for total and differential settlements. This level of removal and replacement will remove the bulk of the Zone 2 fill, which contributes the most to static and seismic settlements, and presents the highest degree of uncertainty due to uncontrolled/undocumented filling. Alternatively, a combination of removal and replacement, and in situ densification by deep dynamic compaction (DDC) may be used to achieve a 70-foot thick compacted fill cap. The depth of effectiveness of DDC in inert debris fills is not well documented but may be estimated at 20 to 25 feet for this type of inert debris fill. Assuming a 20-foot effective depth for DDC, the 70-foot fill cap may be achieved by a combination of

50 feet of removal, followed by DDC of exposed fill and subsequent compacted fill placement to finish grades. If DDC is to be used, its effectiveness and depth of influence will have to be demonstrated by means of a pilot program and pre- and post-DDC evaluations for the test section.

It should be noted that this is an approximate estimate of the minimum cap thickness required to control the surficial settlements to within tolerable limits. Additional evaluation of fill to be left under the cap would likely be required to confirm this cap thickness recommendation.

5.0 LIMITATIONS

This Technical Memorandum (TM) has been prepared by Advanced Earth Sciences, Inc. (AES) for Mnoian Management Inc. in accordance with generally accepted geologic and geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

The subsurface conditions presented in this TM are based on subsurface investigations and field testing information reported by others. The actual subsurface conditions at the site may be different from those reported, and should be verified in the field during construction. Significant differences between the estimated and actual subsurface conditions should be reviewed by AES so that recommendations may be revised as appropriate.

The data, conclusions and recommendations contained herein should be considered to relate only to the specific project and location discussed herein. AES is not responsible for any conclusions or recommendations that may be made by others, unless we have been given an opportunity to review such conclusions or recommendations and concur in writing.

This TM has not been prepared for use by parties other than Mnoian Management, Inc. It may not contain relevant information for the purposes of other parties or other uses. If any changes are made in the project as outlined in this TM, the conclusions and recommendations contained in this TM shall not be valid unless the changes are reviewed and the conclusions and recommendations herein are modified or approved in writing by AES.

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Tables

Table 1
Yearly Fill Operations Summary
Nu-Way Live Oak Pit

Year	Volume Placed (cu. yds.)	Volume Estimates from Fly-Overs	Approximate Elevation Range at End of Year (feet)	No. of Soil Technician Visits	Total No. of Field Density Tests	Average Inspection Frequency (1 visit per indicated cu. yds.)	Average Density Test Frequency (1 test per indicated cu. yds.)	Remarks
1991	969,186 [1]		271 - 280	35	224	27,691	4,327	Volume estimate extrapolated from monthly values from June through December.
1992	936,463 [1]		258 - 291	53	353	17,669	2,653	Volume estimate extrapolated from monthly values from January through September.
1993	455,103		273 - 302	47	171	9,683	2,661	
1994	423,698 [1]		272 - 313	25	152	16,948	2,787	Volume estimate extrapolated from monthly values from January through June.
1995	383,582		272 - 306	16	66	23,974	5,812	
1996	405,624		294 - 312	16	43	25,352	9,433	
1997	474,021		297 - 315	11	49	43,093	9,674	
1998	494,909		312 - 318	9	36	54,990	13,747	
1999	701,599		286 - 320	4	35	175,400	20,046	
2000	786,123	625,132 [2]	330 - 345 [3]	1	5	786,123	157,225	
2001	782,788	720,480 [2]	342 - 356 [3]	5	12	156,558	65,232	
2002	580,909	606,962 [2]	353 - 366 [3]	6	6	96,818	96,818	
2003	520,451	589,348 [2]	358 - 377 [3]	10	18	52,045	28,914	
2004	877,326	788,151 [2]	368 - 388 [3]	8	15	109,666	58,488	
2005	1,083,006	1,023,767 [2]	370 - 400 [3]	45	44	24,067	24,614	
Inspections and testing by Hushmand Associates, Inc.								
2006	1,182,290 [2]	876,223 [2]	378 - 409 [3]	265	844	4,461	1,401	

Notes:

[1]Volumes placed were obtained from available reports; volumes extrapolated from available monthly volumes for indicated years.

[2]Volume estimates from fly-overs; interpolated to get yearly volumes.

[3]Approximate end of year elevations interpolated between fly-over dates.

Table 2-2
Summary of ZK Progress Reports
Nu-Way Live Oak Pit

ZK Progress Report #	Date	Period Covered	Frequency of Field Visits (as stated in ZK report)	Actual Frequency of Field Visits (based on ZK daily reports)*
1	September 5, 1991	January-August 1991	Typically 1 per week	✓
2	December 9, 1991	September-November 1991	Typically 1 per week	✓
3	April 10, 1992	December 1991-March 1992	Typically 2 per week except February and March, when rate too small during rain	✓
4	August 24, 1992	April-June 1992	1 per week to 2 per week	✓
5	December 2, 1992	July-October 1992	1 per week to 2 per week	On average, 1 per week
6	May 4, 1993	November 1992-January 1993	1 per week to 2 per week	On average, 1 per week
7	August 9, 1993	February-April 1993	1 per week to 2 per week	✓
8	August 17, 1993	May-July 1993	1 per week to 2 per week	✓
9	December 1, 1993	August-October 1993	1 per week to 1 per 2 weeks	On average, 1 per week
10	March 25, 1994	November 1993-February 1994	1 per week to 1 per 2 weeks	On average, 1 per week (only 1 visit in December 1993)
11	September 7, 1994	March-August 1994	1 per 2 weeks to 1 per month	✓
12	August 16, 1995	September 1994-July 1995	1 per 2 weeks to 1 per month	On average, 1 per month
13	December 30, 1997	Not stated, but should be interpreted as August 1995-December 1997, i.e, 2½ years	1 per week to 1 per month	On average, 1 per month
14	February 12, 1999	Not stated, but should be interpreted as Year 1998 (entire year)	1 per 2 weeks to 1 per month	1 per month to 1 every 2 months
Not listed	January 14, 2002	Not stated, but should be interpreted as 3 years - Year 1999 through Year 2001	1 per 2 weeks to 1 per month	1 per 3 months to 1 per year (1 visit in 2000)
Not listed	January 29, 2003	Not stated, but assumed to be for Year 2002	1 per month to 1 per 2 months	On average, 1 every 2 months (no visits for 3 consecutive months, Oct.-Dec.)
Not listed	July 31, 2003	Not stated, but assumed to be for 6 months (January-June/July 2003)	1 per month to 1 per 2 months	✓
Not listed	August 31, 2004	August 1, 2003-August 31, 2004	1 per month to 1 per 2 months	✓
Not listed	June 12, 2006	August 31, 2004-June 12, 2006	1 to 3 per month to 1 every 2 to 3 months, based on the rate of material import	✓

*✓ means actual frequency matches with that stated in progress report.

P:\10 Projects\10-101 (Nu-Way Pit)\Technical Memorandum\Tables\Table 2-2 Summary of ZK Progress Reports.xlsx

0916.0172

Table 3-1
Open Voids Evaluation
Nu-Way Live Oak Pit

Test Pit No.	Bench No.	Bulk (Ring) Density Tests			Sand Cone Tests			Open Voids	
		Wet Density (pcf)	Dry Density (pcf)	Void Ratio, e_t -	Wet Density (pcf)	Dry Density (pcf)	Void Ratio, e_s -	Porosity -	Void Ratio, e_o -
TP1	Bench 1	79.7	76.6	-	119.4	116.1	-	-	-
	Bench 2	133.2	125.9	-	105.9	96.7	-	-	-
	Bench 3	114.9	109.6	-	112.3	102.4	-	-	-
	Bench 4	91.0	86.1	-	98.9	90.6	-	-	-
TP2	Bench 1	127.8	121.1	-	116.4	107.5	-	-	-
	Bench 2	133.6	126.4	-	107.2	98.1	-	-	-
	Bench 3	129.2	120.2	-	109.9	97.4	-	-	-
	Bench 4	126.5	117.3	-	124.8	107.5	-	-	-
Average		**117.0**	**110.4**	**0.433**	**111.9**	**102.0**	**0.621**	**0.067**	**0.096**

<u>**Notes:**</u>

1. Large-scale insitu bulk density (ring density) and sand cone density values are based on measurements made by Irvine Geotechnical, Inc. (2008b)
2. Assumed specific gravity of infill material (minus ¾" material) = 2.65
   Assumed specific gravity of concrete fragments (plus ¾" material) = 2.40
3. Average percent by weight of +¾" material = 43 %
   Average percent by weight of -¾" material = 57 %
4. e_t = Void ratio of total material
   e_s = Void ratio of infill soils (minus ¾-inch material)
   e_o = Ratio of open voids
   [See Figure 3-2 for definition of void ratio]

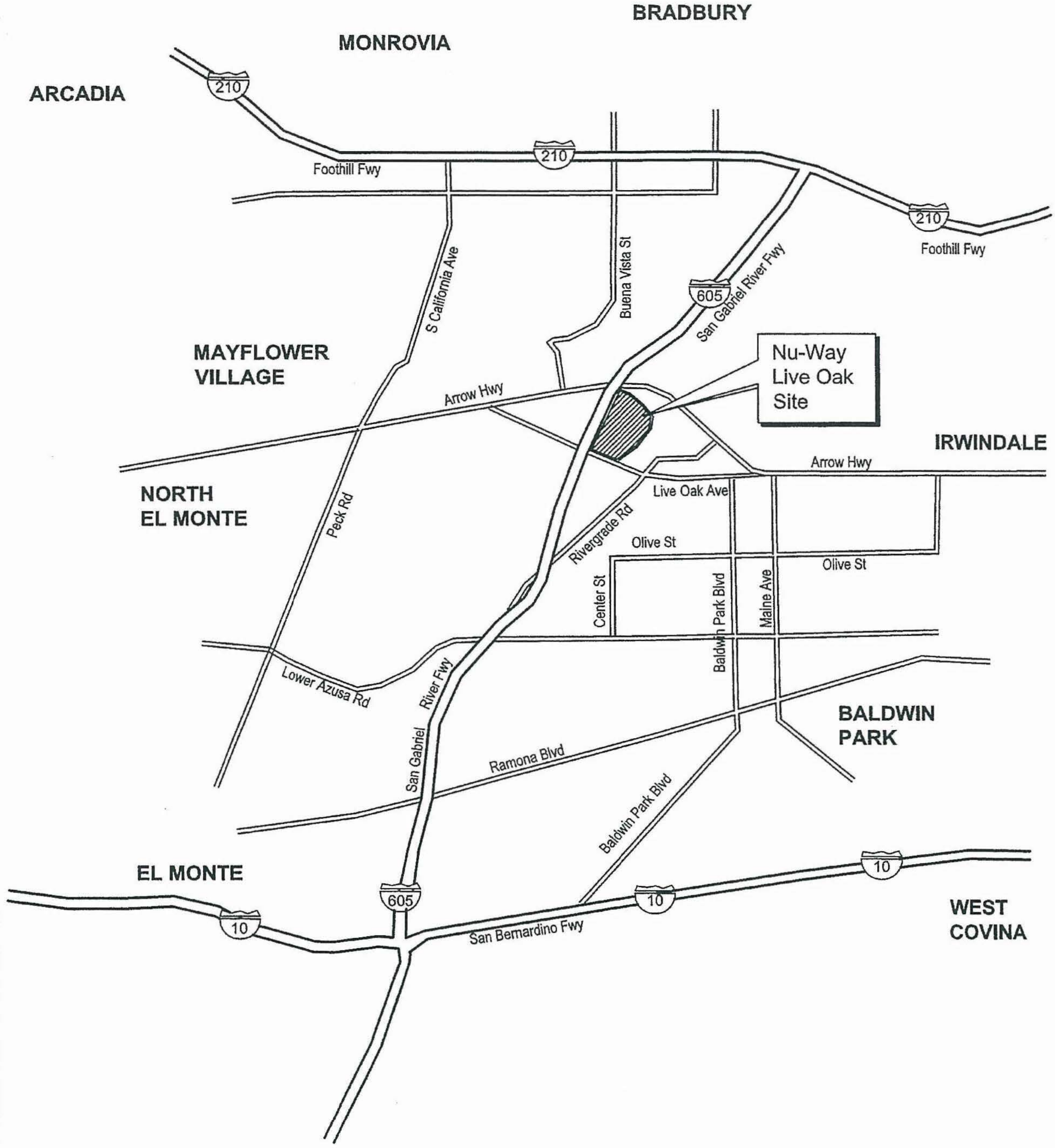
Table 3-2
Estimated Seismic Settlements for Existing Conditions
Nu-Way Live Oak Pit

Description	Parameters	Becker Hammer Boring Locations [1]					
		B1	B2	B3	B4	B5	B6
Densification of Thin Sand Layers	Cumulative Thickness [2] (ft)	5	5	20	0	17	38
	Settlement (in)	0.2	0.2	0.7	0	0.8	2.3
Densification of Rubble Fill Layers	Cumulative Thickness (ft)	105	105	90	110	93	72
	Settlement (in)	0.5	0.5	0.5	0.4	0.5	0.3
Filling of Open Voids due to Migration of Overlying Sands and Collapse of Nested Structure	Open Voids (%) (Porosity or % of Depth)	6.7	6.7	6.7	6.7	6.7	6.7
	Cumulative Thickness of Layered Profile (ft)	70	73	82	81	52	68
	Percent Open Voids Filled (Assumed)	20	20	20	8 [4]	20	20
	Settlement (in)	11.3	11.7	13.2	5.2	8.4	10.9
Estimated Total Seismic Settlement [3]		**11.9**	**12.4**	**14.4**	**5.6**	**9.6**	**13.6**

Notes:

1. IGI (2008a) Report
2. Interpreted from low blow count points [$(N_1)_{60} < 15$] representing sand layers
3. Estimated settlement based on the assumption that 20% of the open voids in the rubble layers get filled due to sand migration and collapse
4. Sand layers at B4 appear to be significantly denser, based on blow counts. Therefore, the percentage of open voids that get filled was assumed to be only 0.4 times that for the other borings

Figures



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Project No.: 10-101

Settlement Analysis
Nu-way Live Oak Pit
Hunt Ortmann

Site Location Map

09-10

Figure 1-1

0916.0177

JUL 0

S	M	T	W	T	F	S
	1	2	3	4	5	6
7	8	9	10	11	12	13
14	15	16	17	18	19	20
21	22	23	24	25	26	27
28	29	30	31			

624

49,152

AUG 0

S	M	T	W	T	F	S
				1	2	3
4	5	6	7	8	9	10
11	12	13	14	15	16	17
18	19	20	21	22	23	24
25	26	27	28	29	30	31

43,461

MAY 0

S	M	T	W	T	F	S
				1	2	3
4	5	6	7	8	9	10
11	12	13	14	15	16	17
18	19	20	21	22	23	24
25	26	27	28	29	30	31

34,430

S	M	T
1	2	3
8	9	10
15	16	17
22	23	24
29	30	

NOV 0

S	M	T	W	T	F	S
					1	2
3	4	5	6	7	8	9
10	11	12	13	14	15	16
17	18	19	20	21	22	23
24	25	26	7'	28	29	30

* Lift thickness from G.A. Nicoll's Daily

36,267

DEC 4

S	M	T	W	T	F	S
1	2	3	4	5	6	7
8	9	10	11	12	13	14
15	16	17	18	19	20	21
22	23	24	25	26	27	28
29	30	31				

33,935

SEP 8

S	M	T	W	T	F	S
	1	2	3	4	5	6
7	8	9	10	11	12	13
14	15	16	17	18	19	20
21	22	23	24	25	26	27
28	29	30				

49,012

S	M	T
5	6	7
12	13	14
19	20	21
26	27	28

OO TESTS 5

MAR 5

S	M	T	W	T	F	S
			1	2	3	4
5	6	7	8	9	10	11
12	13	14	15	16	17	18
19	20	21	22	23	24	25

APR 0

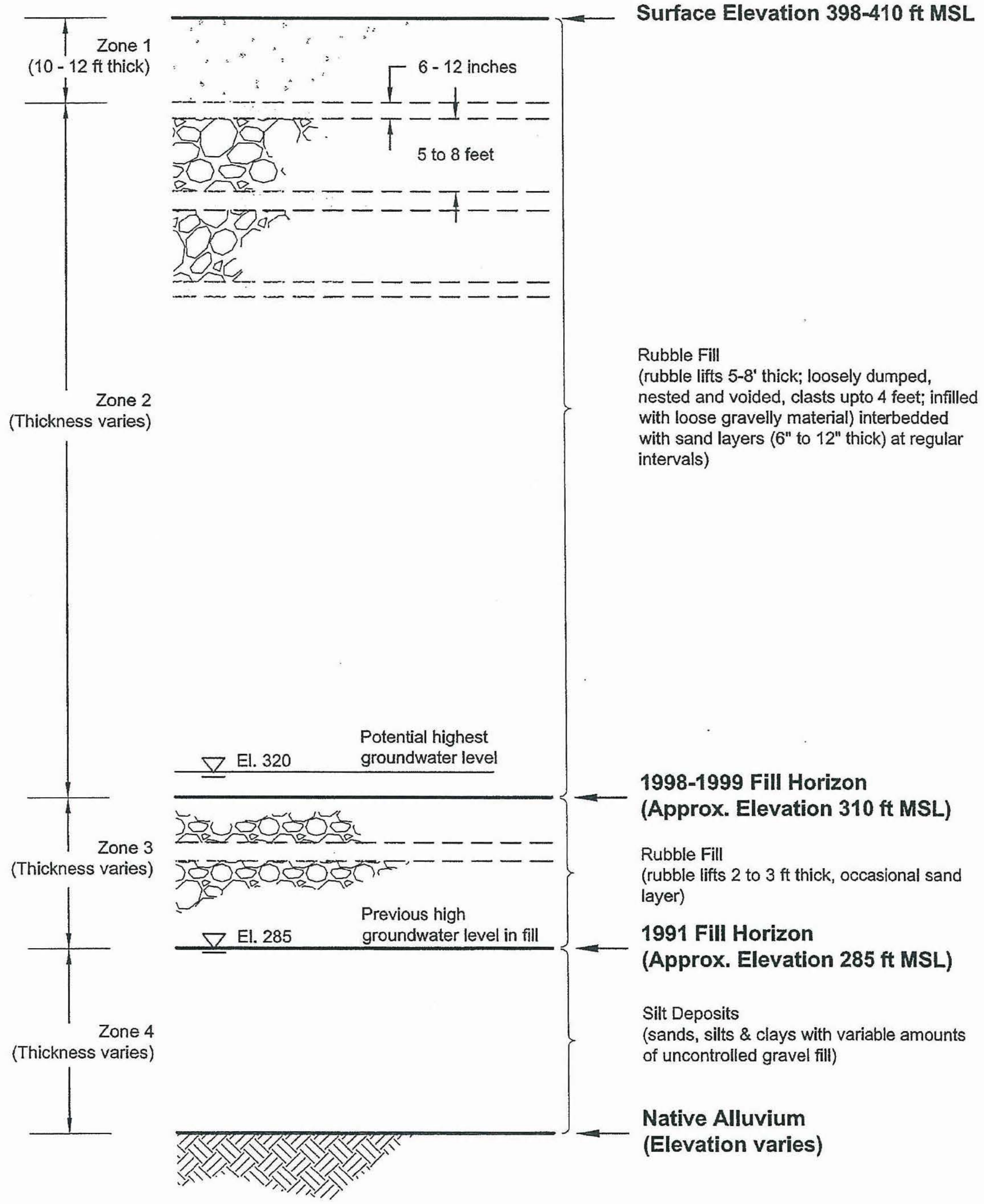
S	M	T	W	T	F	S
						1
2	3	4	5	6	7	8
9	10	11	12	13	14	15
16	17	18	19	20	21	22

VISITS 5

JAN 0

S	M	T	W	T	F	S
	1	2	3	4	5	6
7	8	9	10	11	12	13
14	15	16	17	18	19	20
21	22	23	24	25	26	27

S	M	T
4	5	6
11	12	13
18	19	20



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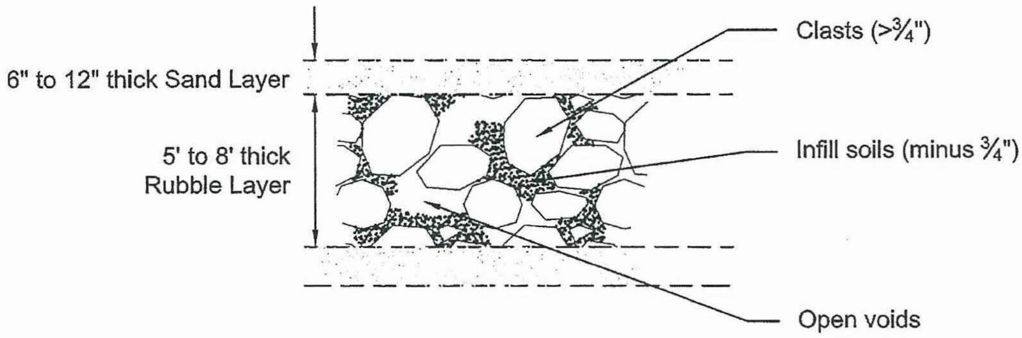
Settlement Analysis
Nu-way Live Oak Pit
Hunt Ortman

Idealized Soil Profile

09-10

Figure 3-1



V_c	$-Volume\ of\ Clasts$
W_c	$-Weight\ of\ Clasts\ (concrete)$
V_s	$-Volume\ of\ Infill\ Soils$
W_s	$-Weight\ of\ Infill\ Soils$
$(V_v)_s$	$-Volume\ of\ Voids\ In\ Soil$
$(V_v)_o$	$-Volume\ of\ Open\ Voids$
G_s	$-Specific\ Gravity\ of\ Infill\ Soils$
G_c	$-Specific\ Gravity\ of\ Concrete\ Clasts$

$$Void\ ratio\ of\ Infill\ soils \quad = e_s = \frac{(V_v)_s}{V_s}$$

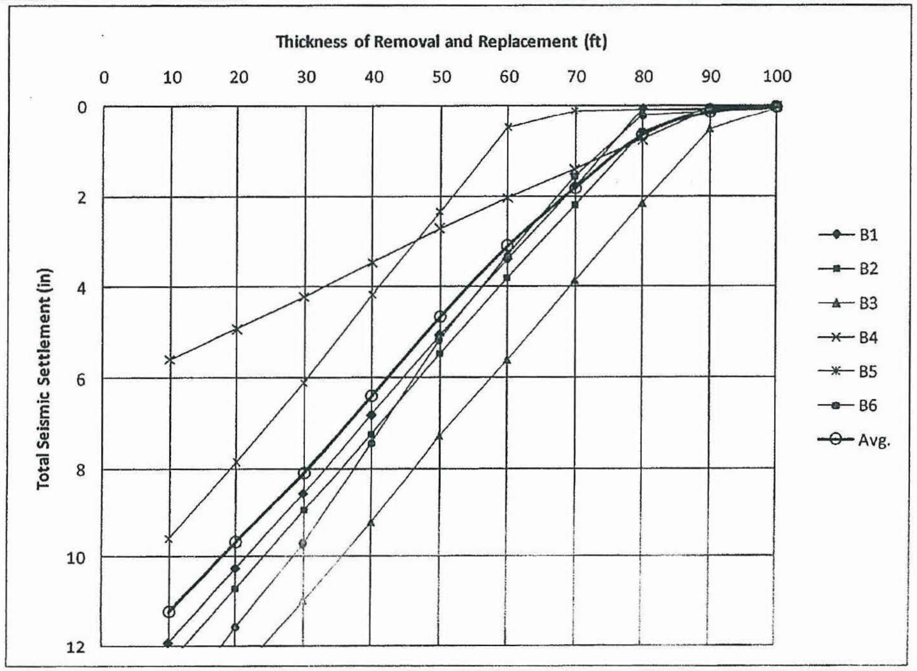
$$Total\ void\ ratio \quad = e_t = \frac{(V_v)_s + (V_v)_o}{V_s + V_c}$$

$$Ratio\ of\ open\ voids \quad = e_o = \frac{(V_v)_o}{V_s + V_c} = e_t - \frac{e_s}{\left(1 + \frac{W_c G_s}{W_s G_c}\right)}$$

$$Porosity = \left(\frac{Volume\ of\ open\ voids}{Total\ volume\ of\ fill}\right) = \frac{(V_v)_o}{V_s + V_c + (V_v)_s + (V_v)_o}$$

AES Advanced Earth Sciences, Inc. Geotechnical and Environmental Consultants	Project No.: 10-101 Settlement Analysis Nu-way Live Oak Pit Hunt Ortman
Definition of Open Voids 09-10	Figure 3-2

Boring Number	B1				B2				B3				B4				B5				B6				
Open Voids, FV (% by Volume)	6.7				6.7				6.7				6.7				6.7				6.7				
Open Voids Filled by Soil Intrusion & Collapse (% of FV assumed)	20				20				20				8 (= 0.4 x 20)				20				20				
Depth Contributing to Open Voids (ft)	70				73				82				81				52				68				
	Estimated Seismic Settlement (in)																								
Cap Thickness (ft)	Sand Layer	Rubble	Filling of Open Voids	Total	Sand Layer	Rubble	Filling of Open Voids	Total	Sand Layer	Rubble	Filling of Open Voids	Total	Sand Layer	Rubble	Filling of Open Voids	Total	Sand Layer	Rubble	Filling of Open Voids	Total	Sand Layer	Rubble	Filling of Open Voids	Total	Average
10	0.2	0.5	11.3	11.9	0.2	0.5	11.7	12.4	0.7	0.5	13.2	14.4	0.0	0.4	5.2	5.6	0.8	0.5	8.4	9.6	2.3	0.3	10.9	13.6	**11.2**
20	0.2	0.5	9.6	10.3	0.2	0.4	10.1	10.7	0.7	0.5	11.6	12.7	0.0	0.3	4.6	4.9	0.7	0.4	6.8	7.9	2.0	0.3	9.3	11.6	**9.7**
30	0.2	0.4	8.0	8.6	0.1	0.4	8.5	9.0	0.7	0.4	10.0	11.0	0.0	0.3	3.9	4.2	0.6	0.3	5.1	6.1	1.8	0.2	7.7	9.7	**8.1**
40	0.1	0.3	6.4	6.8	0.1	0.3	6.9	7.2	0.7	0.2	8.4	9.2	0.0	0.2	3.3	3.5	0.4	0.2	3.5	4.2	1.2	0.2	6.1	7.4	**6.4**
50	0.1	0.1	4.8	5.1	0.1	0.1	5.3	5.5	0.4	0.1	6.8	7.3	0.0	0.1	2.6	2.7	0.3	0.1	1.9	2.3	0.6	0.1	4.5	5.1	**4.7**
60	0.1	0.1	3.2	3.4	0.1	0.1	3.7	3.8	0.4	0.1	5.1	5.6	0.0	0.1	2.0	2.1	0.1	0.1	0.3	0.5	0.4	0.1	2.9	3.3	**3.1**
70	0.1	0.1	1.6	1.8	0.1	0.1	2.1	2.2	0.3	0.1	3.5	3.9	0.0	0.0	1.4	1.4	0.0	0.1	0.0	0.1	0.2	0.1	1.3	1.6	**1.8**
80	0.0	0.1	0.0	0.1	0.1	0.0	0.5	0.6	0.2	0.0	1.9	2.2	0.0	0.0	0.7	0.7	0.0	0.1	0.0	0.1	0.2	0.0	0.0	0.2	**0.6**
90	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.2	0.0	0.3	0.5	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.1	0.1	0.0	0.0	0.1	**0.2**
100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.1	**0.0**



Note: Settlement Estimate Assuming 20% of Open Voids are Filled (p=20%)

AES Advanced Earth Sciences, Inc. Geotechnical and Environmental Consultants

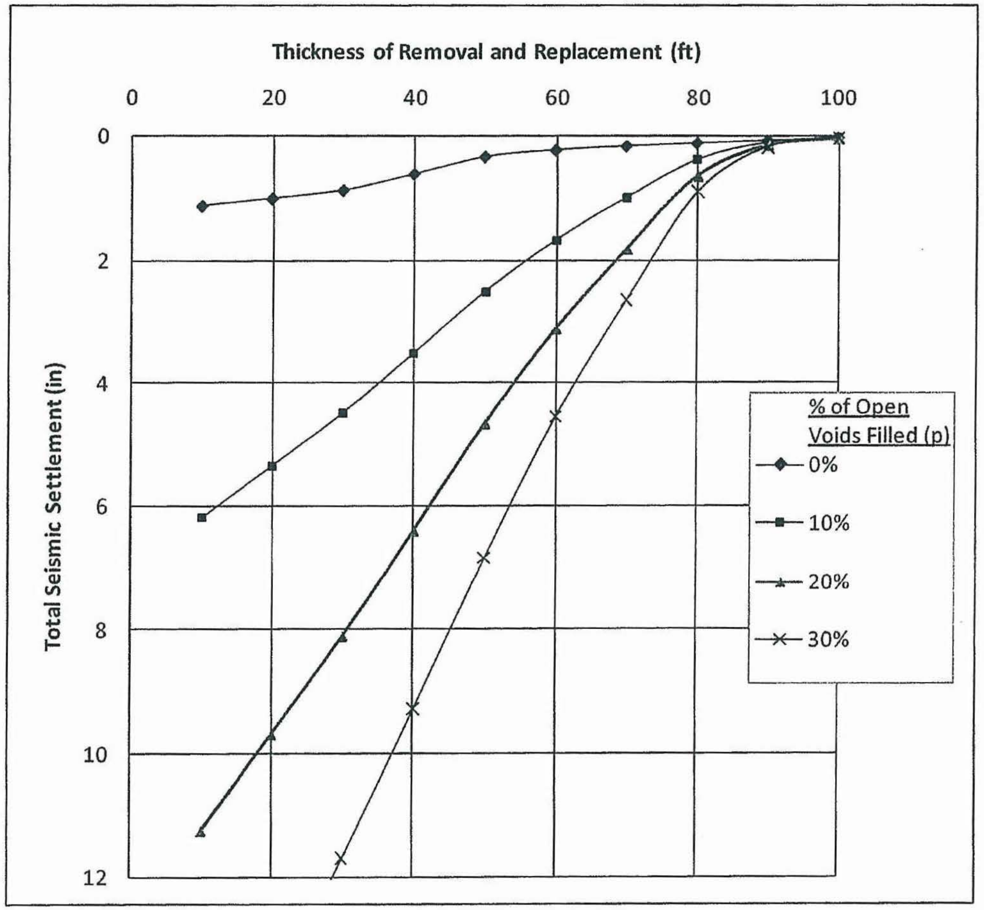
Project No.: 10-101

Settlement Analysis
Nu-Way Live Oak Pit
Hunt Ortmann

Seismic Settlement versus Thickness of Removal and Replacement at Each Borehole Location

09-10 Figure 3-3

0916.0180



Cap Thickness (ft)	Average Total Seismic Settlement (inches) for Different % of Open Voids Filled (p)			
	0%	10%	20%	30%
10	1.1	6.2	11.2	16.3
20	1.0	5.3	9.7	14.0
30	0.9	4.5	8.1	11.7
40	0.6	3.5	6.4	9.3
50	0.3	2.5	4.7	6.8
60	0.2	1.7	3.1	4.6
70	0.2	1.0	1.8	2.6
80	0.1	0.4	0.6	0.9
90	0.1	0.1	0.2	0.2
100	0.0	0.0	0.0	0.0

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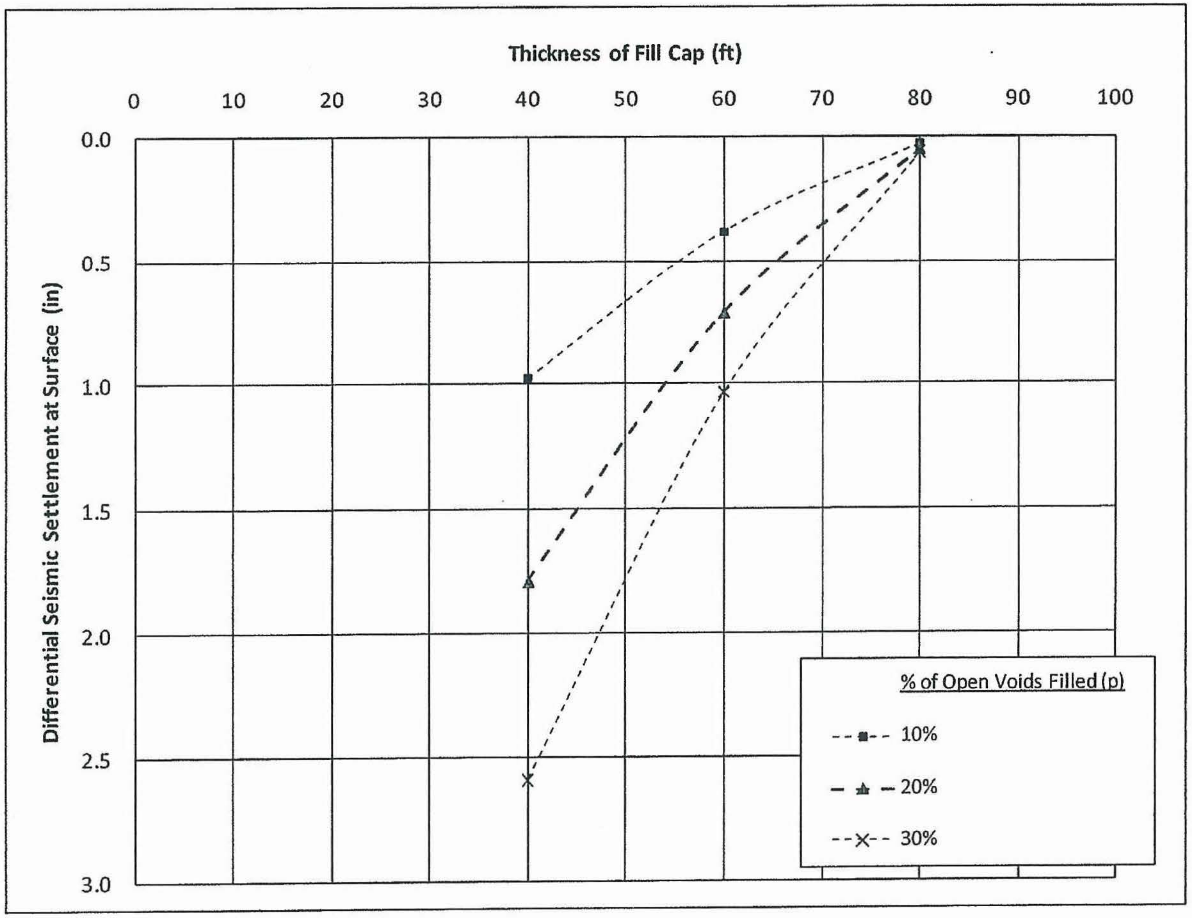
Project No.: 10-101

Settlement Analysis
Nu-Way Live Oak Pit
Hunt Ortmann

Average Seismic Settlement for Various Percentages of Open Voids Filled

09-10

Figure 3-4



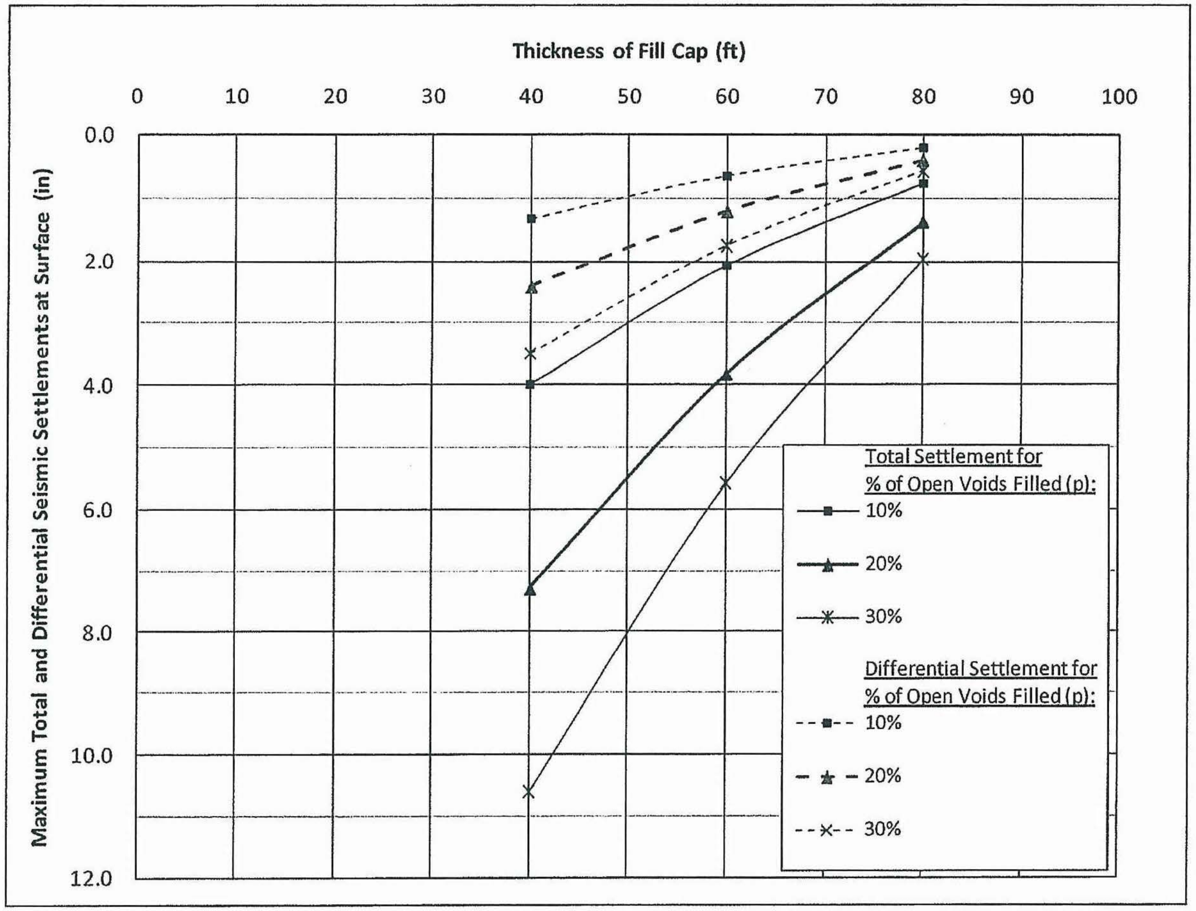
Note:
Differential settlements are estimated over a horizontal distance of 30 ft

Advanced Earth Sciences, Inc. Geotechnical and Environmental Consultants	Project No.: 10-101 Settlement Analysis Nu-Way Live Oak Pit Hunt Ortmann

Surface Manifestation of Differential Settlement - Isolated Differential Settlement

09-10 Figure 3-5

0916.0182



Note:
Differential settlements are estimated over a horizontal distance of 30 ft

Advanced Earth Sciences, Inc.
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Project No.: 10-101
Settlement Analysis
Nu-Way Live Oak Pit
Hunt Ortmann

Surface Manifestation of Total and Differential Settlement – Randomly Varying Settlements

09-10

Figure 3-6

0916.0183

Appendix A
Test Trench Photographs (from GLA and IGI)

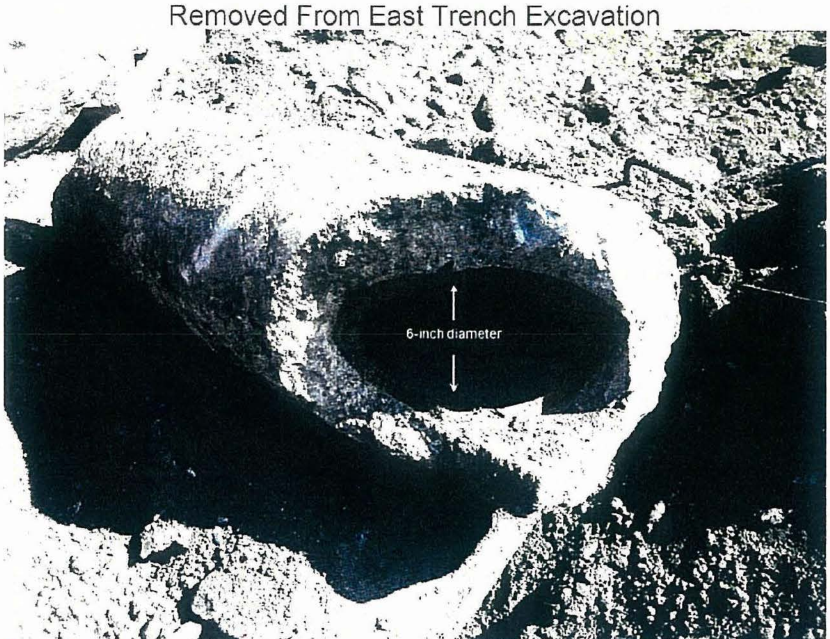
Photographs from GLA

Removed From East Trench Excavation

Geo-Logic ASSOCIATES

IGT08576

Removed From East Trench Excavation



Geo-Logic ASSOCIATES

IGT08577

Material Removed From East Trench Excavation

Geo-Logic ASSOCIATES

IGT08578

Removed From East Trench Excavation

Geo-Logic ASSOCIATES

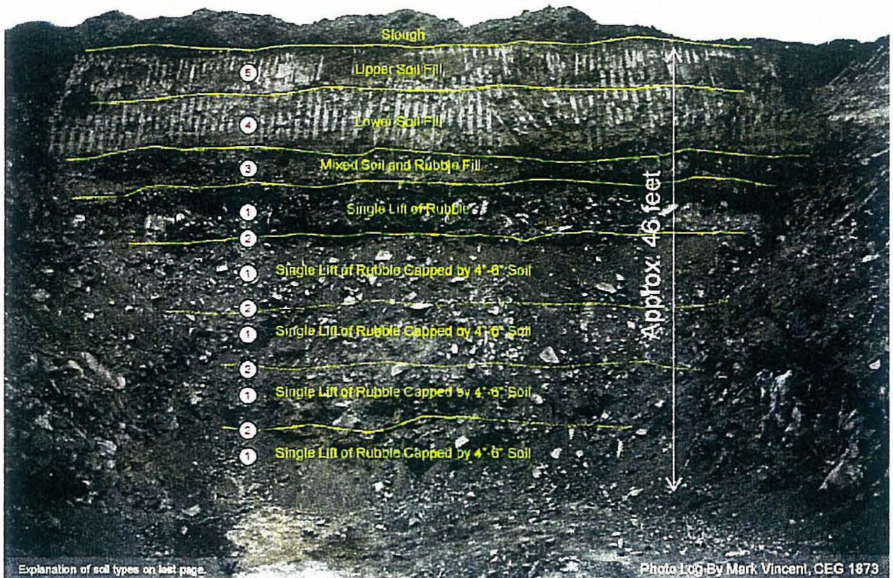
IGT08579

Removed From West Trench Excavation

Geo-Logic ASSOCIATES

IGT08580

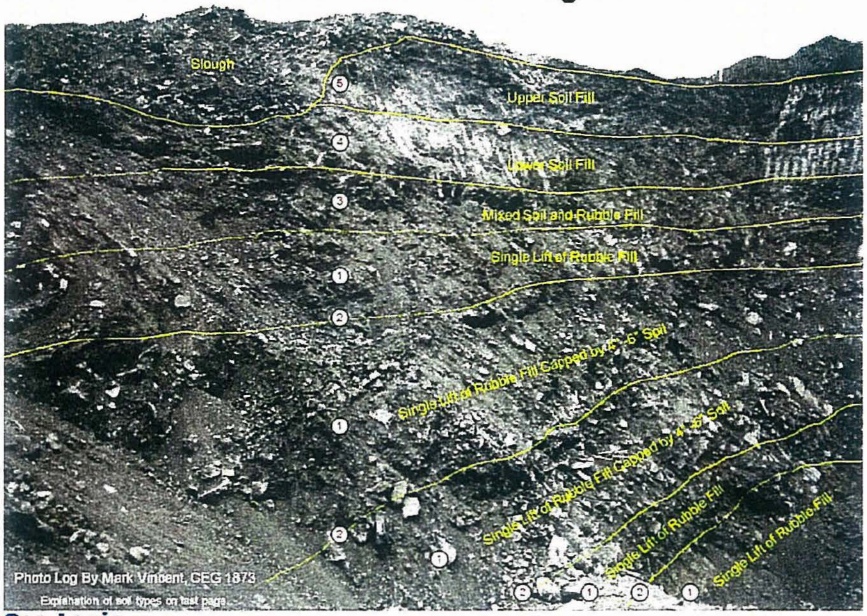
West Trench Facing Northeast



Geo-Logic ASSOCIATES

IGT08581

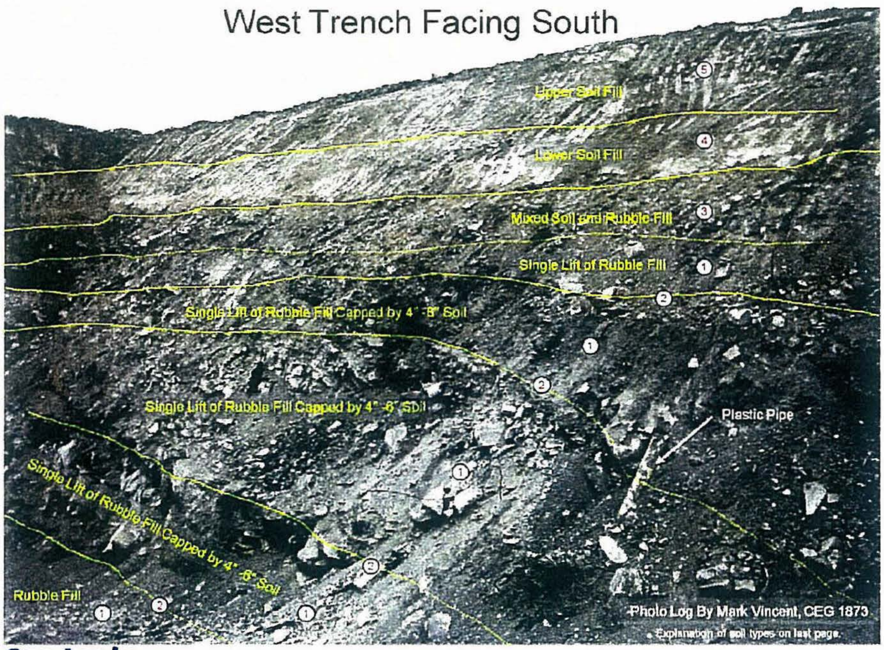
West Trench Facing North



Geo-Logic ASSOCIATES

IGT08582

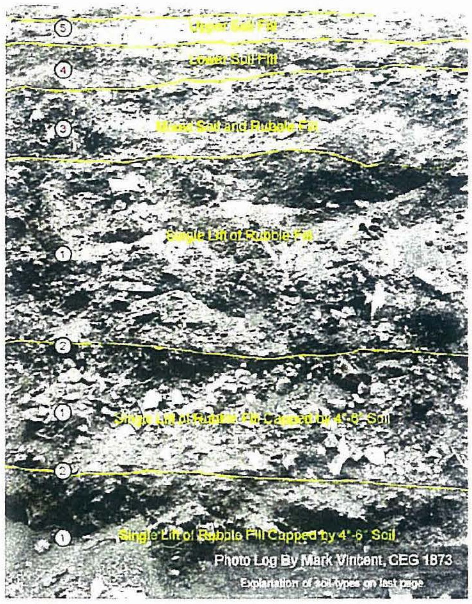
West Trench Facing South



Geo-Logic ASSOCIATES

IGT08583

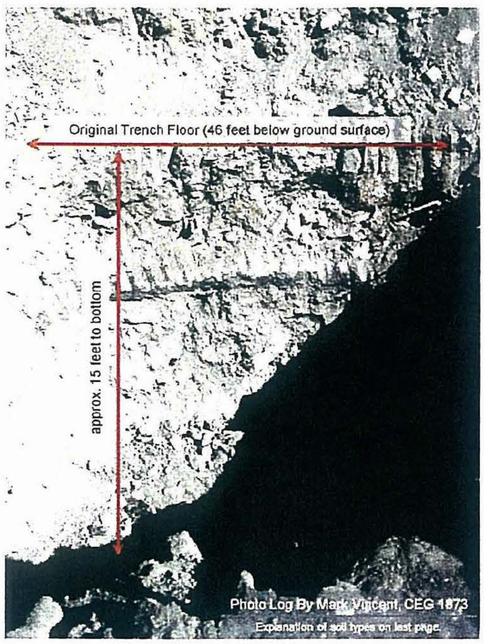
West Trench Facing South - Detail



Geo-Logic ASSOCIATES

IGT08584

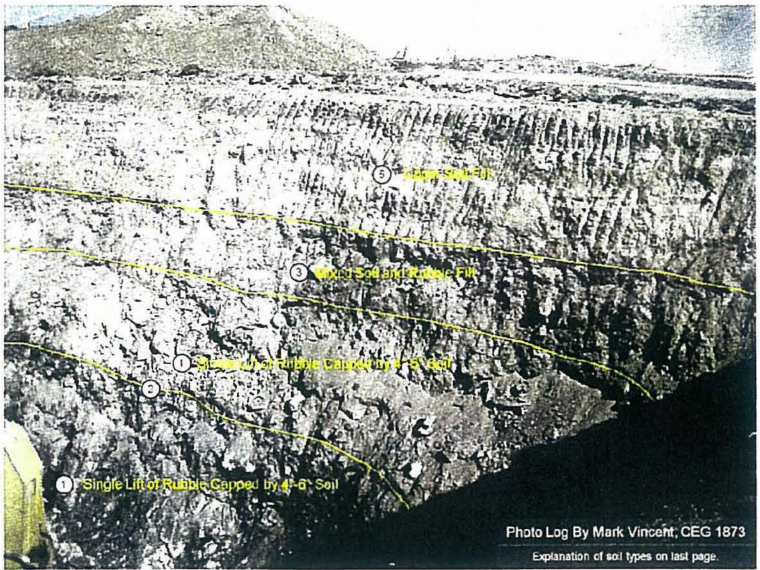
West Trench – Detail of Deepened Excavation



Geo-Logic ASSOCIATES

IGT08585

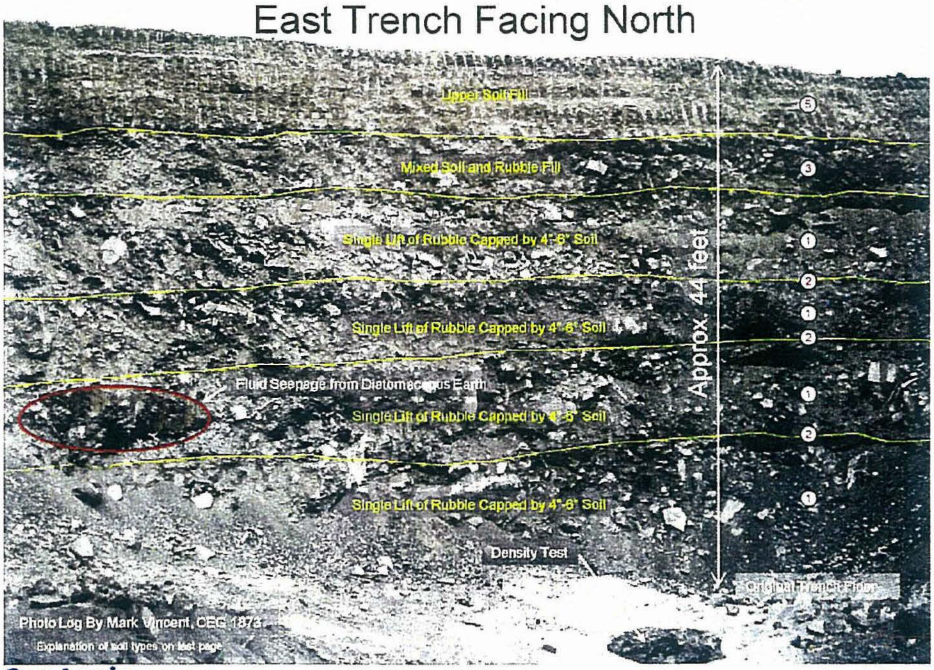
East Trench Facing North - View From Above During Excavation



Geo-Logic ASSOCIATES

IGT08586

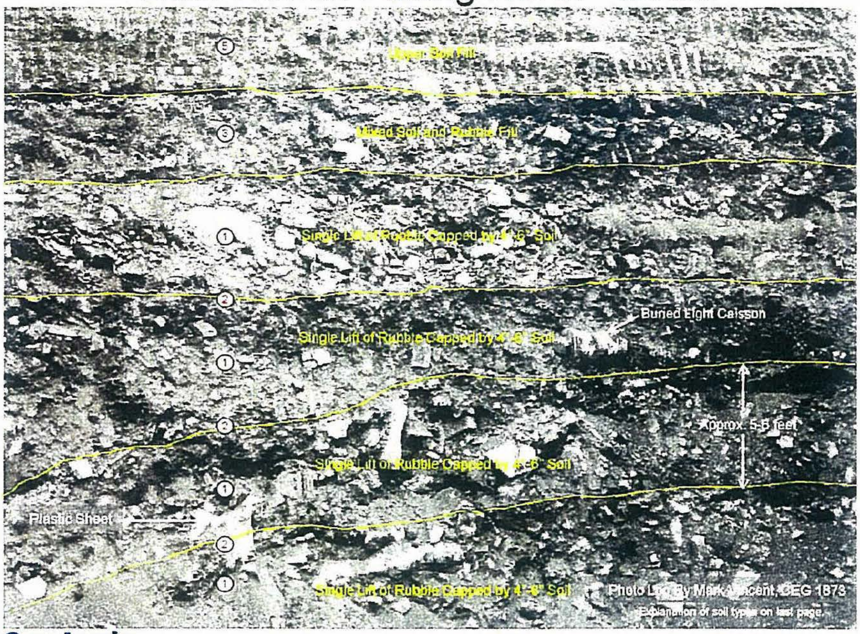
East Trench Facing North



Geo-Logic ASSOCIATES

IGT08587

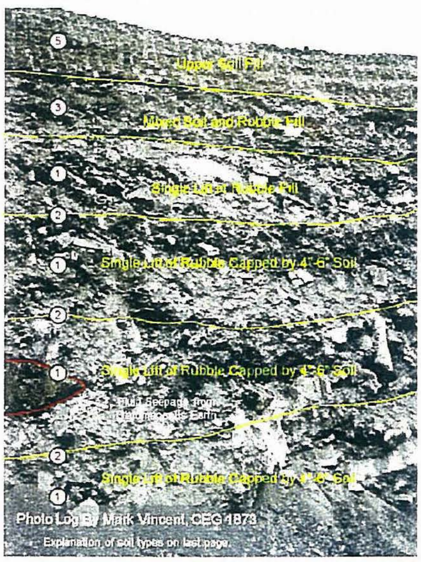
East Trench Facing North - Detail



Geo-Logic ASSOCIATES

IGT08588

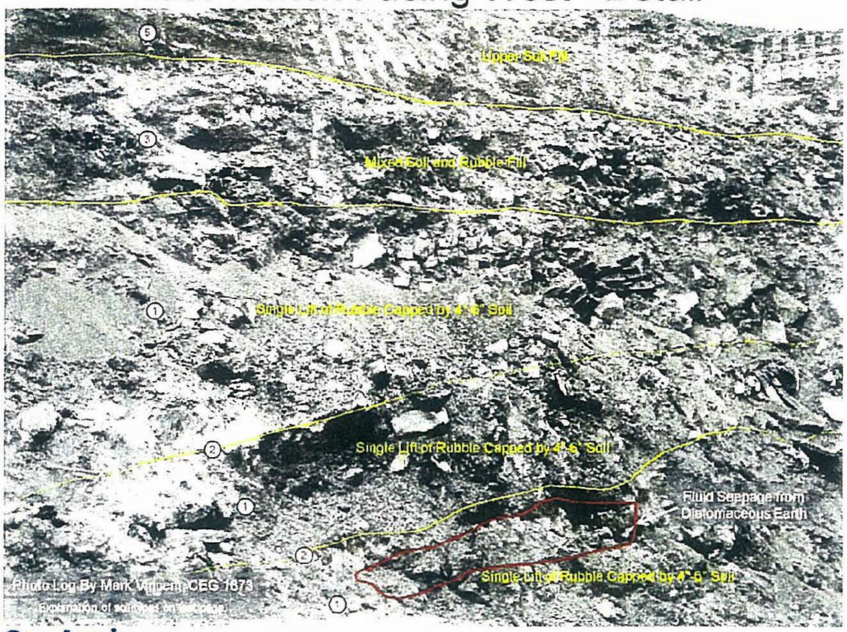
East Trench Facing North - Detail



Geo-Logic ASSOCIATES

IGT08589

East Trench Facing West - Detail



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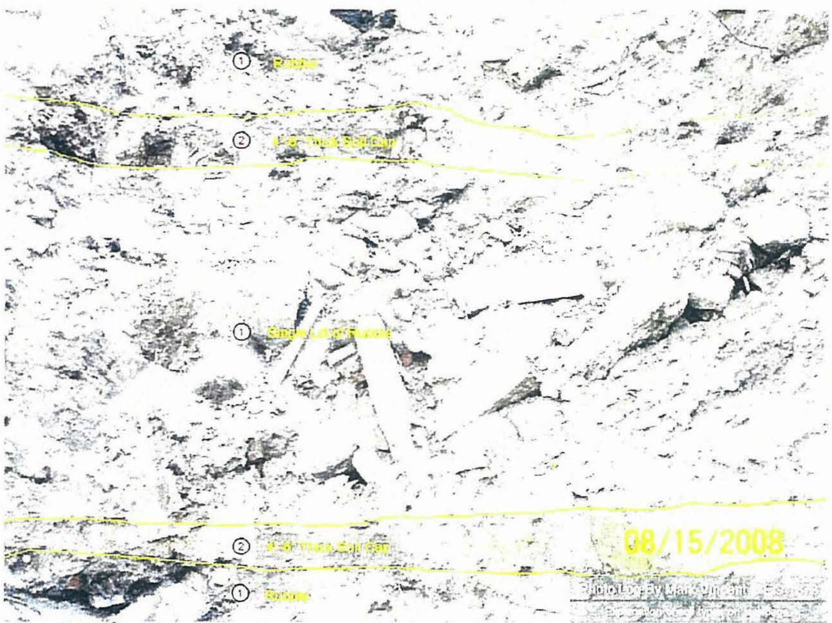
East Trench – Northeast Corner Detail Near Bottom



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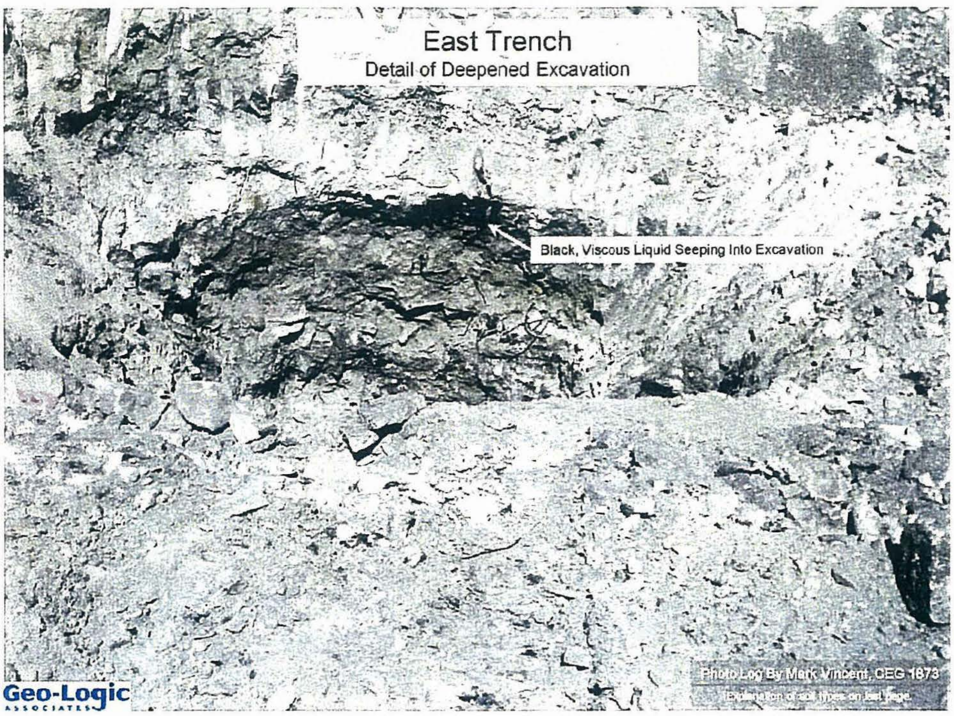
East Trench

Concrete Footing at Depth of Approximately 20 Feet Below Ground Surface

Geo-Logic ASSOCIATES

IGT08592



East Trench

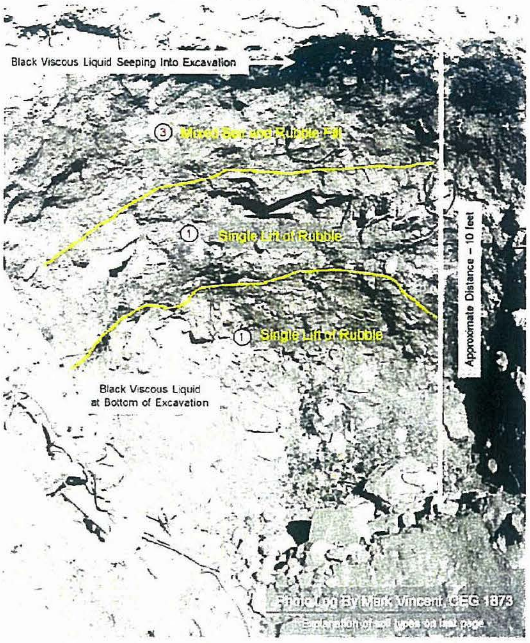
Detail of Deepened Excavation

Geo-Logic ASSOCIATES

IGT08593

East Trench

Detail of Deepened Excavation Bottom

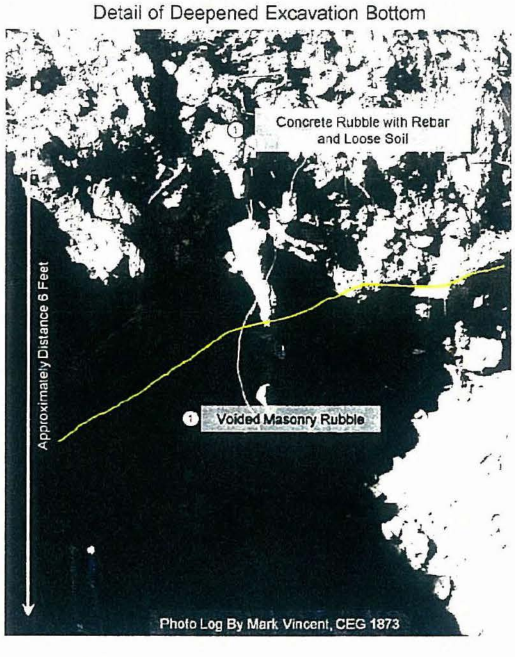


Geo-Logic ASSOCIATES

IGT08594

East Trench

Detail of Deepened Excavation Bottom



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IGT08595

Description of Fill Types From Previous Pages

- 1 – Rubble Fill - consisting primarily of concrete with abundant protruding rebar, floor tile, cement and asphalt shingles, bricks, soil, crushed glass, and diatomaceous earth filter packing. Many blocks of concrete have extreme aspect ratios (ratio of the length to width) with some maximum dimensions in excess of four feet. Most of the blocks are nested with some loose, dry, dark gray, organic rich, clayey soil backfill around the blocks. Extensive rubble zones are voided and have no soil fill. These rubble fill lifts or layers appear to be formed by end dumping of debris from trucks into layers four to five feet thick. The lifts of layers were then capped by a four to six inch thick layer of silty sand to sandy silt. Areas of seepage observed in the excavations consist of what appears to be diatomaceous earth filter packing. The material was most likely placed in the fill while saturated and have remained wet due to a capillary break resulting from being surrounded by concrete blocks with little or no soil backfill between blocks.

- 2 – Soil - Tan to brown silty sand to sandy silt with rare dark gray sandy clay containing only minor amounts of fine to medium gravel. The layers are found sandwiched between single, thick lifts of rubble fill.

- 3 – Soil with Rubble - Tan to brown silty sand to sandy silt with abundant coarse gravel to boulder size clasts of concrete and asphalt. This layer is typically approximately one to two feet thick.

- 4 – Lower Soil Fill – Tan to gray sandy silt to sandy clay contains minor amounts of fine to medium gravel composed mostly of rock with minor amounts of concrete. Well compacted and slightly moist during excavation.
-
- 5 - Upper Soil Fill – Tan silty sand to sandy silt contains minor amounts of fine to medium gravel composed mostly of rock. Well compacted and slightly moist to dry during excavation.

Mark Vincent, CEG 1873

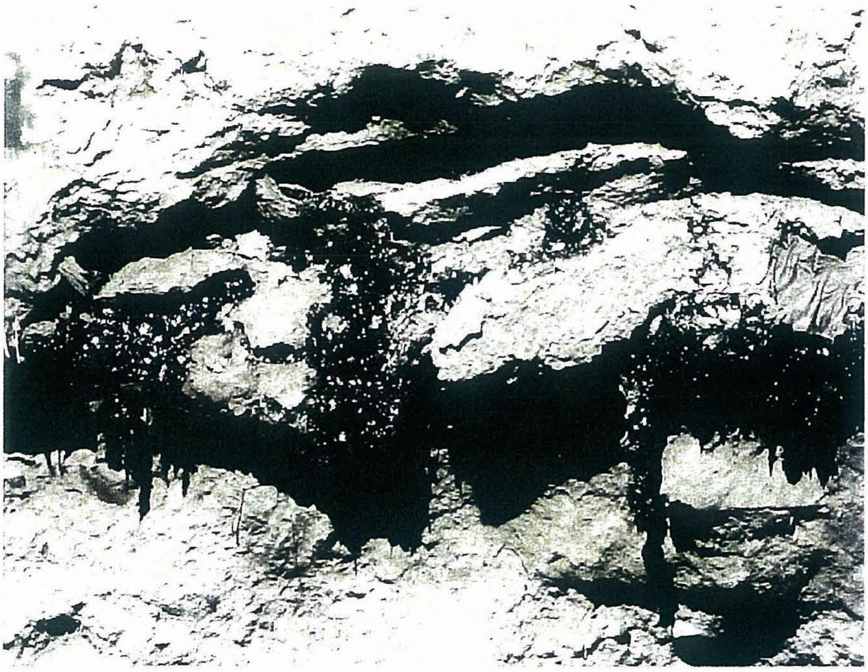
Geo-Logic Associates

IGT08596

Photographs from IGI

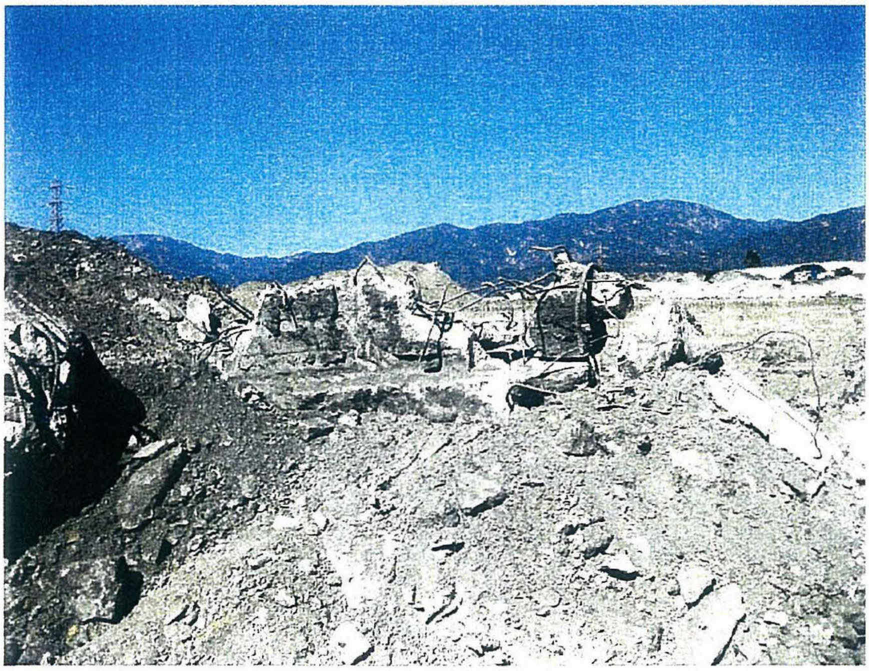
Test Pit Walls

Test Pit Walls



Test Pit Walls

Excavated Debris



Excavated Debris

Excavated Debris

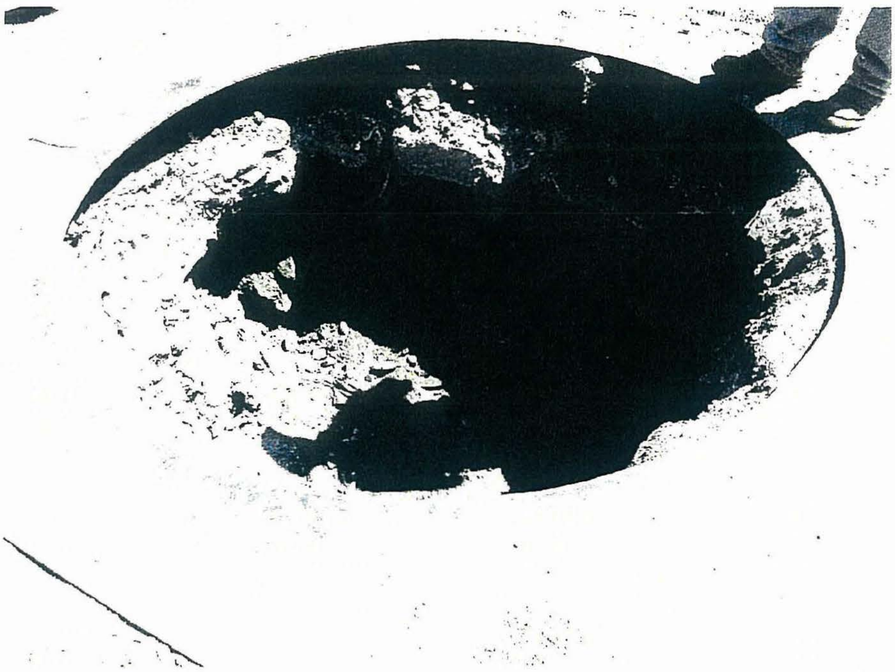
Voids

Voids

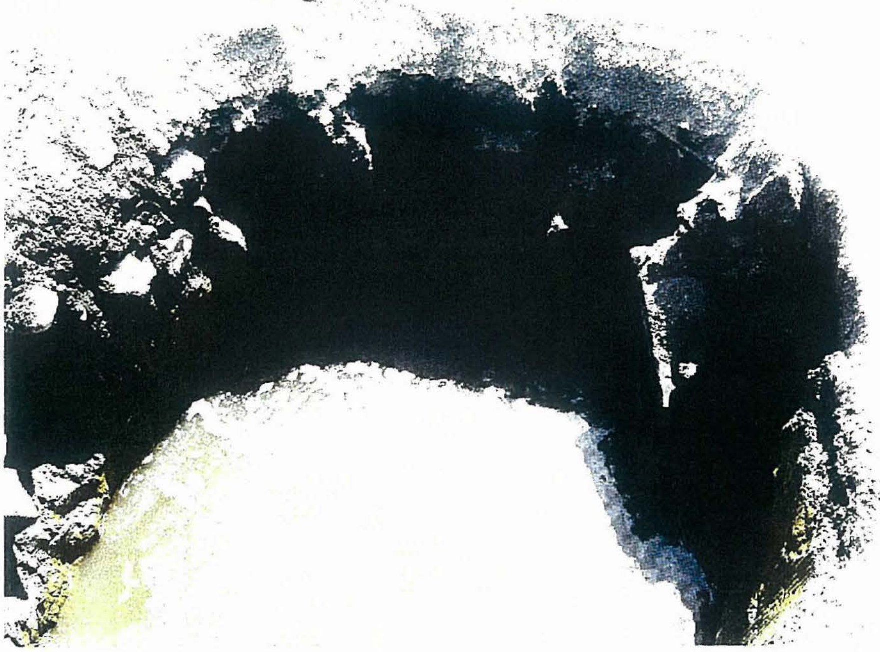
Voids

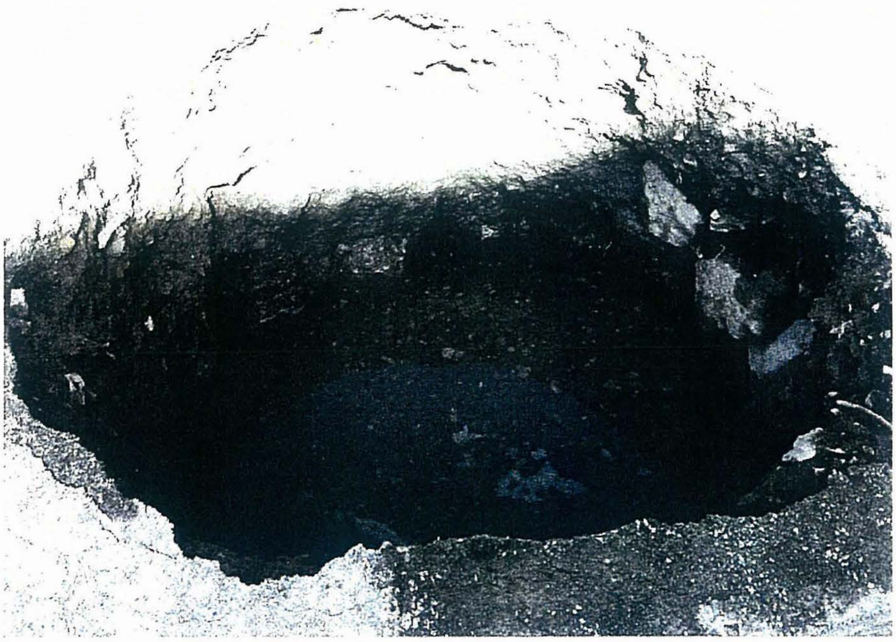
Voids

Voids



Ring Density Pits in Rubble Fill

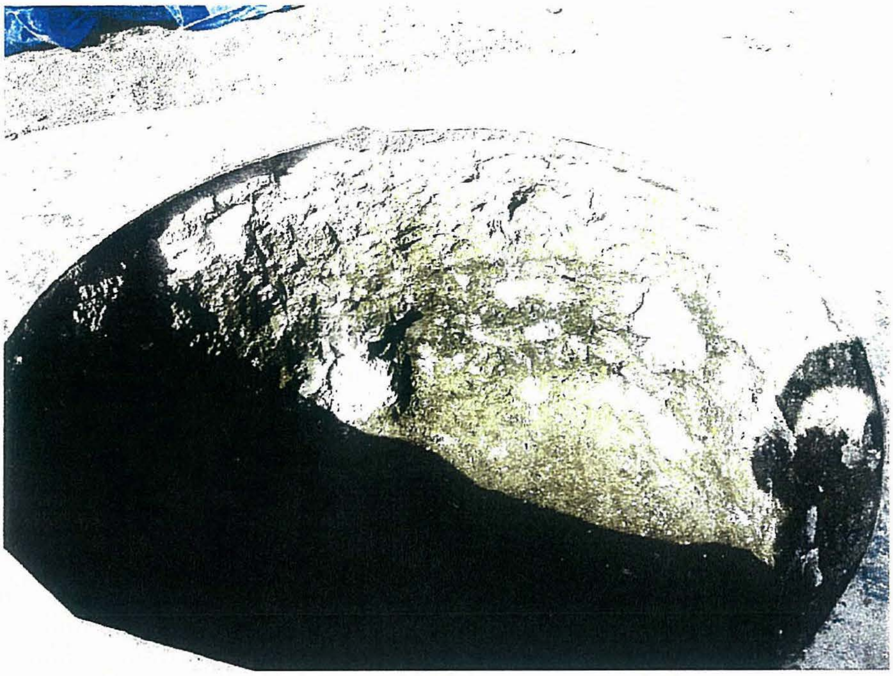




Ring Density Pits in Rubble Fill

Ring Density Pits in Rubble Fill

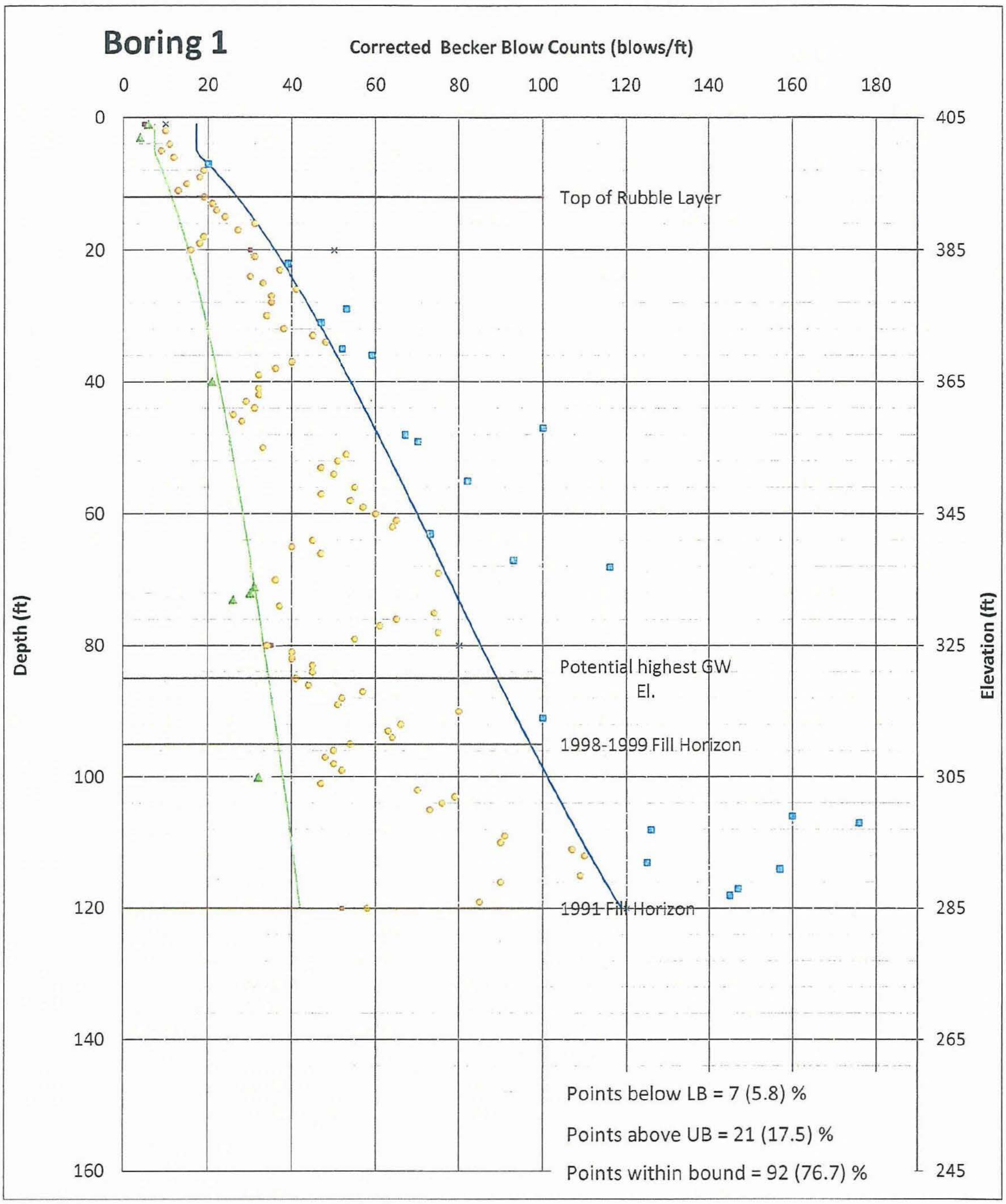
Ring Density Pits in Properly Compacted Debris Fill



Ring Density Pits in Properly Compacted Debris Fill

Appendix B

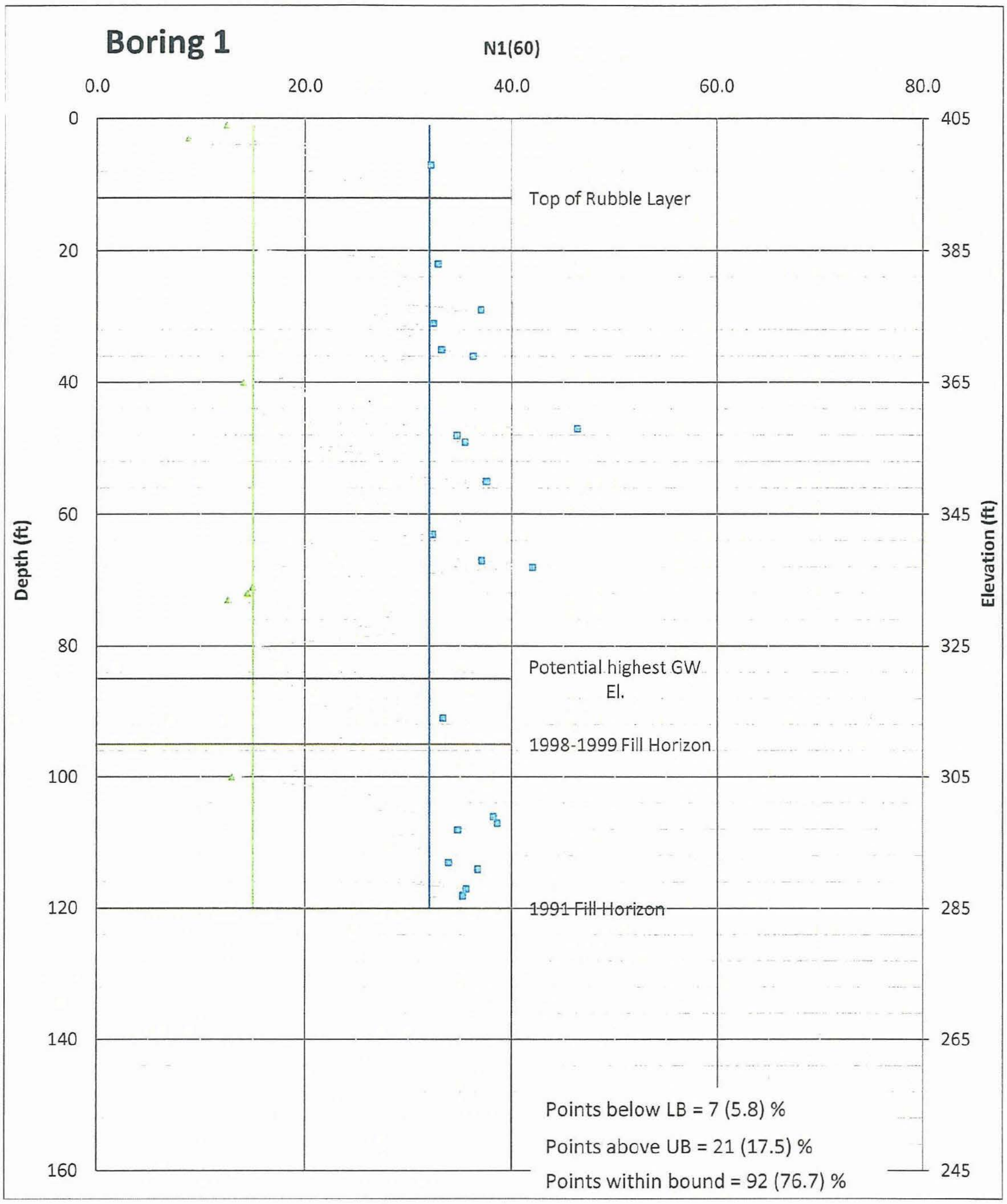
Becker Hammer Blowcount Profiles (from IGI)



Corrected Becker Hammer Blow Counts

Lower Bound (LB) =Becker Blow Counts Corresponding to $N_1(60)$ <15

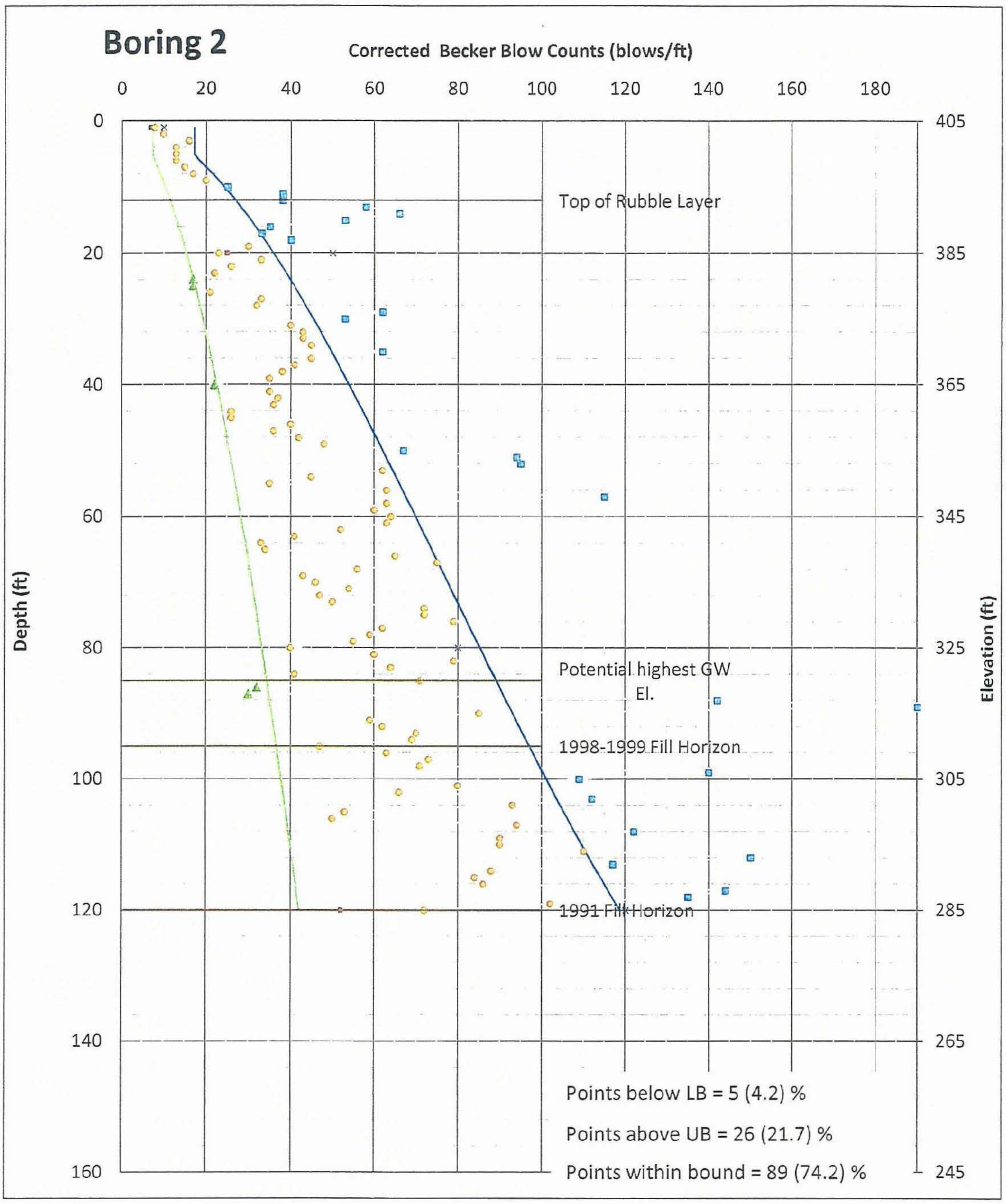
Upper Bound (UB) =Becker Blow Counts Corresponding to $N_1(60)$ >32



Equivalent $N_1(60)$ from Becker Hammer Blow Counts

Lower Bound (LB) = $N_1(60)$ <15

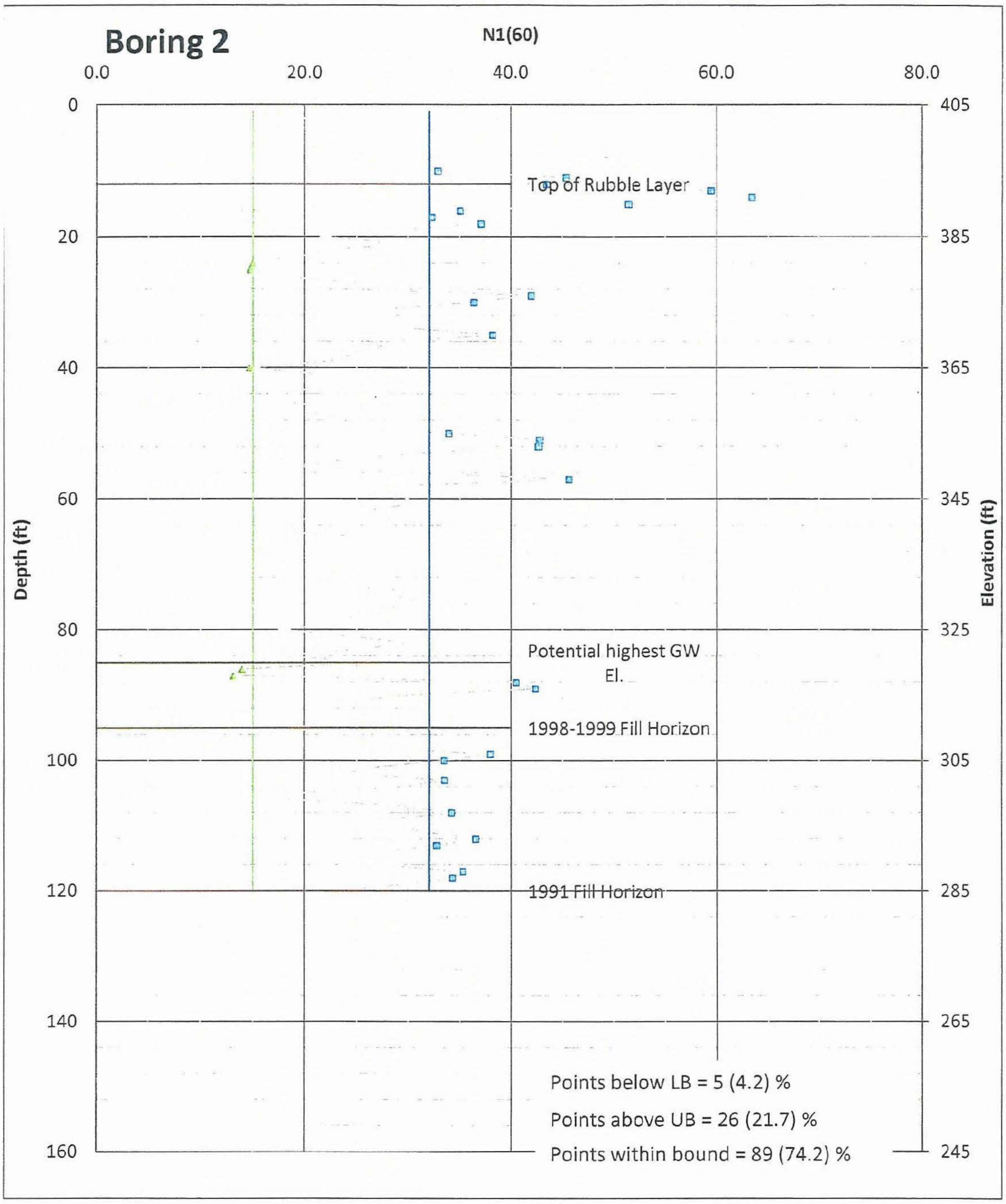
Upper Bound (UB) = $N_1(60)$ >32



Corrected Becker Hammer Blow Counts

Lower Bound (LB) =Becker Blow Counts Corresponding to $N_1(60)$ <15

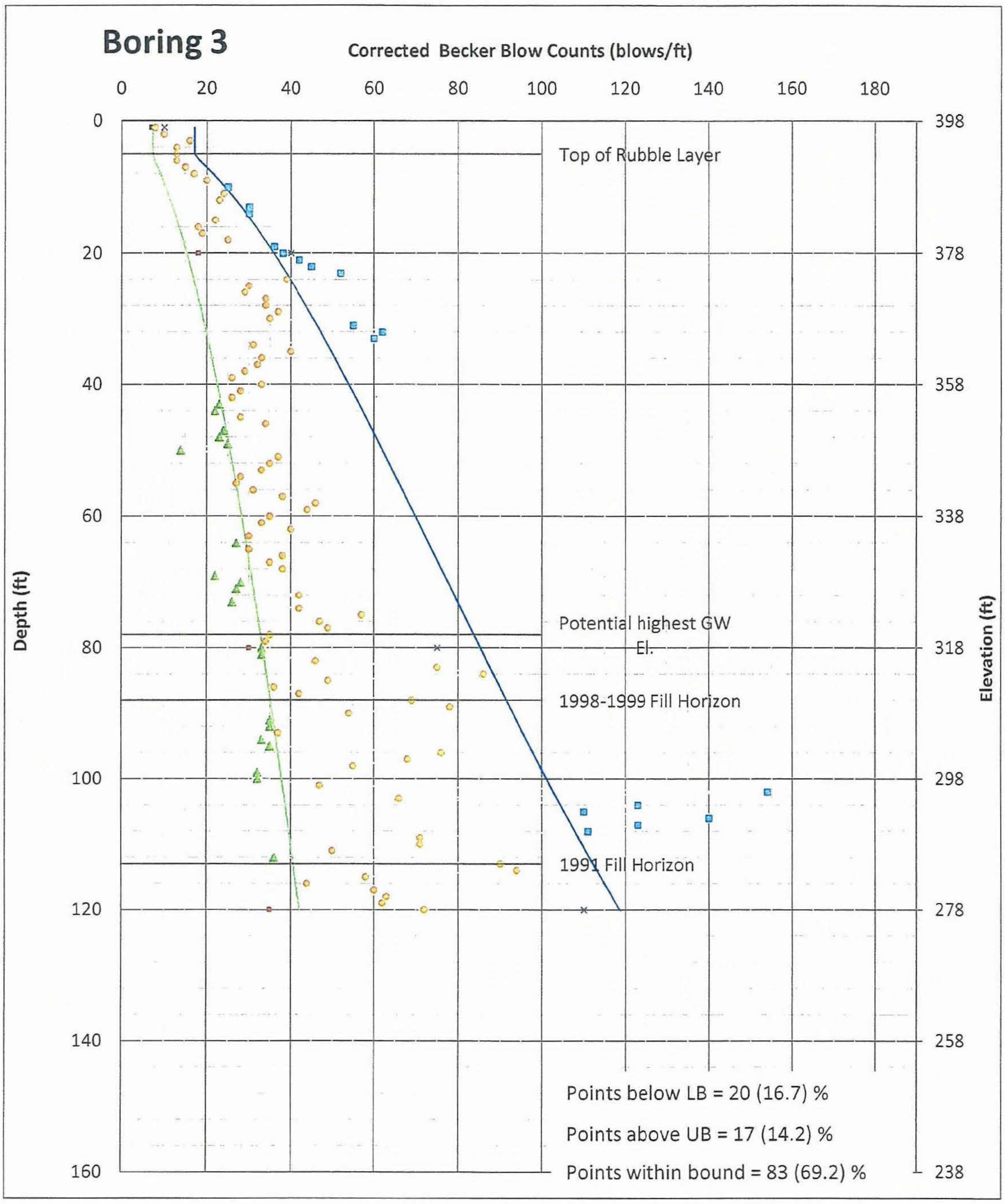
Upper Bound (UB) =Becker Blow Counts Corresponding to $N_1(60)$ >32



Equivalent $N_1(60)$ from Becker Hammer Blow Counts

—— Lower Bound (LB) = $N_1(60)$ <15

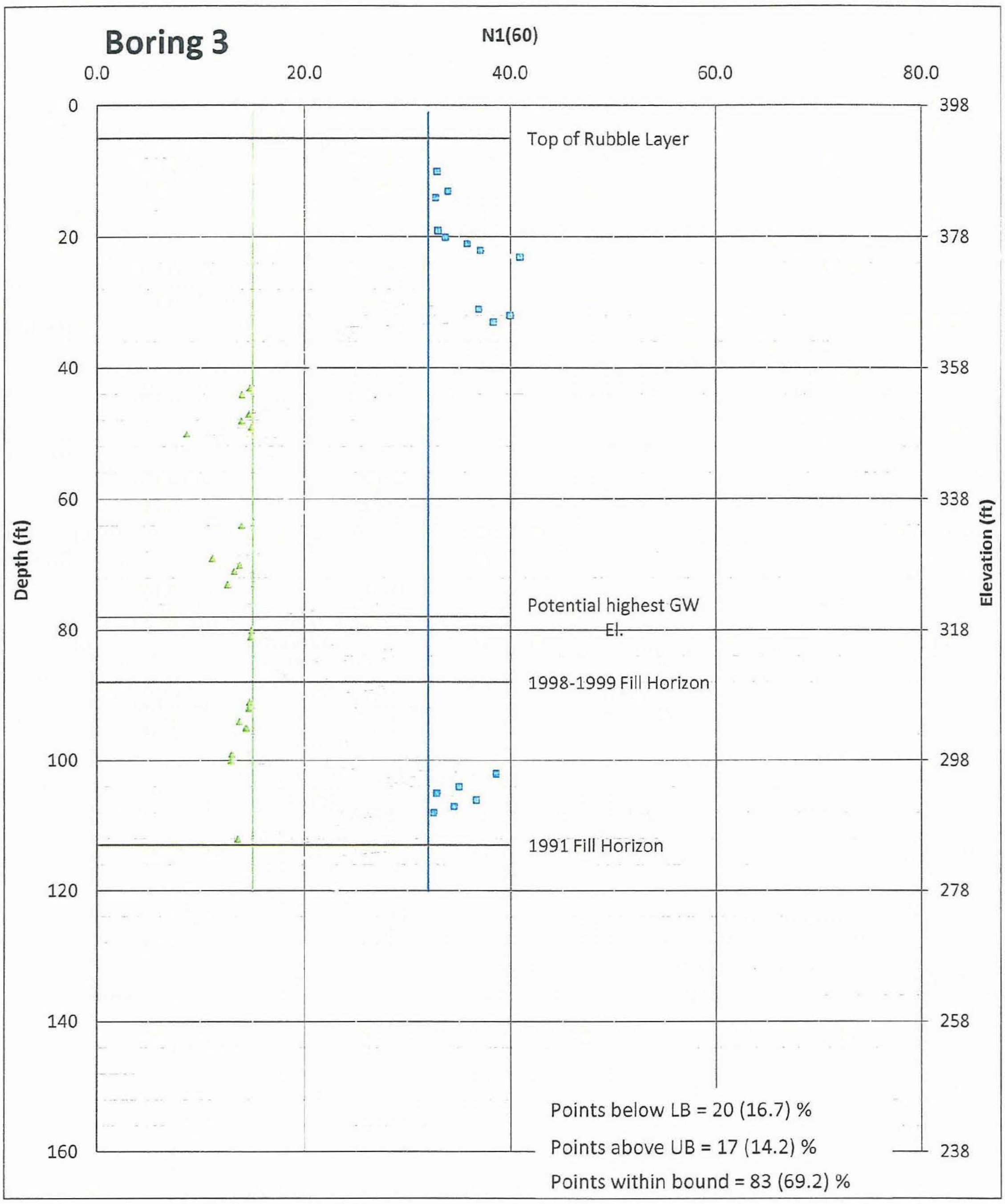
—— Upper Bound (UB) = $N_1(60)$ >32



Corrected Becker Hammer Blow Counts

Lower Bound (LB) =Becker Blow Counts Corresponding to N(60) <15

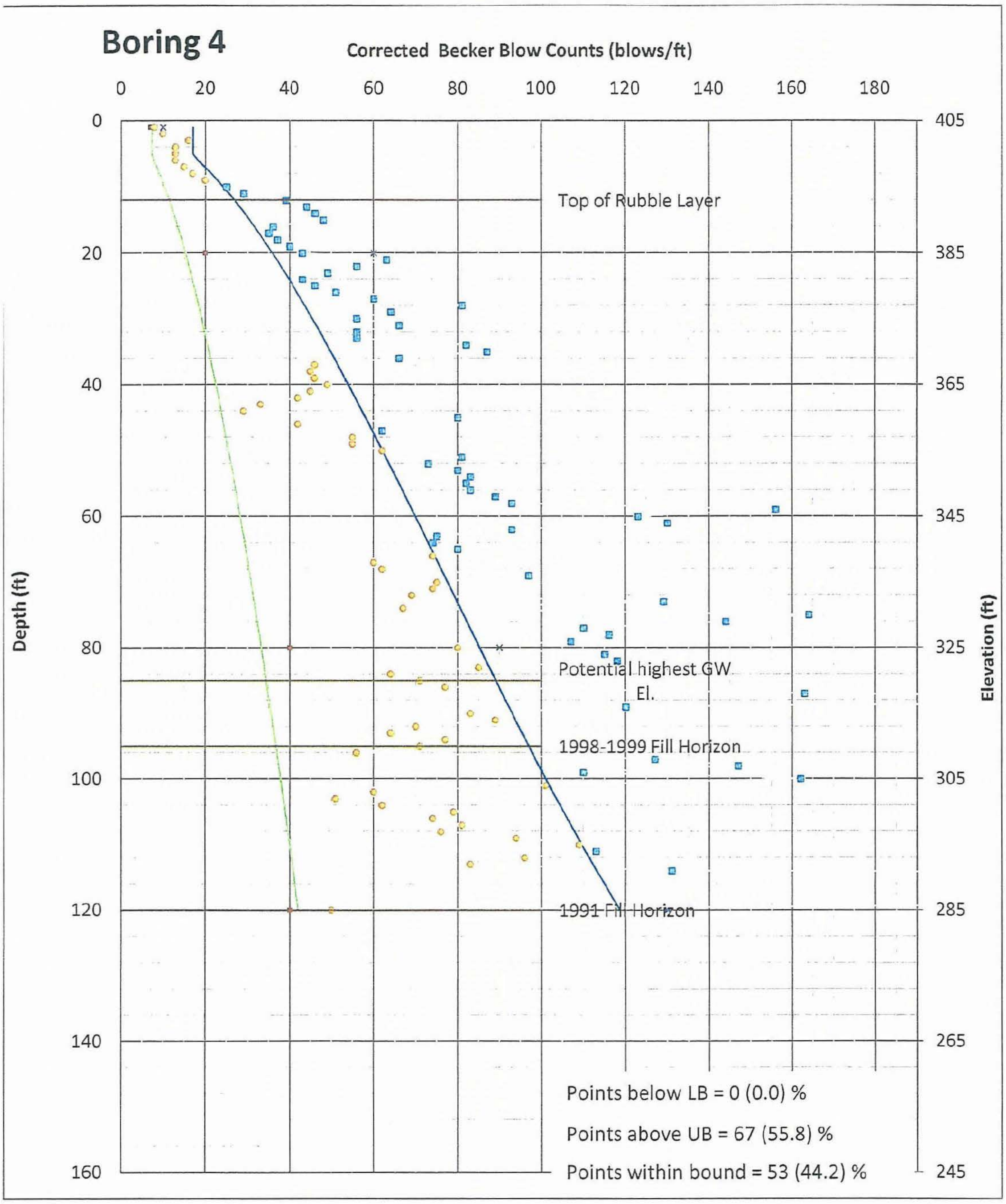
Upper Bound (UB) =Becker Blow Counts Corresponding to N(60) >32



Equivalent $N_1(60)$ from Becker Hammer Blow Counts

Lower Bound (LB) = $N_1(60) < 15$

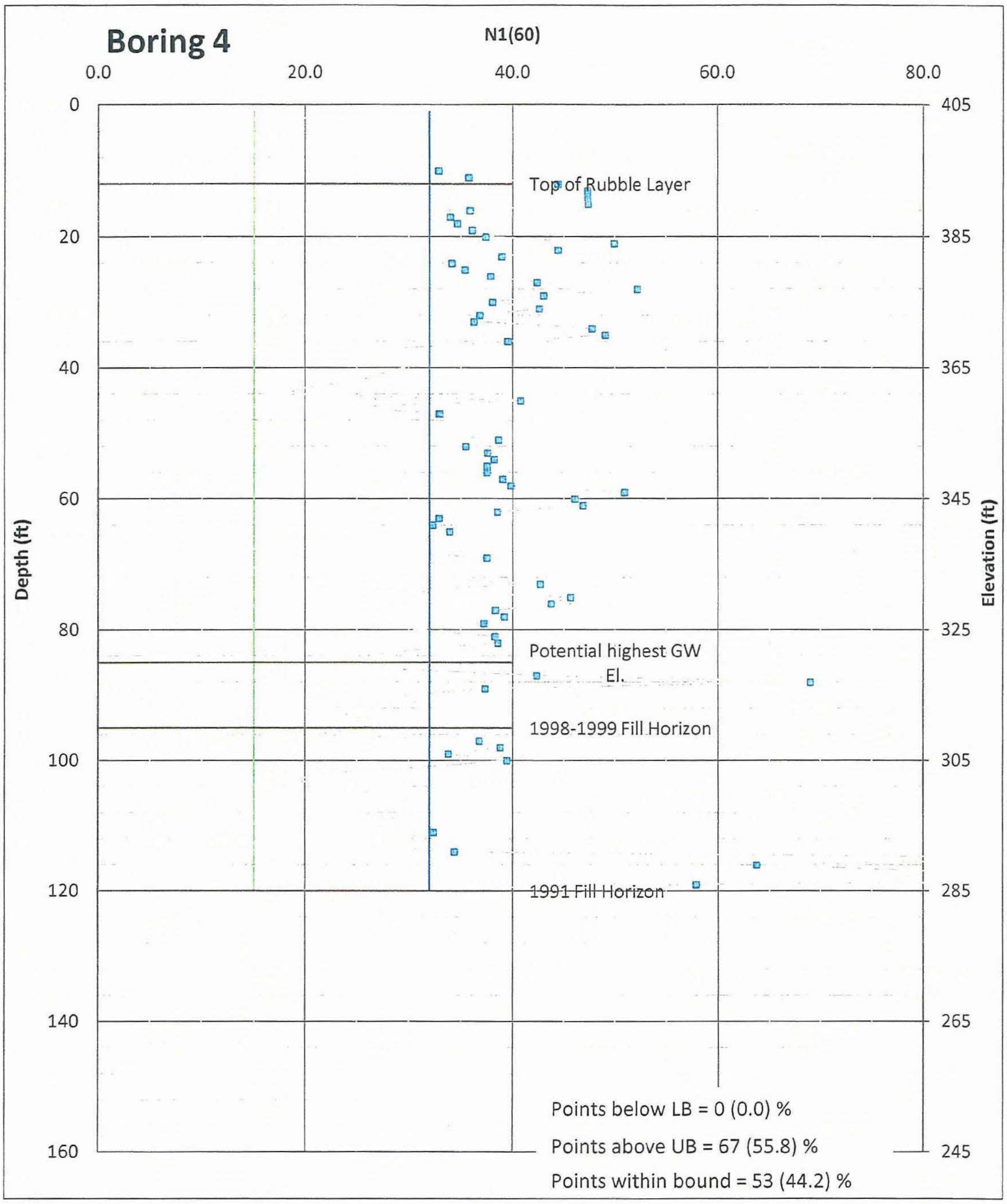
Upper Bound (UB) = $N_1(60) > 32$



Corrected Becker Hammer Blow Counts

Lower Bound (LB) =Becker Blow Counts Corresponding to $N_1(60) < 15$

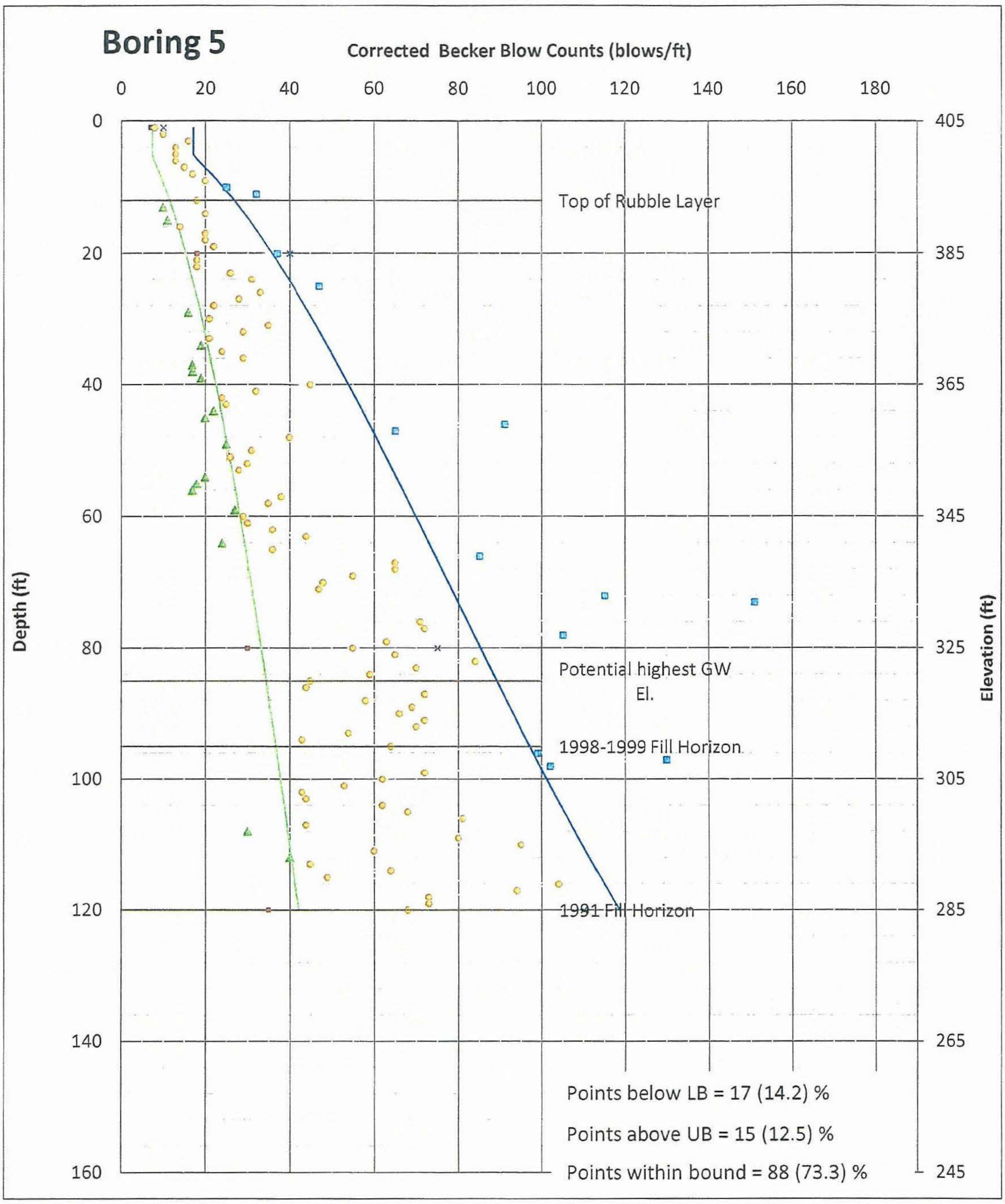
Upper Bound (UB) =Becker Blow Counts Corresponding to $N_1(60) > 32$



Equivalent $N_1(60)$ from Becker Hammer Blow Counts

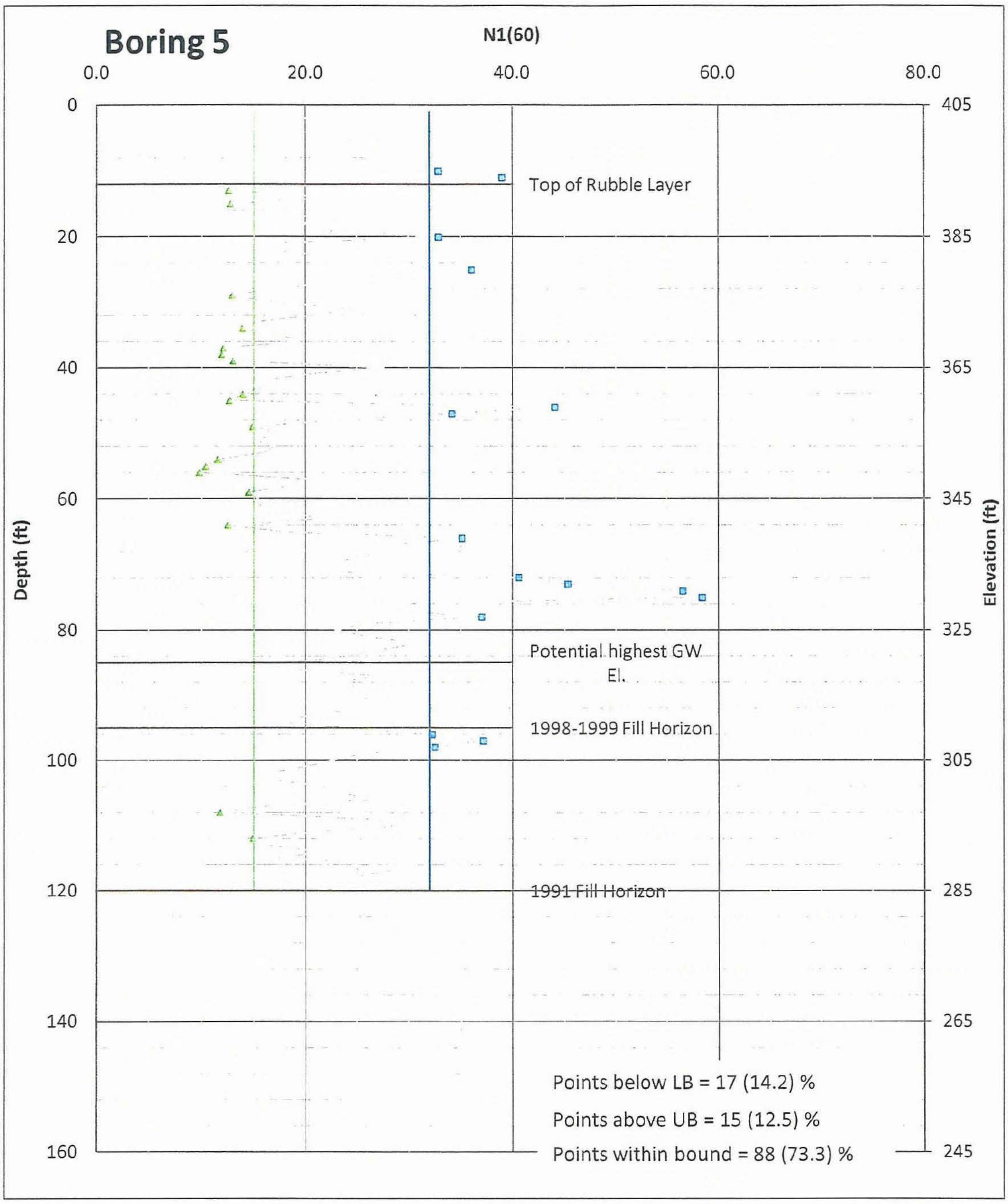
Lower Bound (LB) = $N_1(60) < 15$

Upper Bound (UB) = $N_1(60) > 32$



Corrected Becker Hammer Blow Counts

- Lower Bound (LB) =Becker Blow Counts Corresponding to N(60) <15
- Upper Bound (UB) =Becker Blow Counts Corresponding to N(60) >32



Equivalent $N_1(60)$ from Becker Hammer Blow Counts

Lower Bound (LB) = $N_1(60) < 15$

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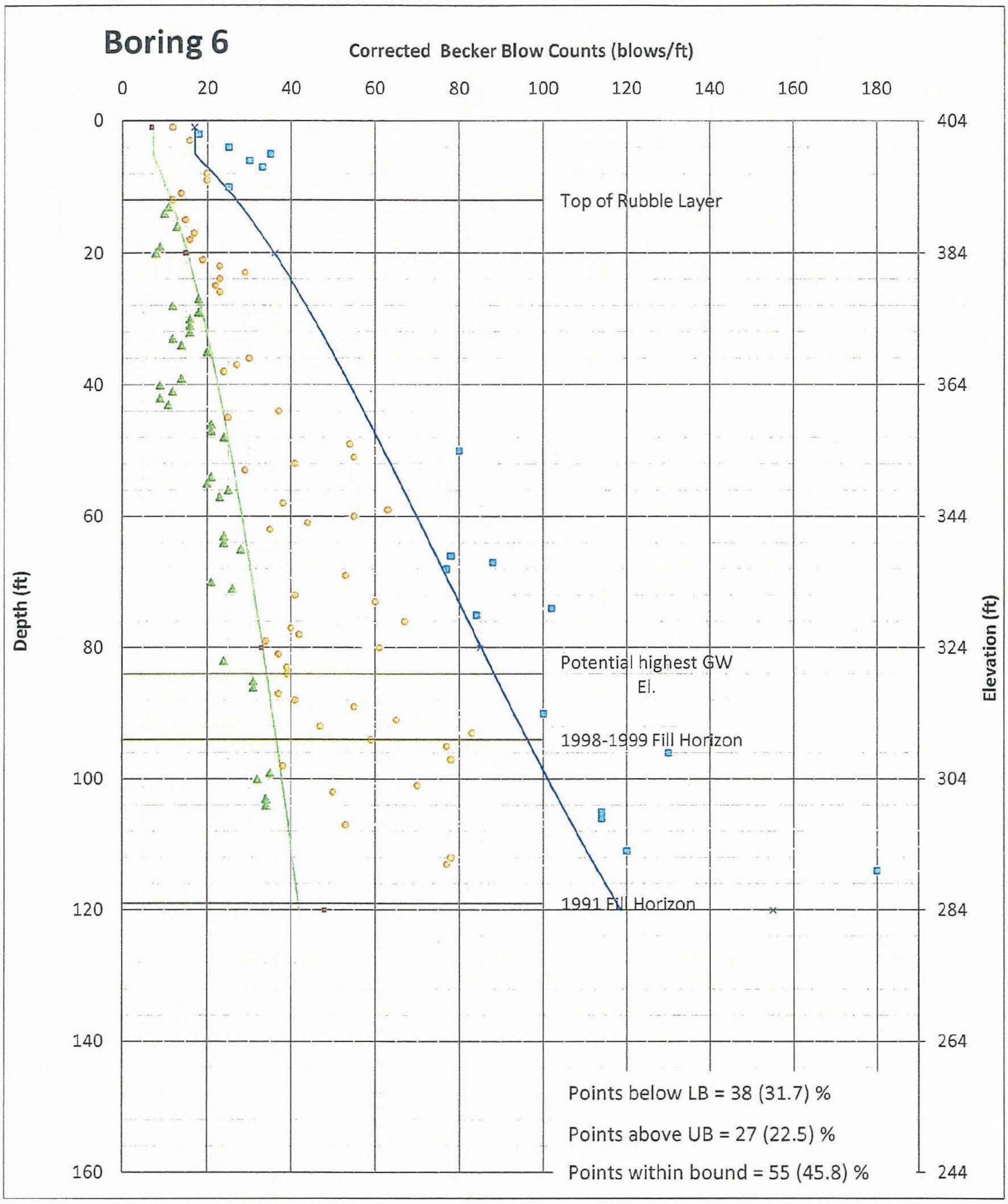

Corrected Becker Hammer Blow Counts

Lower Bound (LB) =Becker Blow Counts Corresponding to $N_1(60)$ <15

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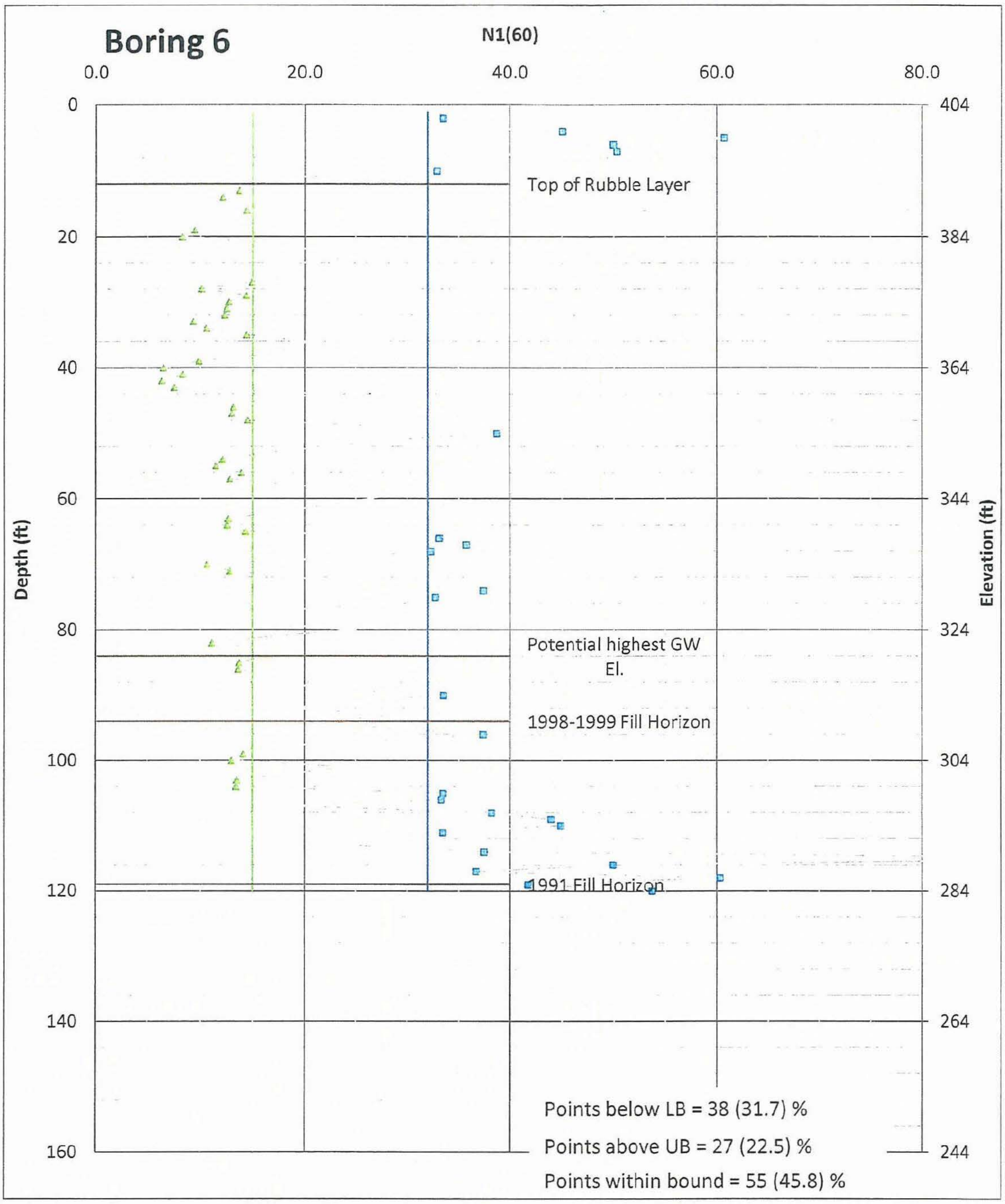


Equivalent $N_1(60)$ from Becker Hammer Blow Counts

Lower Bound (LB) = $N_1(60)$ <15

Upper Bound (UB) = $N_1(60)$ >32

Appendix C
FLAC Model Analysis

FLAC Analysis for Nu-Way Live Oak Pit

Contents

1. Summary

The Nu-Way Live Oak Pit consists of up to 110 ft of improperly placed rubble fill that is vulnerable to large seismic settlements. Potential seismic settlements were estimated based on several parameters including, but not limited to, various field tests, field observations, and empirical correlations for the design earthquake. The estimated seismic settlements occur at various depths below the ground surface. The surface expression of the settlement will include total and differential settlement. The magnitude of the total and differential settlements manifesting at the surface will depend on the depth at which the settlement occurs and the thickness of compacted fill cap overlying the horizon at which the settlement takes place.

The Computer Program FLAC (Fast Lagrangian Analysis of Continua, Itasca, 2006) was used to model the impact of cap thickness on surface manifestation of the underlying settlements. The program uses finite difference numerical techniques to model the non-linear stress – deformation patterns within soils.

A typical cross section (Cross Section C-C' from the IGI report) was analyzed for this purpose. The soil cap (engineered fill) consisting of excavated, processed and recompacted materials was modeled as a non liner elastic – perfectly plastic material with a Mohr-Coulomb yield criterion.

The initial shear modulus (G_{max}) of the soil cap was selected from the shear wave velocity measured from the single downhole geophysical seismic velocity survey performed in Borehole B-4 (Terra-Physics, 2008 and IGI, 2008a). The shear wave velocity of 880 feet/second measured in the upper portion of the existing fill was selected. The shear modulus was degraded as a function of the shear strain based on the G/Gmax backbone curve proposed for sands by Seed & Idriss (1970).

The 2-dimensional cross section was first initialized under gravity load to calculate and apply the in situ stresses. Surface manifestation of settlement was simulated by considering the soil cap and applying the calculated displacements of the existing fills left in place below the fill cap as vertical nodal displacements along the interface between the fill cap and existing fill.

The surface manifestation of settlements occurring at depth was modeled in two different ways:

a) ***Specific Differential Settlement Applied at a Single Location (isolated differential settlement)*** – In the first approach, a specific differential settlement was applied at a single location at the base of the fill cap and the corresponding response at the surface was modeled. The differential settlement due to seismic action was selected to be half of the total settlement, occurring over a relatively short horizontal distance of about 30 feet.

   Surface manifestation of this differential settlement was evaluated for three different thicknesses of fill cap: 40, 60 and 80 feet, respectively. The resulting surficial differential settlements corresponding to different thicknesses of fill cap are illustrated in Figure 1. The figure also provides the range of estimated differential settlement for different percentages of fines intrusion into open voids.

b) ***Randomly Varying Settlement Applied at the Base of the Fill Cap*** – In this approach, the settlement of the rubble fill underlying the fill cap was assumed to vary randomly between the maximum and minimum values calculated. This was accomplished by the use of a random number generator, whereby the specified settlement at any node was calculated as:

   Specified settlement = min.settlement + r . (max.settlement-min.settlement), where r is a random number between 0 and 1.

   The results are summarized in Figure 2. The figure shows the maximum values of surficial total and differential settlements calculated.

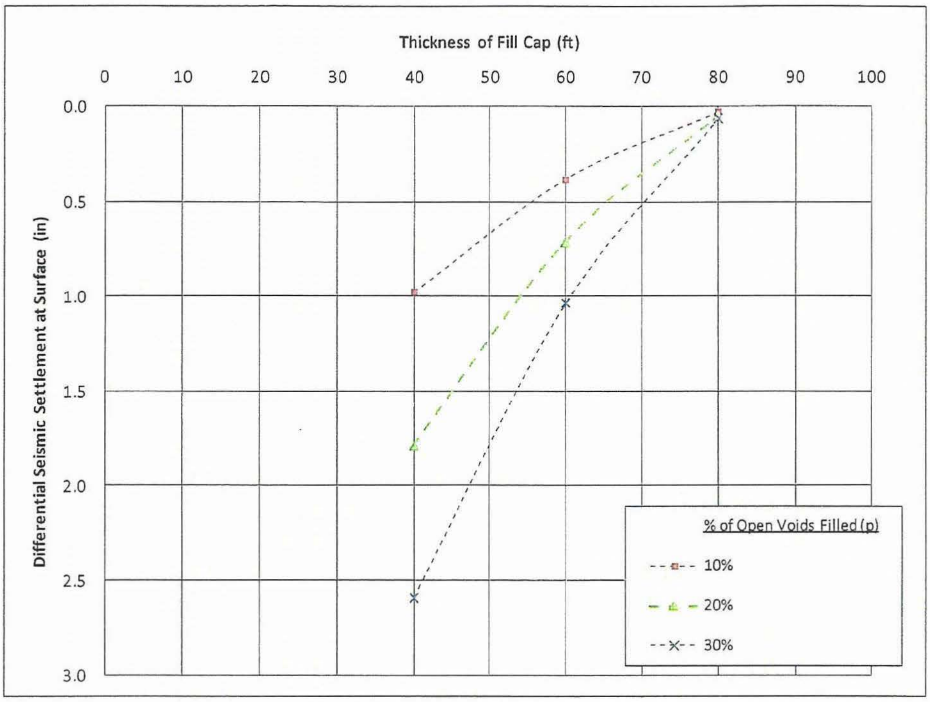


Figure 1. Differential Settlement at Surface vs. Cap Thickness - Isolated Differential Settlement – Case (a)

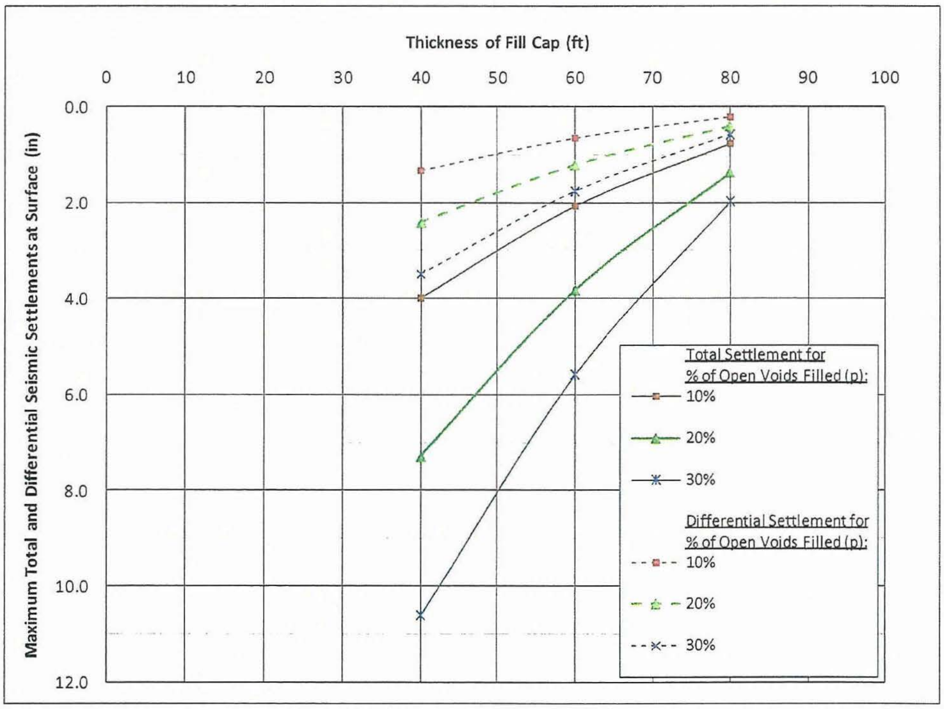


Figure 2. Maximum Total and Differential Settlements at Surface vs. Cap Thickness – Randomized Settlement – Case (b)

2. Modeling

2.1 Grids and Boundary Conditions

Taking advantage of symmetry, only one-half of the cross-section was modeled. A zone (mesh) size of 5'x5' was selected. A schematic diagram of the geometric model analyzed is shown in Figure 3 below.

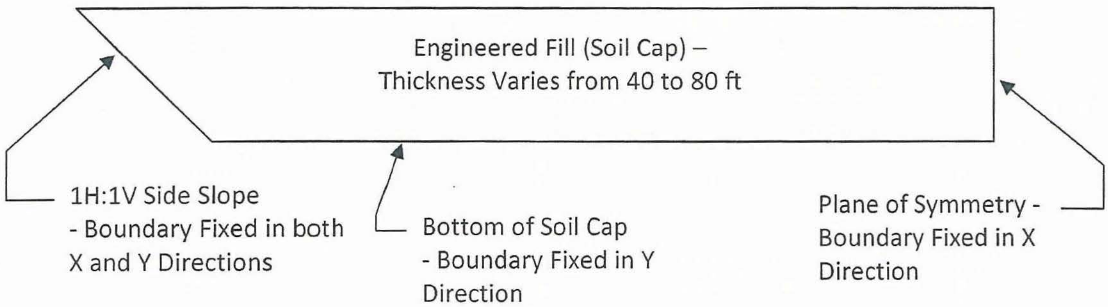


Figure 3. Schematic of Geometric Model Analyzed

Settlements were applied at the bottom (of the cap) grid points (nodes) as a velocity (displacement per time step) boundary condition. In order to minimize the shock to the model, the displacement rate was gradually increased from zero to a stable value (ramping) and then gradually decreased to zero. The increments were calculated from specified total number of steps and estimated total nodal displacement.

2.2 Soil Parameters

Model: Mohr-Coulomb

γ: 3.73 lbm/cu.ft.

Gmax: 2.89×10^6 psf (calculated based on Vs of 880 ft/s)

Kmax: 8.67×10^6 psf (calculated based on υ of 0.35)

C: 250 psf

Ø: 30°

At each time step, the shear modulus of each zone was degraded as a function of the shear strain based on the G/Gmax backbone curve proposed for sands by Seed & Idriss (1970).

2.3 Seismic Settlement Estimates Used for the Analysis

Seismic settlements were estimated at each boring location B1 through B6. Figure 4 shows the estimated total settlements against removal and recompaction (soil cap) thickness at the boring locations and the average settlement curve. For the case (a), differential settlement was assumed to be one-half of the average total settlement. For the Case (b), settlements were randomized between minimum and maximum estimates.

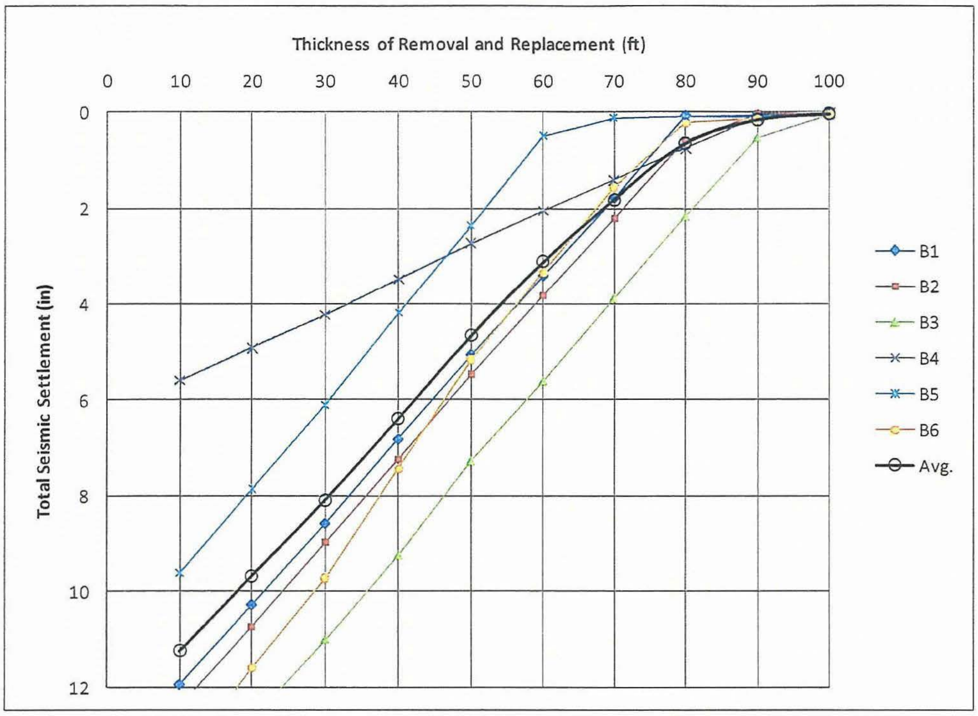


Figure 4. Estimated Total Seismic Settlement of Rubble below the Soil Cap at Each Boring Location

3. Input Files

3.1 Isolated Differential Settlement Over 30 ft

```
new
Title:
Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

;Differential settlement applied over 30ft length for 40, 60 and 80-foot soil caps
;Model only one-half of the soil cap; No settlement of side slope
;Use Modulus values revised using Vs consistent with downhole measurement (880ft/s)
;Run by: TS      Date: Nov 2010

config extra 2
;Initial model for 80 ft soil cap
grid 167,16
model elastic

;Assign coordinates and cut 1:1 side slope
gen same 0. 80. 835. 80. 835. 0.
gen line 80.0,0.0 0.0,80.0
model null region 2 3
group 'null' region 2 3
group delete 'null'

group 'User:new1' notnull
model mohr notnull group 'User:new1'
```

```
prop density=3.73 bulk=8.67E6 shear=2.89e6 cohesion=250.0 friction=30.0 &
 dilation=0.0 tension=0.0 notnull group 'User:new1'

fix  x y mark
fix  y i 18 168 j 1
fix  x i 168 j 1 17
set gravity=32.18504
history 999 unbalanced

def nigp
int n_igp
n_igp=igp
end
nigp

;Solve as elastic material to initialize stresses & then apply specified soil
parameters
solve elastic

def mod_ini
command
ini xd=0.
ini yd=0.
end_command
 loop ii (1,izones)
  loop jj (1,jzones)
      if model(ii,jj) # 1 then
       ex_1(ii,jj)=shear_mod(ii,jj)
       state(ii,jj)=0
      endif
  end_loop
 end_loop
end

;Initialize displacements to zero
mod_ini

his 1 yd i=1 j=17    ;check

;G/Gmax Backbone Curve for Upper Sand (Seed and Idriss, 1970)
table 1 0.,1. 0.000001,1. 0.00000316,1. 0.00001,0.99 0.0000316,0.96 0.0001,0.85 &
0.000316,0.655 0.001,0.37 0.00316,0.19 0.01,0.085 0.1,0.0085

save el_D30NWP.sav

;40-foot thick soil cap ________________________________*************************

m n i=1,167 j=1,8
fix  y i 10 168 j 9

his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=9
his 5 yd i=162 j=9

set large
```

```
def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
      udapp=dis_inc*float(ns)
else
      if ns<=(n_step-ns_ramp)
            udapp=dis_inc*float(ns_ramp)
      else
            udapp=dis_inc*float(n_step-ns)
      endif
endif

command
ini yv=0. var udapp 0. i 162,168 j 9
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
      shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
      bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 tot_dis=-2.66e-1 ;should be negative
superstep

solve
save NL_40D30.sav

;*** plot commands ***
set output pl_D30NWP40_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_D30NWP40_dis.emf
plot pen his 2 3 4 5
set output pl_D30NWP40_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_D30NWP40_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_D30NWP40_st.emf
plot pen state block
```

```
set hisfile his_D30NWP40.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1) o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of simulation'
loop ii (1,igp)
      arr1(ii)=string(x(ii,jgp))
      arr2(ii)=string(y(ii,jgp))
      arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open('Sur_40D30.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_igp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out

;60-foot thick soil cap ________________________________**************************

res el_D30NWP.sav

m n i=1,167 j=1,4

fix y i 14 168 j 5

his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=5
his 5 yd i=162 j=5

set large

def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
      udapp=dis_inc*float(ns)
else
      if ns<=(n_step-ns_ramp)
            udapp=dis_inc*float(ns_ramp)
      else
            udapp=dis_inc*float(n_step-ns)
      endif
endif

command
ini yv=0. var udapp 0. i 162,168 j 5
```

```
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
      shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
      bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 tot_dis=-1.3e-1 ;should be negative
superstep

solve
save NL_60D30.sav

;*** plot commands ***
set output pl_D30NWP60_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_D30NWP60_dis.emf
plot pen his 2 3 4 5
set output pl_D30NWP60_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_D30NWP60_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_D30NWP60_st.emf
plot pen state block

set hisfile his_D30NWP60.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1) o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of simulation'
loop ii (1,igp)
      arr1(ii)=string(x(ii,jgp))
      arr2(ii)=string(y(ii,jgp))
      arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open ('Sur_60D30.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_igp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out

;80-foot thick soil cap ________________________________**************************
```

```
res el_D30NWP.sav

his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=1
his 5 yd i=162 j=1

set large

def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
      udapp=dis_inc*float(ns)
else
      if ns<=(n_step-ns_ramp)
            udapp=dis_inc*float(ns_ramp)
      else
            udapp=dis_inc*float(n_step-ns)
      endif
endif

command
ini yv=0. var udapp 0. i 162,168 j 1
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
      shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
      bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 tot_dis=-0.27e-1 ;should be negative
superstep

solve
save NL_80D30.sav

;*** plot commands ***
set output pl_D30NWP80_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_D30NWP80_dis.emf
```

```
plot pen his 2 3 4 5
set output pl_D30NWP80_def.emf
plot pen grid mag 0 gr grid mag 100 red
window 700 850 -50 100
set output pl_D30NWP80_def_zo.emf
pl pen grid mag 0 gr grid mag 100 red
set output pl_D30NWP80_st.emf
plot pen state block

set hisfile his_D30NWP80.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1) o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of simulation'
loop ii (1,igp)
      arr1(ii)=string(x(ii,jgp))
      arr2(ii)=string(y(ii,jgp))
      arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open ('Sur_80D30.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_igp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out
ret
```

3.2 Randomly Varying Settlement

```
new
Title:
Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

;Differential settlement applied randomly over entire length for 40, 60 and 80-foot
soil caps
;Potential settlements gradually increase from the edge to about 100 ft
;Model only one-half of the soil cap; No settlement of side slope
;Use Modulus values revised using Vs consistent with downhole measurement (880ft/s)
;Run by: TS       Date: Nov 2010

config extra 2
;Initial model for 80 ft soil cap
grid 167,16
model elastic
;Assign coordinates and cut 1:1 side slope
gen same 0. 80. 835. 80. 835. 0.
gen line 80.0,0.0 0.0,80.0
model null region 2 3
group 'null' region 2 3
group delete 'null'

group 'User:new1' notnull
model mohr notnull group 'User:new1'
prop density=3.73 bulk=8.67E6 shear=2.89e6 cohesion=250.0 friction=30.0 &
```

```
 dilation=0.0 tension=0.0 notnull group 'User:new1'

fix  x y mark
fix  y i 18 168 j 1
fix  x i 168 j 1 17
set gravity=32.18504
history 999 unbalanced

def nigp
int n_igp
n_igp=igp
end
nigp
def nran
array n_ran(n_igp)
loop ii(1,n_igp)
n_ran(ii)=urand
end_loop
end
nran
;Solve as elastic material to initialize stresses & then apply specified soil
parameters
solve elastic

def mod_ini
command
ini xd=0.
ini yd=0.
end_command
 loop ii (1,izones)
  loop jj (1,jzones)
      if model(ii,jj) # 1 then
       ex_1(ii,jj)=shear_mod(ii,jj)
       state(ii,jj)=0
      endif
  end_loop
 end_loop
end

;Initialize displacements to zero
mod_ini
his 1 yd i=1 j=17    ;check

;G/Gmax Table for Upper Sand (Seed and Idriss, 1970; Idriss, 1990)
table 1 0.,1. 0.000001,1. 0.00000316,1. 0.00001,0.99 0.0000316,0.96 0.0001,0.85 &
0.000316,0.655 0.001,0.37 0.00316,0.19 0.01,0.085 0.1,0.0085

save el_RNWP2.sav

;40-foot thick soil cap ________________________________*************************

m n i=1,167 j=1,8
fix  y i 10 168 j 9
his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=9
his 5 yd i=162 j=9
```

```
set large

def superstep
array dis_inc(n_igp)
float dif_dis udapp
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec

loop ii(10,20)
dis_inc(ii)=(ii-9)*(min_dis+n_ran(ii)*dif_dis)/Q/12.
end_loop

loop ii(21,igp)
dis_inc(ii)=(min_dis+n_ran(ii)*dif_dis)/Q
end_loop

loop ns (1,n_step)
 if ns<=ns_ramp
      udapp=float(ns)
 else
      if ns<=(n_step-ns_ramp)
            udapp=float(ns_ramp)
      else
            udapp=float(n_step-ns)
      endif
 endif

      loop ii(10,igp)
            uda=dis_inc(ii)*udapp
            yvel(ii,9)=uda
      end_loop
command
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
      shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
      bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 min_dis=-2.9e-1 dif_dis=-4.81e-1 ;should be negative
superstep

solve
save NL_40R.sav

;*** plot commands ***
set output pl_RNWP240_un.emf
set plot emf
```

```
;plot name: Unbalanced force
plot pen history 999
set output pl_RNWP240_dis.emf
plot pen his 2 3 4 5
set output pl_RNWP240_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_RNWP240_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_RNWP240_st.emf
plot pen state block
set hisfile his_RNWP240.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1) o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of simulation'
loop ii (1,igp)
      arr1(ii)=string(x(ii,jgp))
      arr2(ii)=string(y(ii,jgp))
      arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open('Sur_40R.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_igp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out

;60-foot thick soil cap ________________________________**************************

res el_RNWP2.sav
m n i=1,167 j=1,4
fix y i 14 168 j 5
his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=5
his 5 yd i=162 j=5
set large

def superstep
array dis_inc(n_igp)
float dif_dis udapp
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec

loop ii(14,20)
dis_inc(ii)=(ii-13)*(min_dis+n_ran(ii)*dif_dis)/Q/8.
end_loop

loop ii(21,igp)
dis_inc(ii)=(min_dis+n_ran(ii)*dif_dis)/Q
```

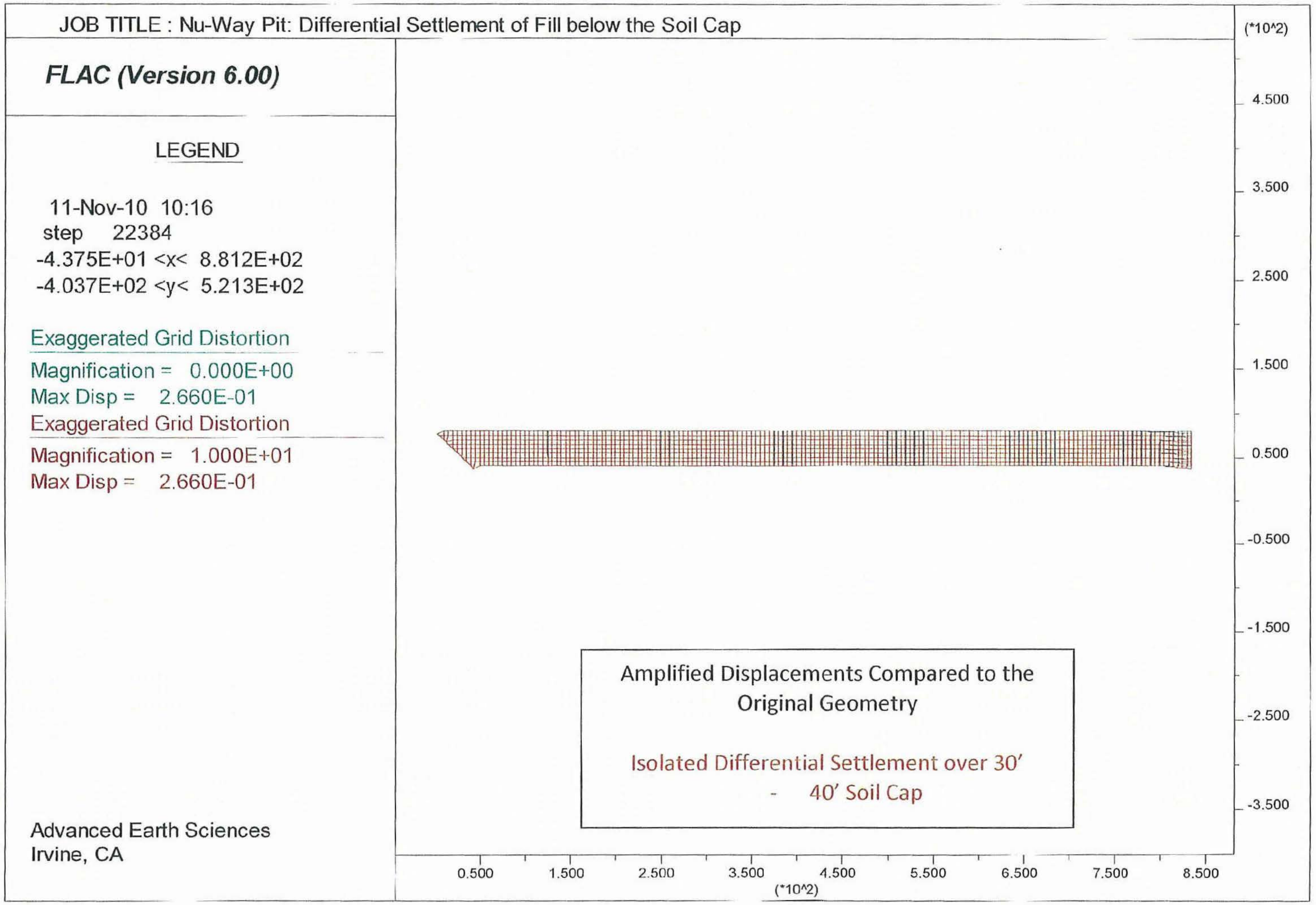
```
end_loop
loop ns (1,n_step)
 if ns<=ns_ramp
      udapp=float(ns)
 else
      if ns<=(n_step-ns_ramp)
            udapp=float(ns_ramp)
      else
            udapp=float(n_step-ns)
      endif
 endif

      loop ii(14,igp)
            uda=dis_inc(ii)*udapp
            yvel(ii,5)=uda
      end_loop
command
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
      shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
      bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop
end_loop
end

set n_step=20000 min_dis=-0.4e-1 dif_dis=-4.27e-1 ;should be negative
superstep

solve
save NL_60R.sav

;*** plot commands ***
set output pl_RNWP260_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_RNWP260_dis.emf
plot pen his 2 3 4 5
set output pl_RNWP260_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_RNWP260_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_RNWP260_st.emf
plot pen state block

set hisfile his_RNWP260.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1) o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of simulation'
```

```
o_h2(1)='Y-Coordinate of surface nodes at the end of simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of simulation'
loop ii (1,igp)
      arr1(ii)=string(x(ii,jgp))
      arr2(ii)=string(y(ii,jgp))
      arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open ('Sur_60R.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_igp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out

;80-foot thick soil cap ________________________________*************************

res el_RNWP2.sav
his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=1
his 5 yd i=162 j=1
set large

def superstep
array dis_inc(n_igp)
float dif_dis udapp
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec

loop ii(18,20)
dis_inc(ii)=(ii-17)*(min_dis+n_ran(ii)*dif_dis)/Q/4.
end_loop

loop ii(21,igp)
dis_inc(ii)=(min_dis+n_ran(ii)*dif_dis)/Q
end_loop
loop ns (1,n_step)
 if ns<=ns_ramp
      udapp=float(ns)
 else
      if ns<=(n_step-ns_ramp)
            udapp=float(ns_ramp)
      else
            udapp=float(n_step-ns)
      endif
 endif

      loop ii(18,igp)
            uda=dis_inc(ii)*udapp
            yvel(ii,1)=uda
      end_loop
command
step 1
end_command
```

```
 loop ii (1,izones)
  loop jj (1,jzones)
      shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
      bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 min_dis=-0.06e-1 dif_dis=-1.73e-1 ;should be negative
superstep

solve
save NL_80R.sav

;*** plot commands ***
set output pl_RNWP280_un.emf
set plot emf
;plot name: Unbalanced force
plot pen history 999
set output pl_RNWP280_dis.emf
plot pen his 2 3 4 5
set output pl_RNWP280_def.emf
plot pen grid mag 0 gr grid mag 100 red
window 700 850 -50 100
set output pl_RNWP280_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_RNWP280_st.emf
plot pen state block
set hisfile his_RNWP280.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1) o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of simulation'
loop ii (1,igp)
      arr1(ii)=string(x(ii,jgp))
      arr2(ii)=string(y(ii,jgp))
      arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open ('Sur_80R.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_igp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out
ret
```

4. Output Plots

4.1 Isolated Differential Settlement Over 30 ft



FLAC-18

0916.0247

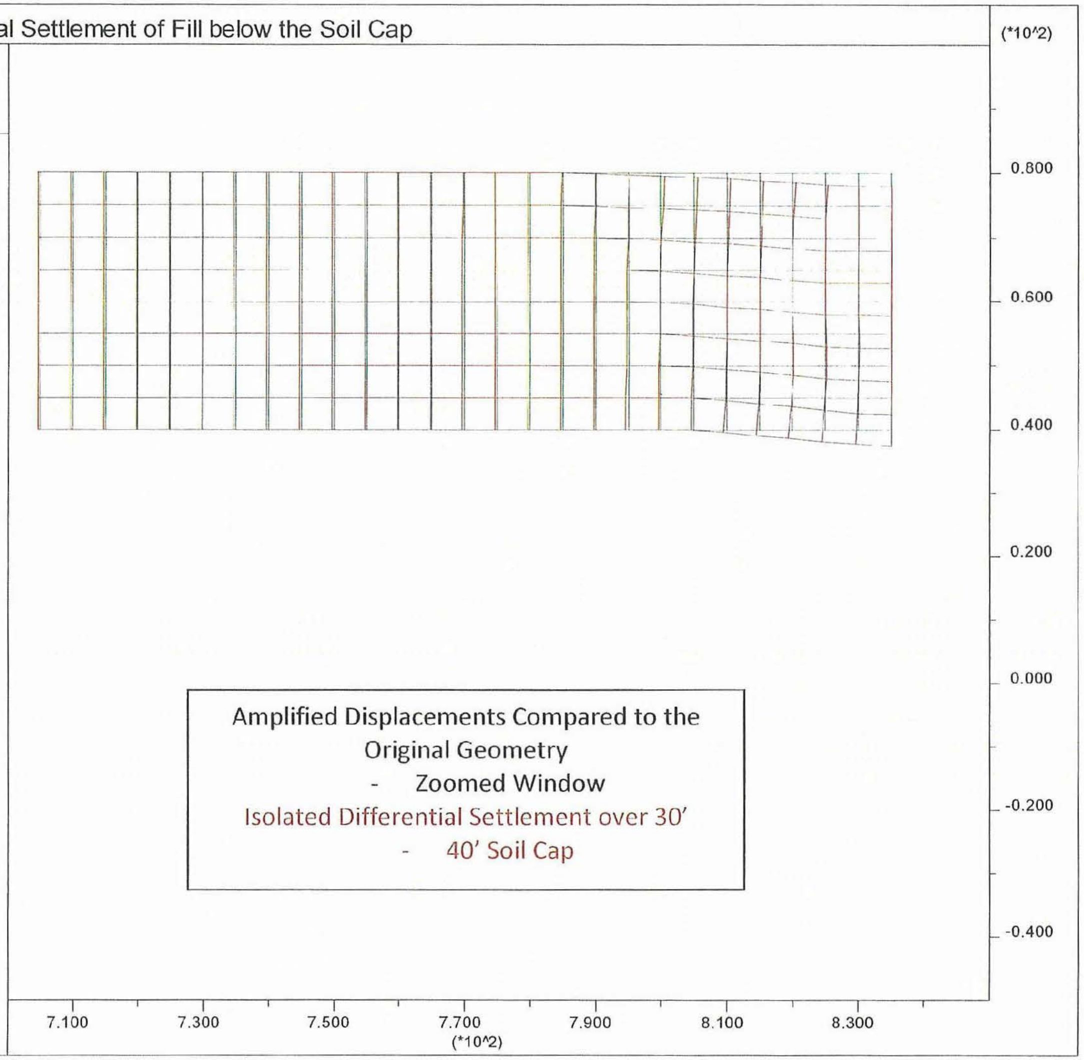
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:16
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.660E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 2.660E-01



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FLAC-20

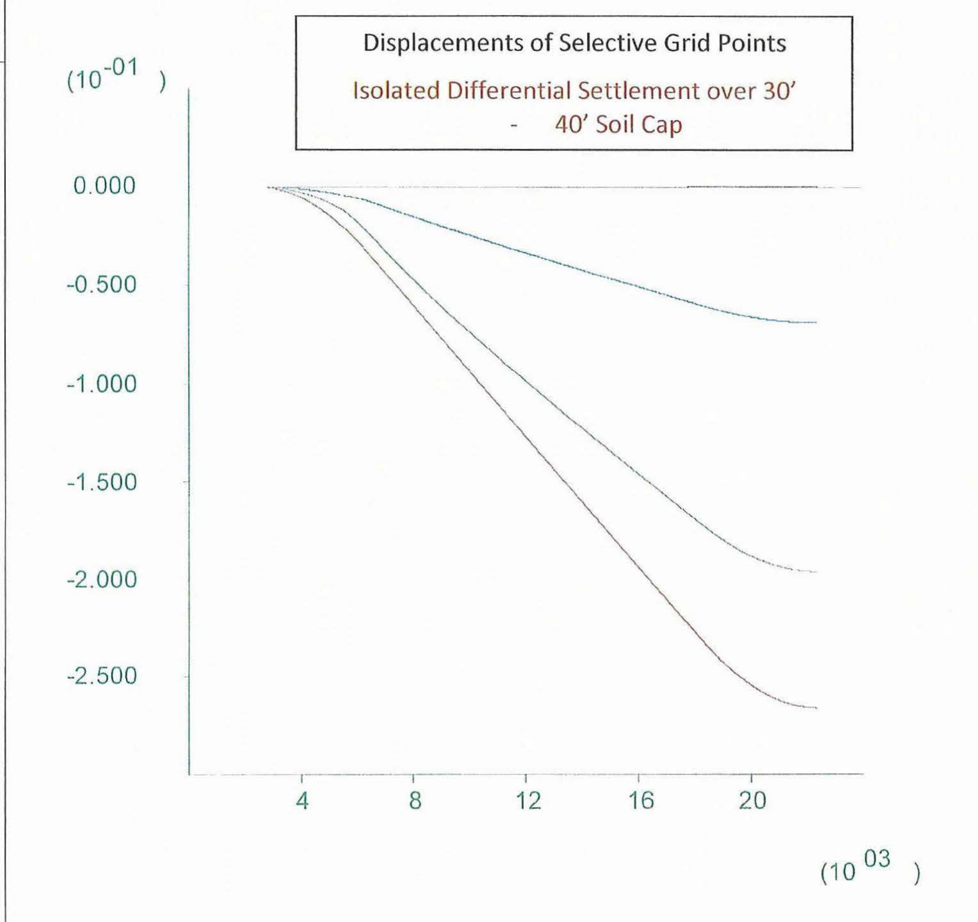
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:16
step 22384

HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 9)
5 Y displacement(162, 9)
X-axis :
Number of steps



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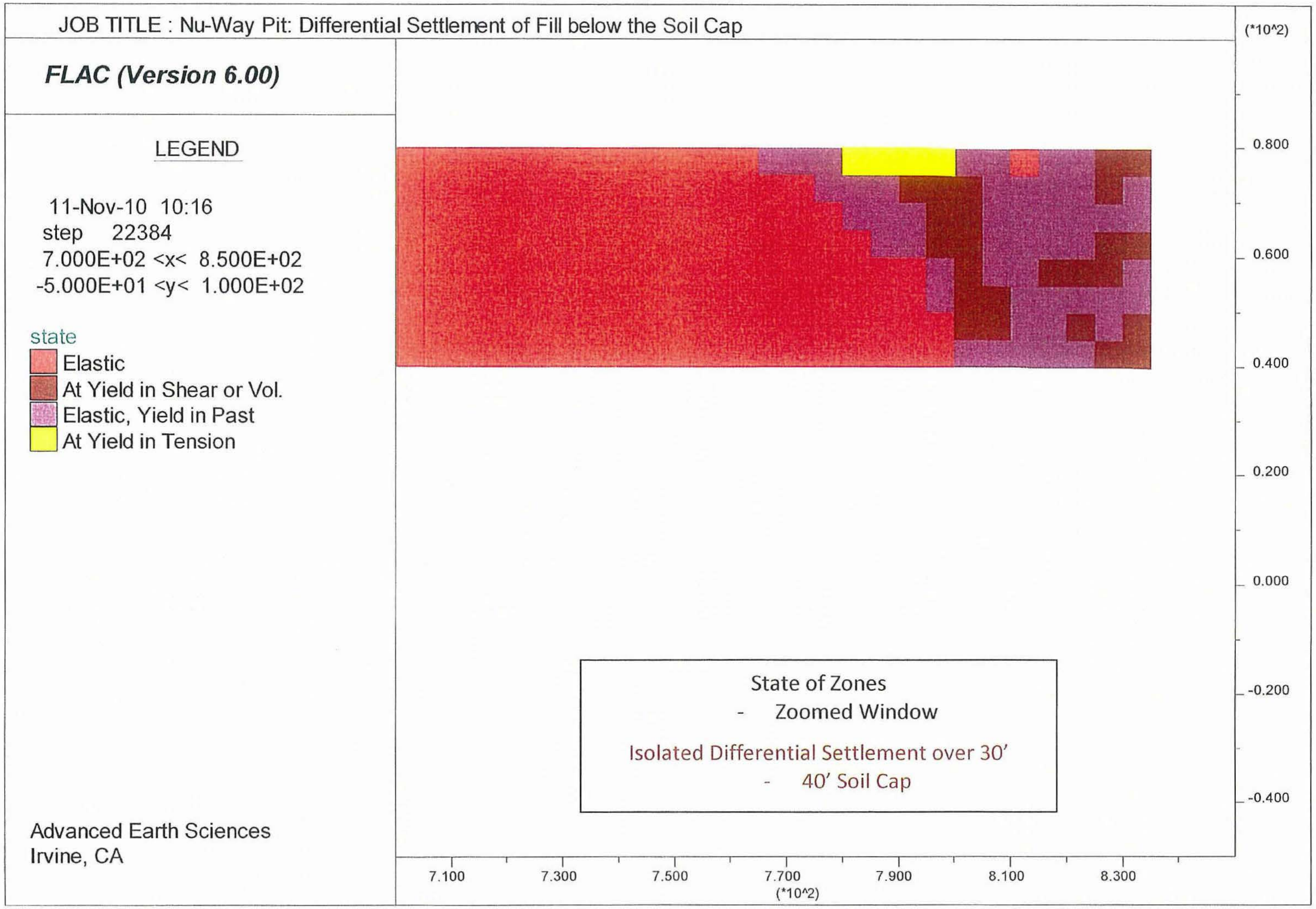


FLAC-22

0916.0251

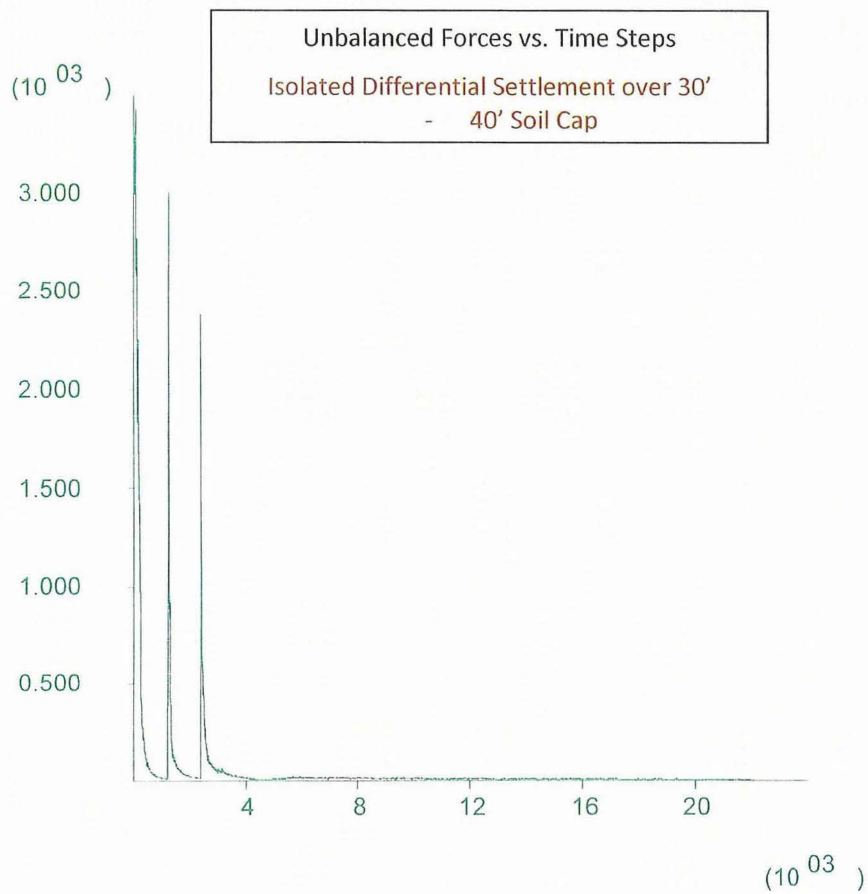
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:16
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:18
step 22384
-4.375E+01 <x< 8.812E+02
-4.125E+02 <y< 5.125E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 1.300E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 1.300E-01

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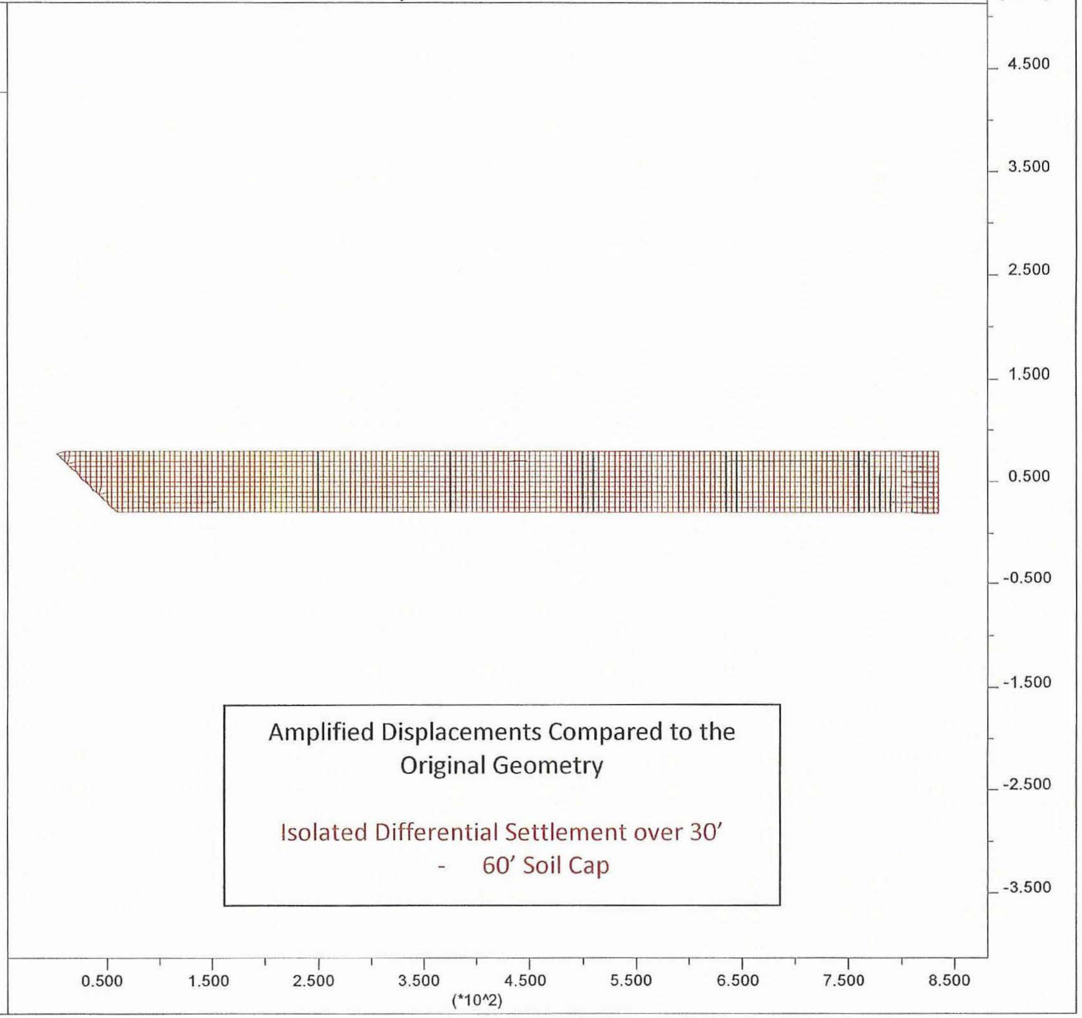

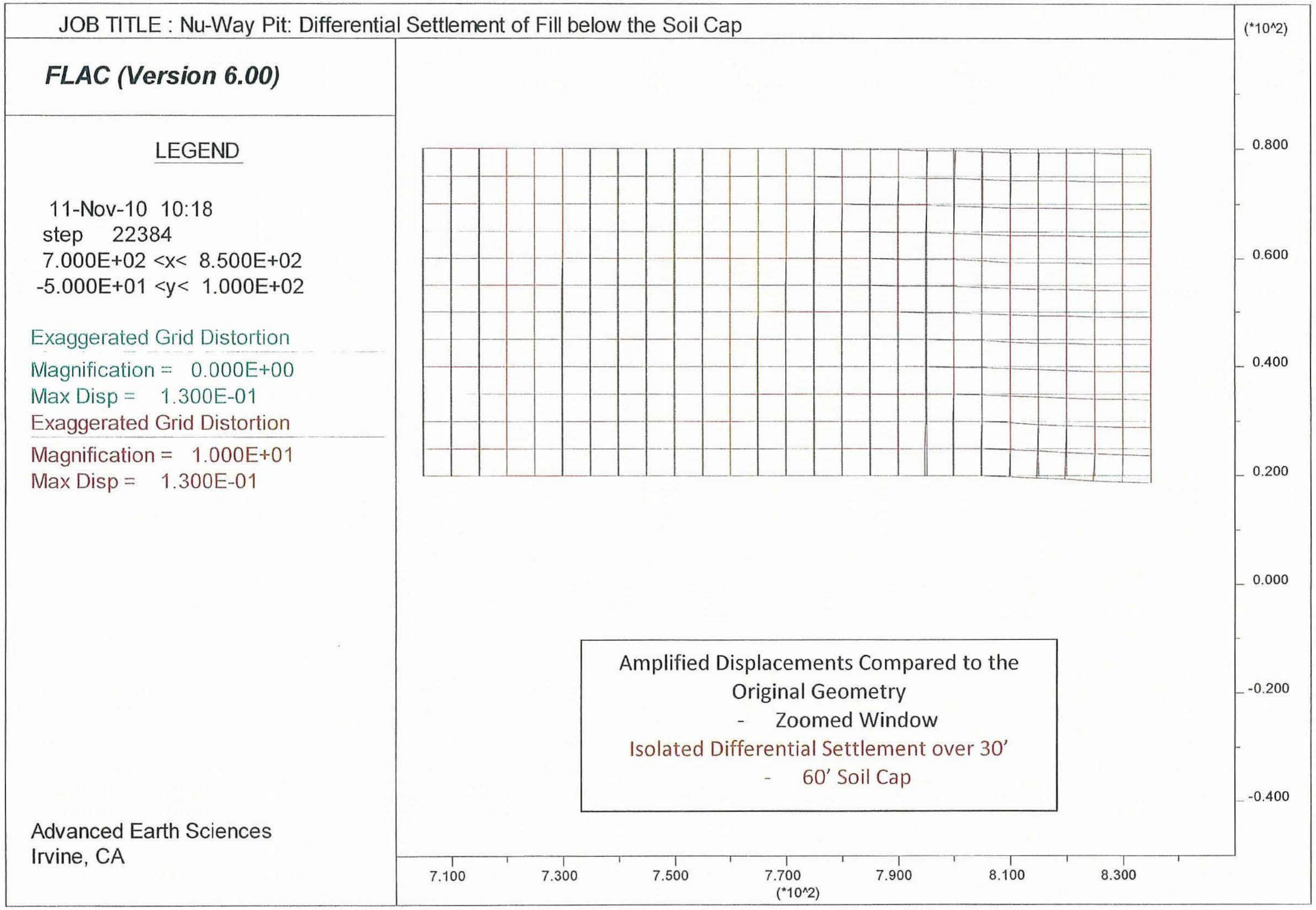
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 10:18
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02
Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 1.300E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 1.300E-01
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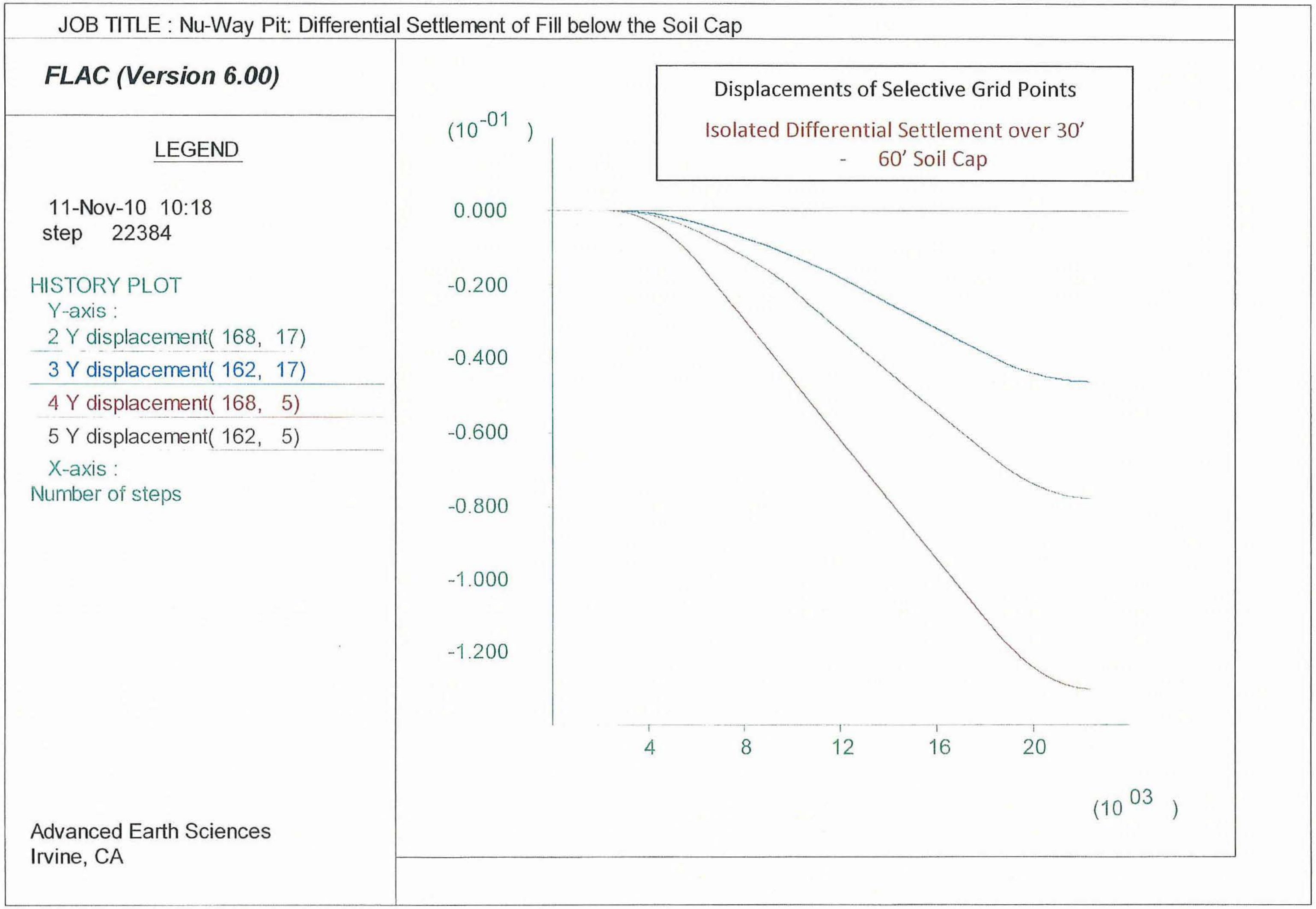
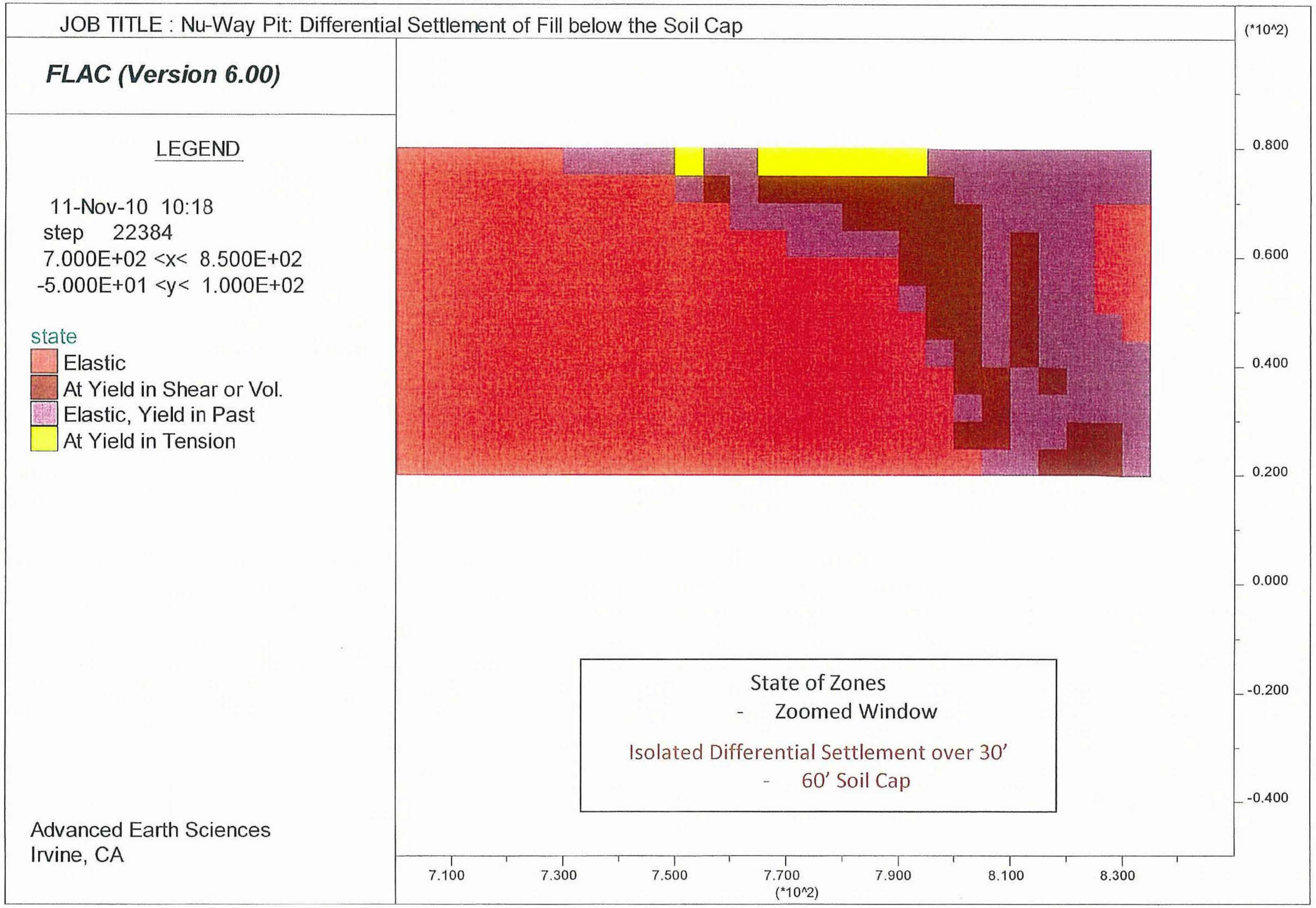
Irvine, CA
Amplified Displacements Compared to the Original Geometry - Zoomed Window
Isolated Differential Settlement over 30' - 60' Soil Cap
(*10^2)
0.800
0.600
0.400
0.200
0.000
-0.200
-0.400
7.100
7.300
7.500
7.700
7.900
8.100
8.300
(*10^2)

FLAC-25


JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 10:18
step 22384
HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 5)
5 Y displacement(162, 5)
X-axis :
Number of steps
Advanced Earth Sciences
Irvine, CA
Displacements of Selective Grid Points
Isolated Differential Settlement over 30' - 60' Soil Cap
(10^-01)
0.000
-0.200
-0.400
-0.600
-0.800
-1.000
-1.200
4
8
12
16
20
(10^03)

FLAC-26


JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 10:18
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02
state
Elastic
At Yield in Shear or Vol.
Elastic, Yield in Past
At Yield in Tension
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(*10^2)
0.800
0.600
0.400
0.200
0.000
-0.200
-0.400
7.100
7.300
7.500
7.700
7.900
8.100
8.300
(*10^2)
State of Zones
- Zoomed Window
Isolated Differential Settlement over 30'
- 60' Soil Cap

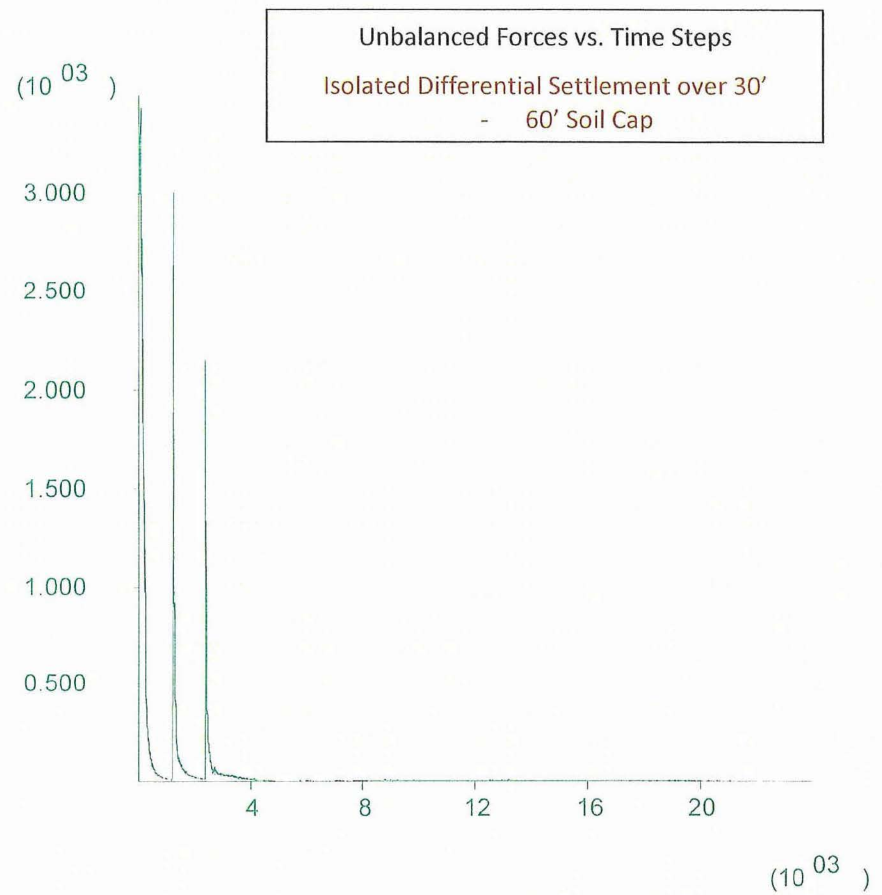
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:18
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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FLAC-28

0916.0257

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:21
step 22384
-4.375E+01 <x< 8.812E+02
-4.225E+02 <y< 5.025E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.700E-02
Exaggerated Grid Distortion
Magnification = 1.000E+02
Max Disp = 2.700E-02

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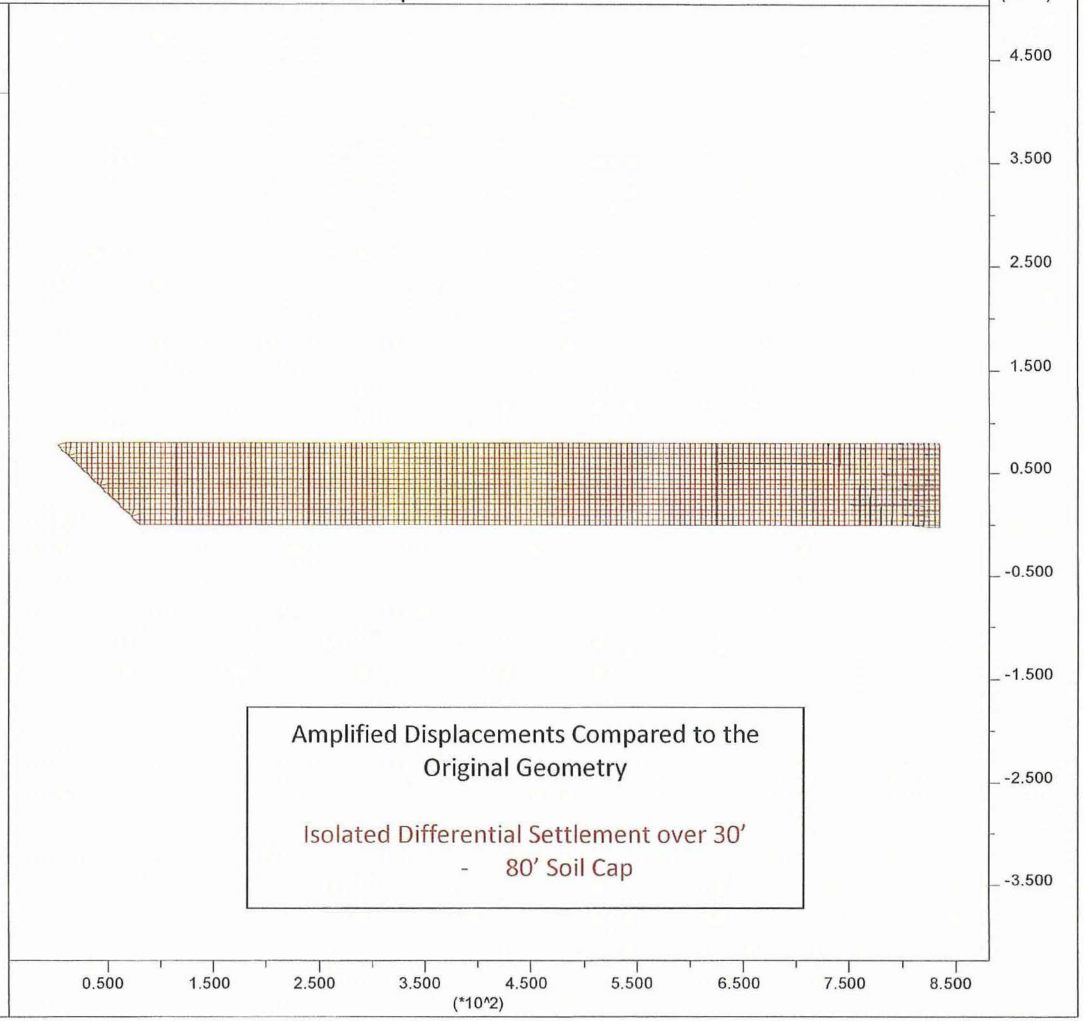

0916.0258

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:21
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.700E-02

Exaggerated Grid Distortion
Magnification = 1.000E+02
Max Disp = 2.700E-02

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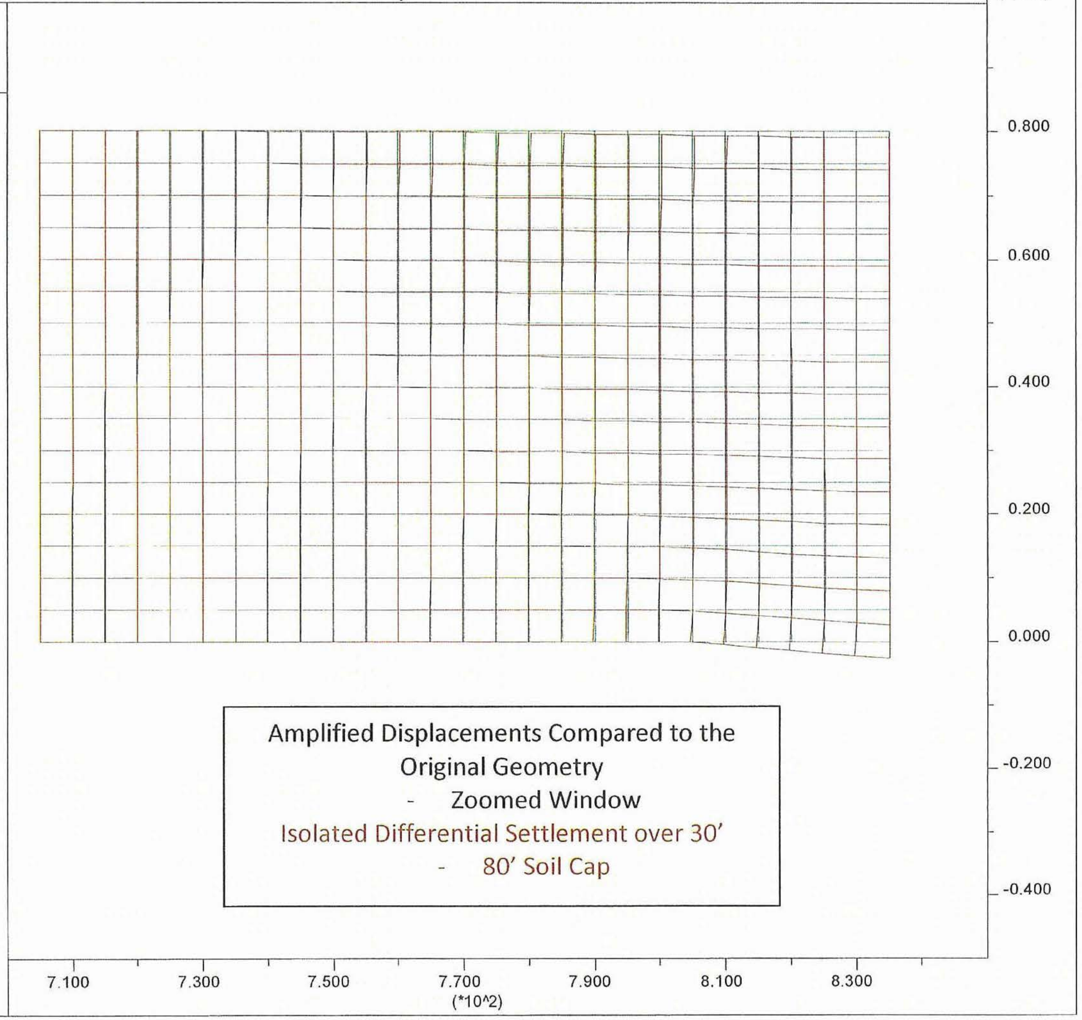

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:21
step 22384

HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 1)
5 Y displacement(162, 1)
X-axis :
Number of steps

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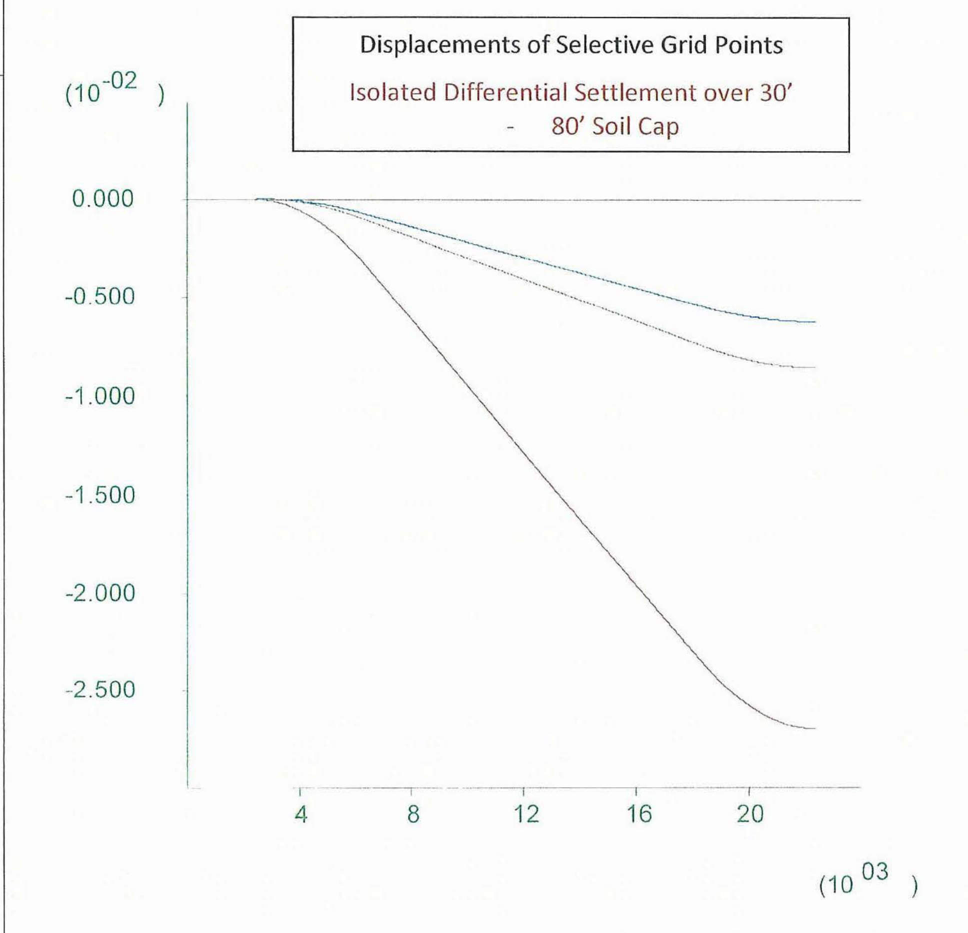
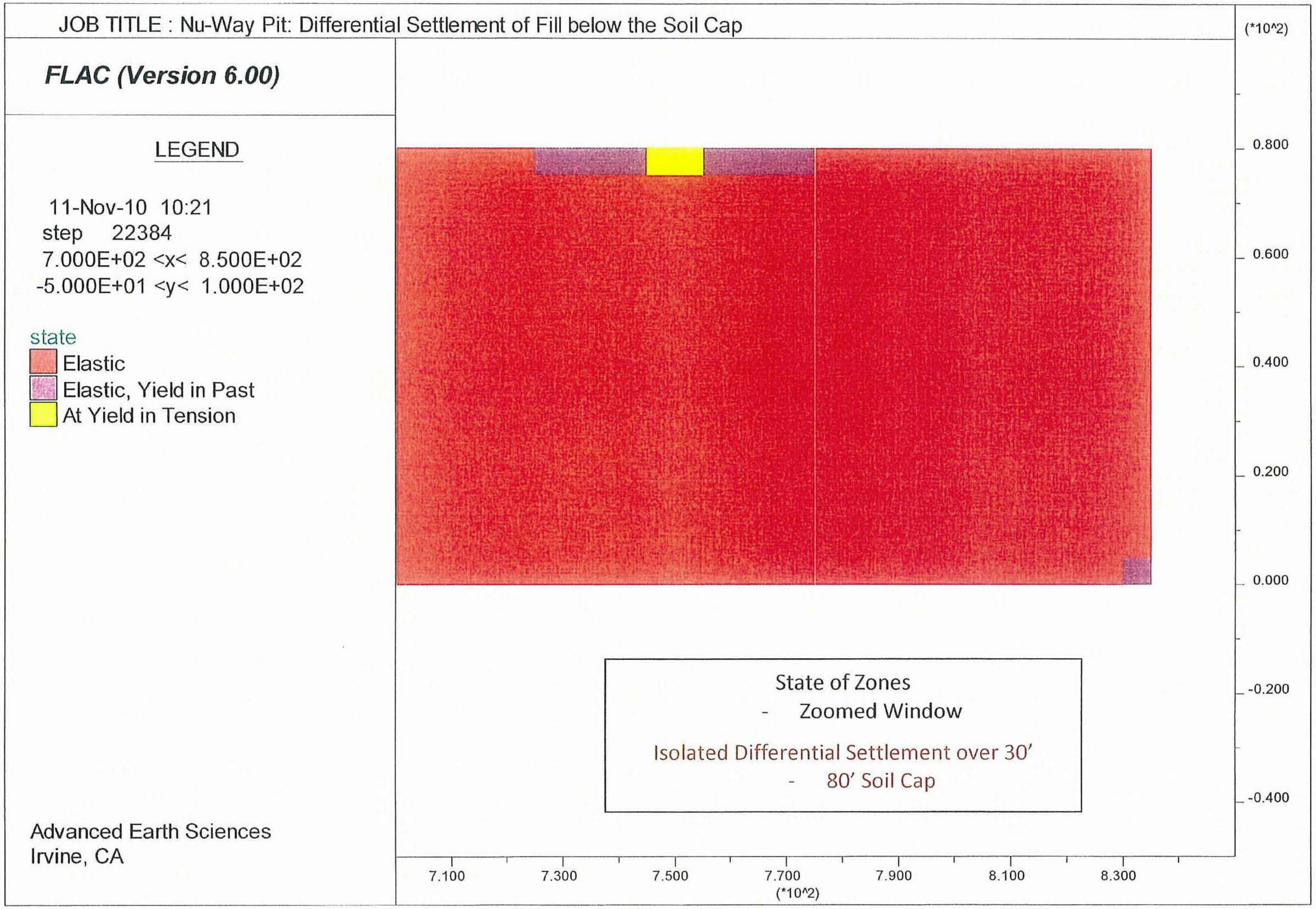


0916.0260



FLAC-32

0916.0261

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:21
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps

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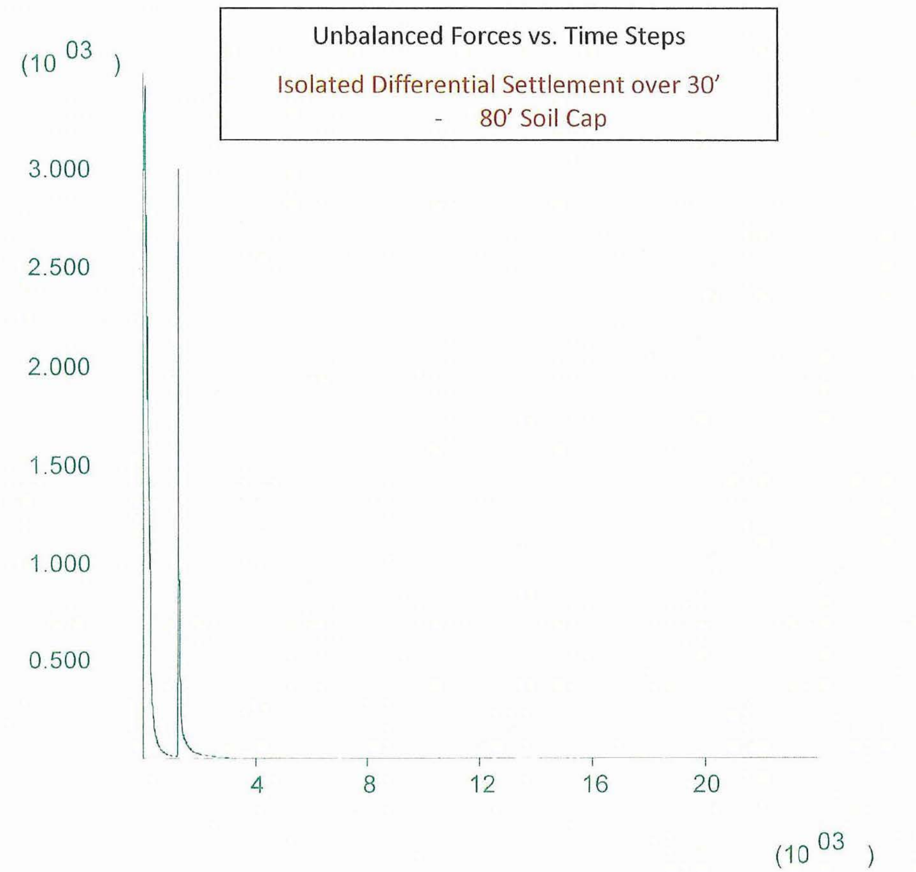

4.2 Randomly Varying Settlement

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:07
step 22384
-4.375E+01 <x< 8.812E+02
-4.037E+02 <y< 5.213E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 7.788E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 7.788E-01

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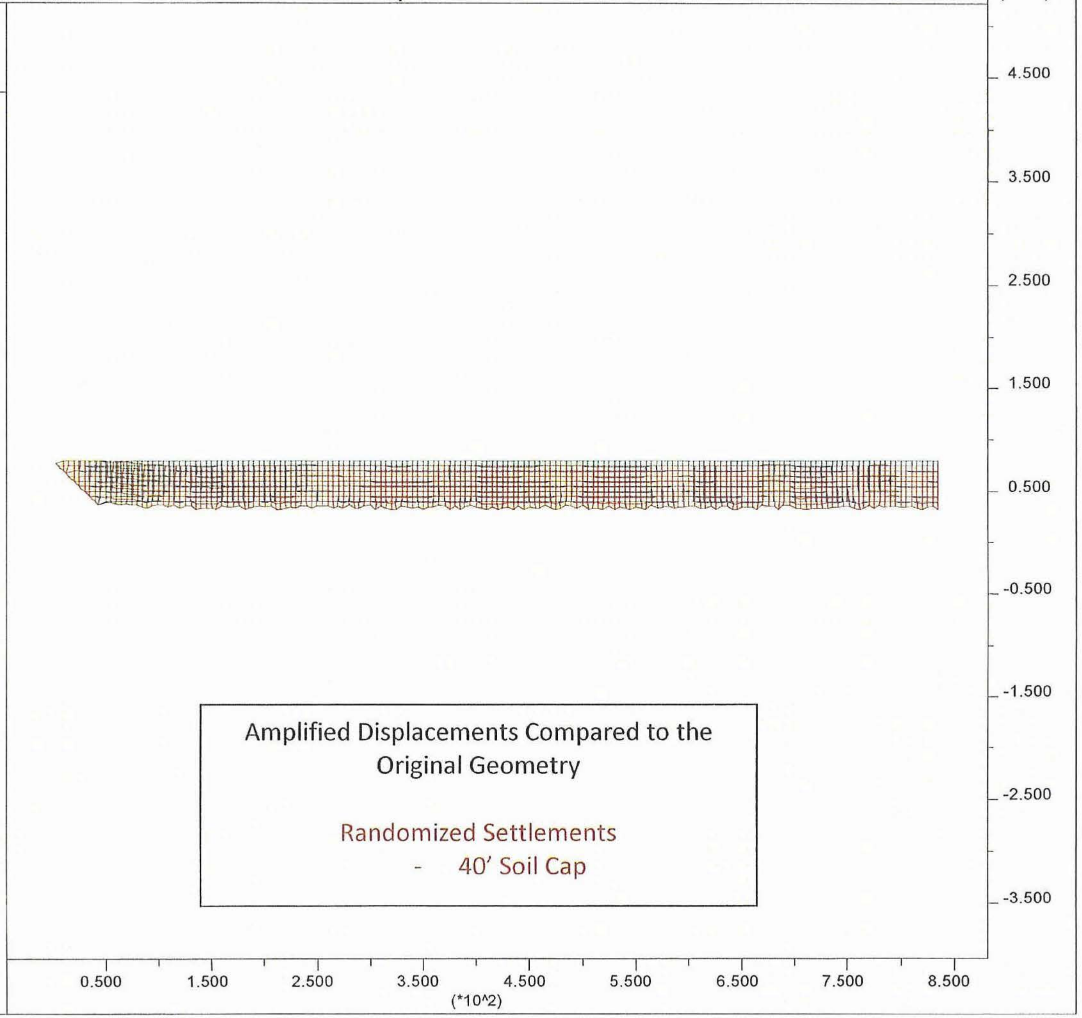

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:07
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 7.596E-01

Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 7.596E-01

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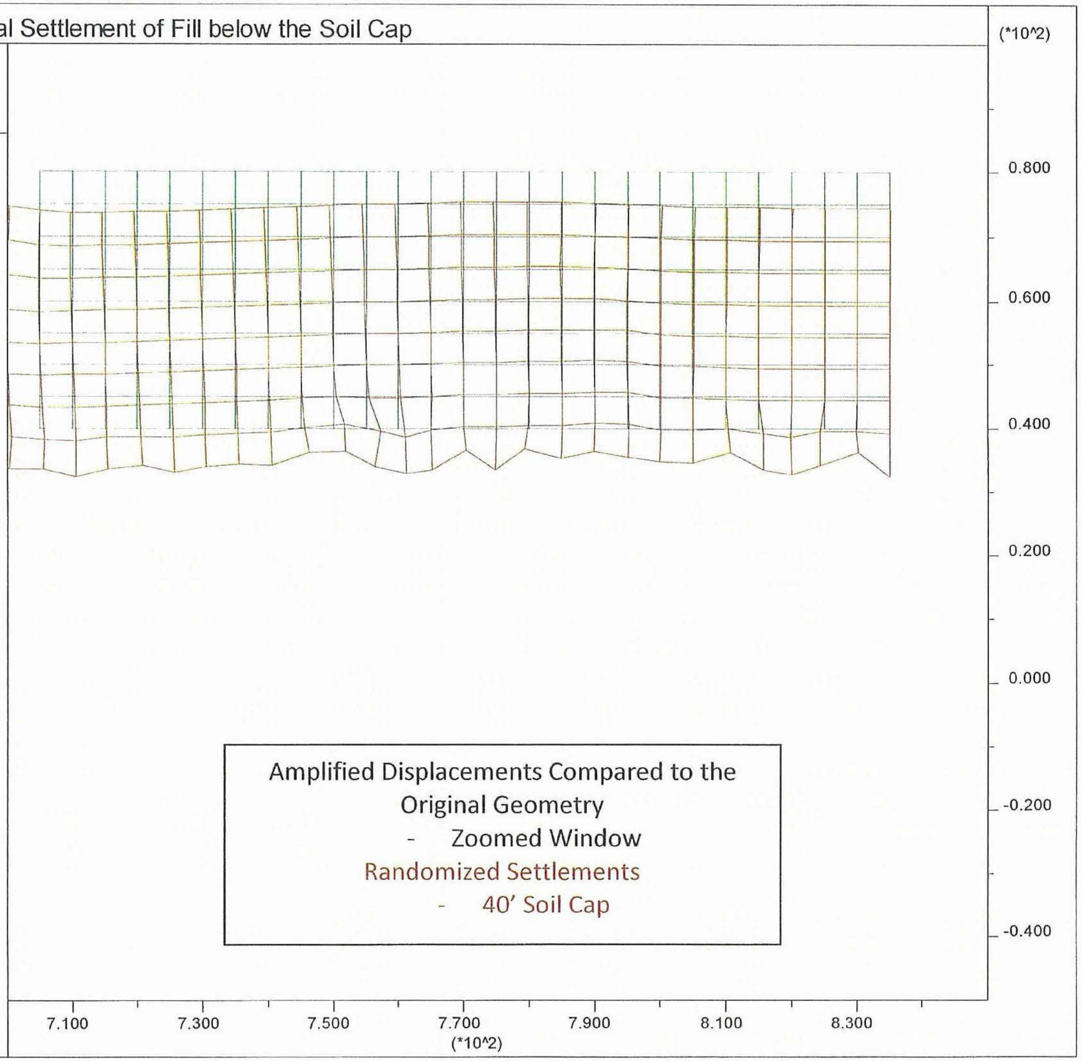
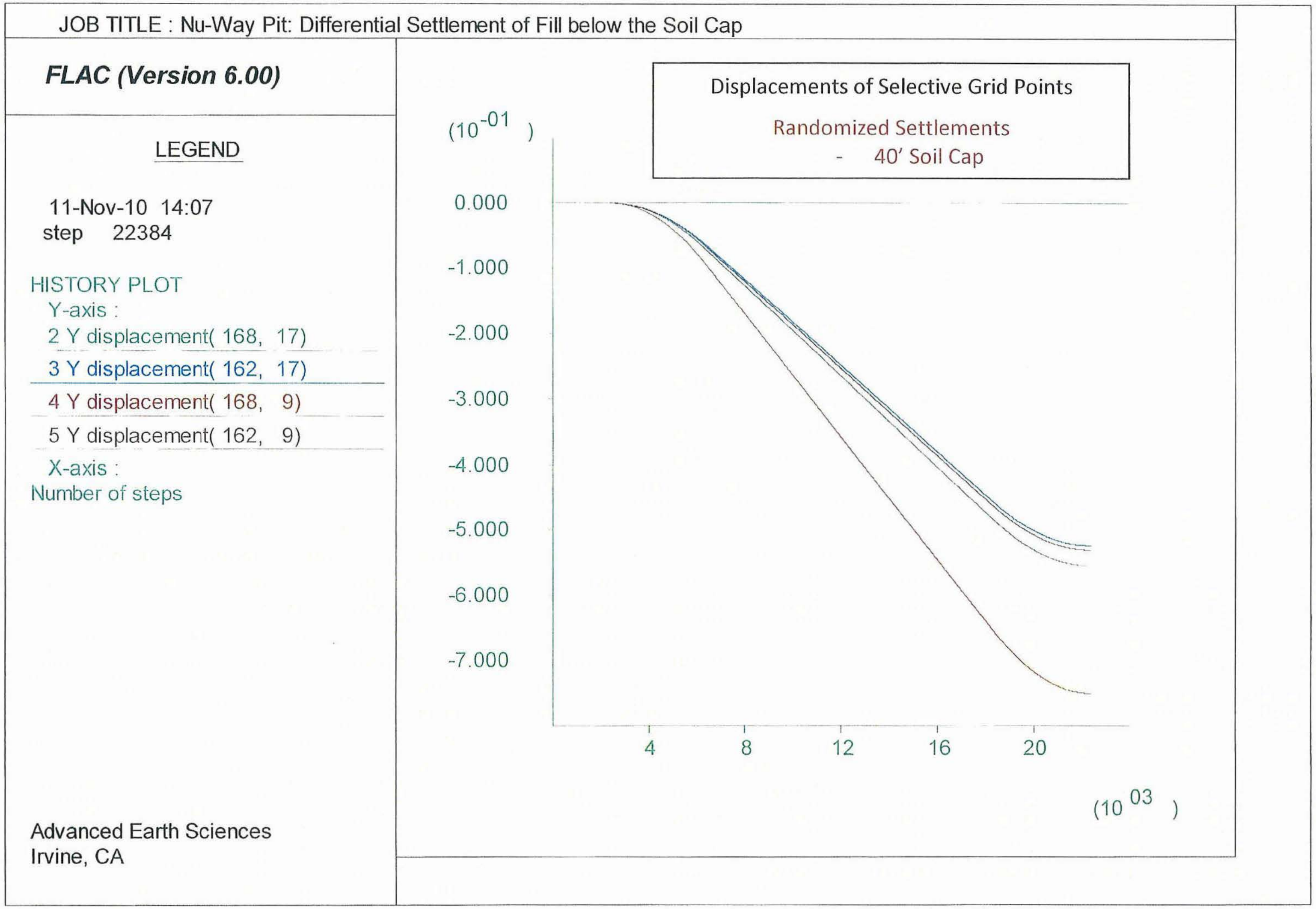
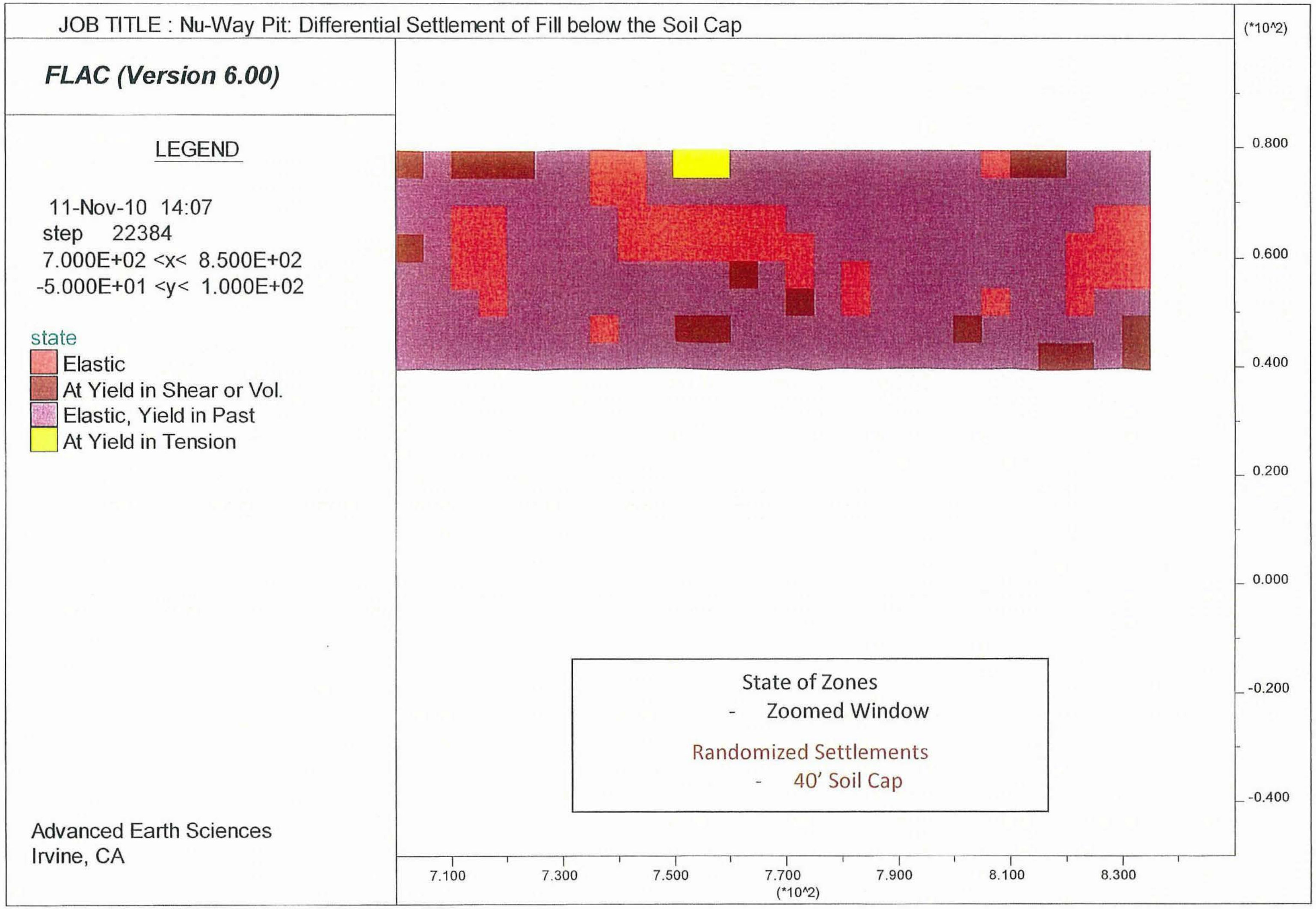


FLAC-35

0916.0264


JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 14:07
step 22384
HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 9)
5 Y displacement(162, 9)
X-axis :
Number of steps
Advanced Earth Sciences
Irvine, CA
Displacements of Selective Grid Points
Randomized Settlements
- 40' Soil Cap
(10-01)
0.000
-1.000
-2.000
-3.000
-4.000
-5.000
-6.000
-7.000
4
8
12
16
20
(10 03)



FLAC-37

0916.0266

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:07
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps

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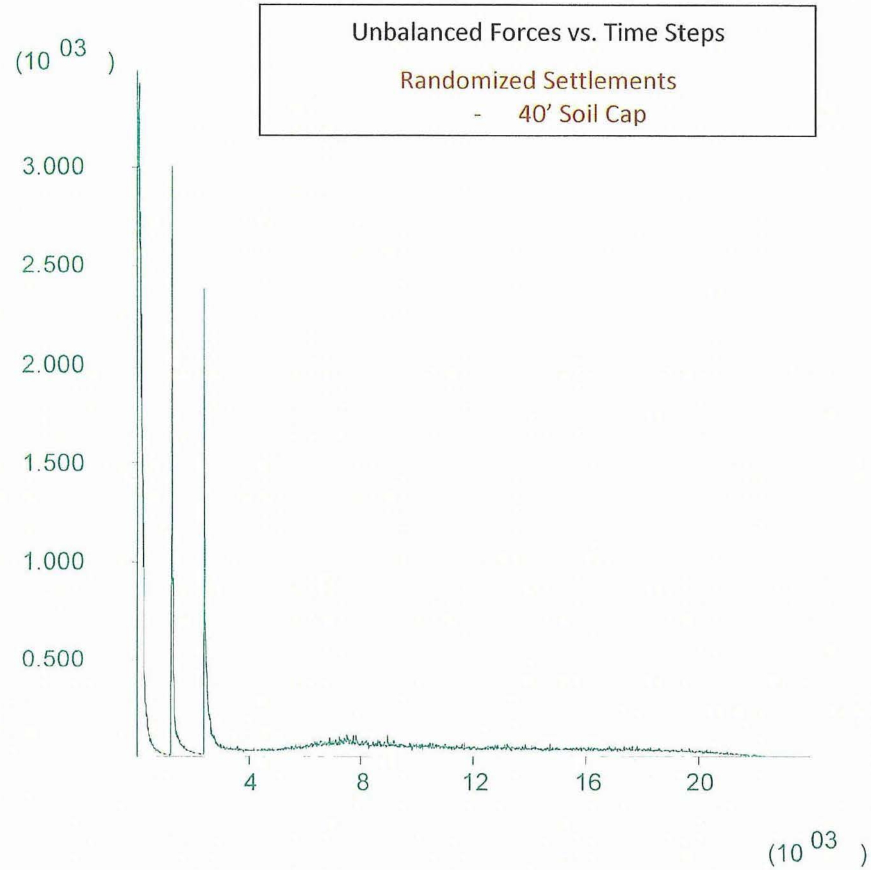
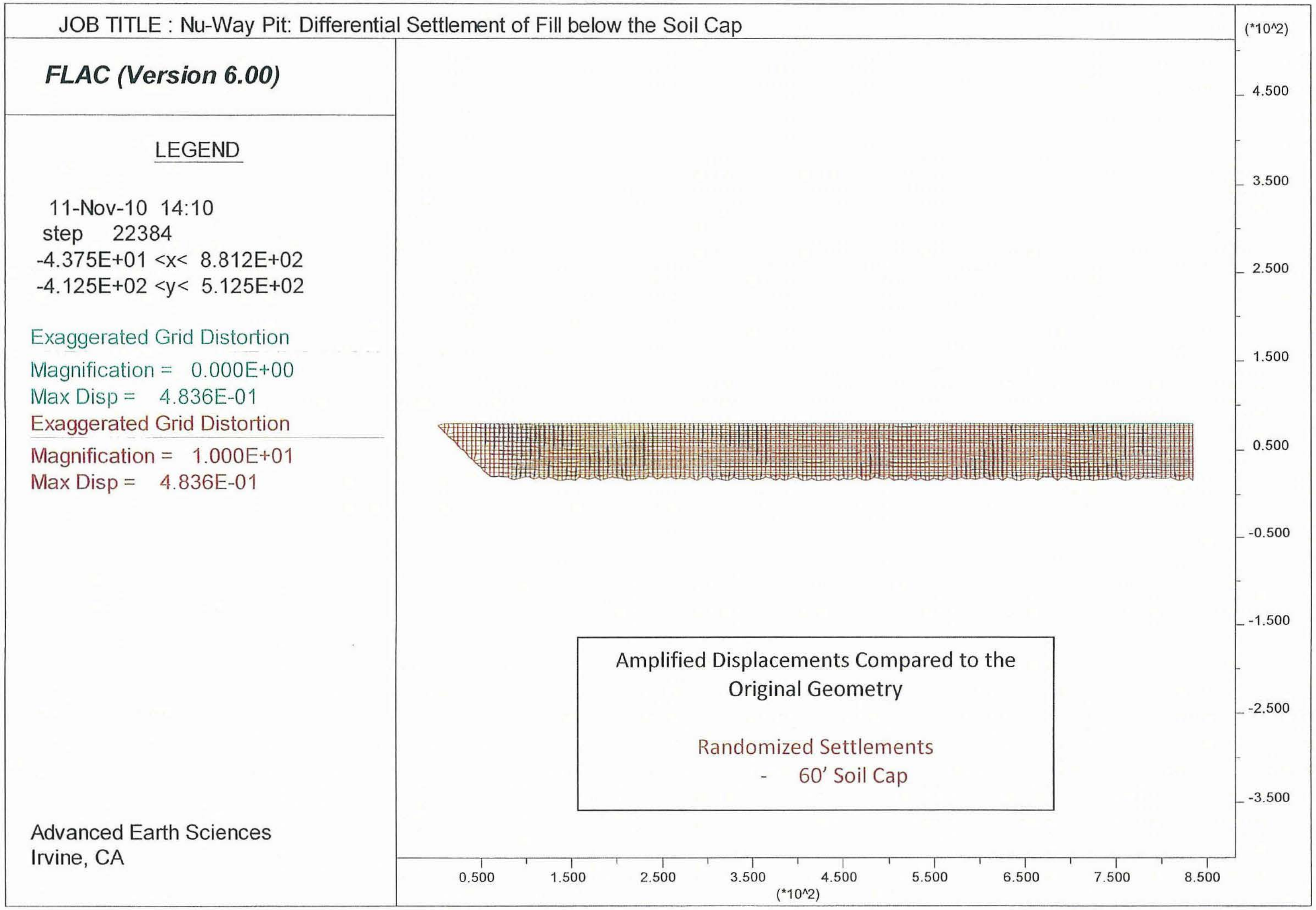
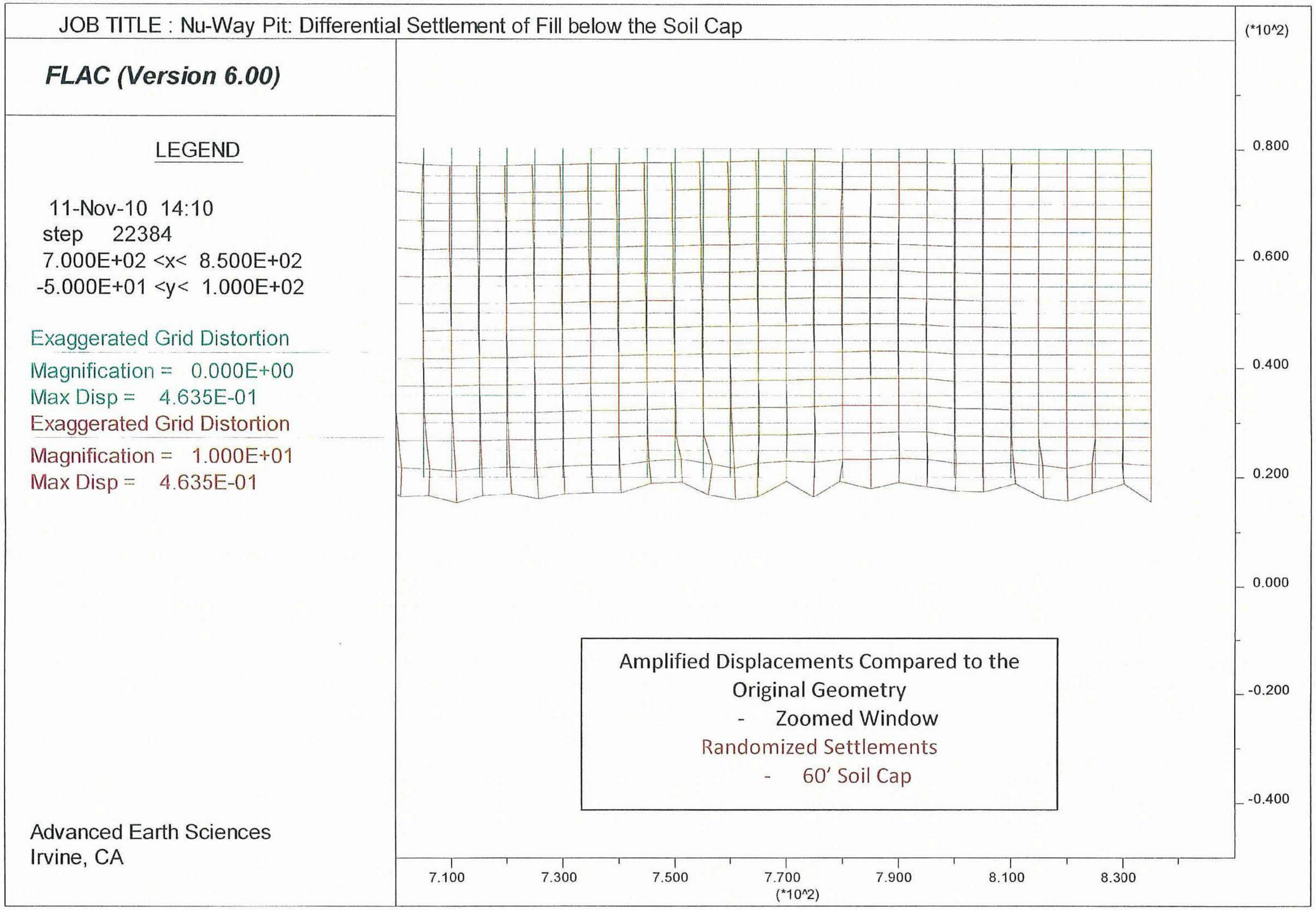



JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 14:10
step 22384
-4.375E+01 <x< 8.812E+02
-4.125E+02 <y< 5.125E+02
Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 4.836E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 4.836E-01
Advanced Earth Sciences
Irvine, CA
Amplified Displacements Compared to the Original Geometry
Randomized Settlements
- 60' Soil Cap
(*10^2)
4.500
3.500
2.500
1.500
0.500
-0.500
-1.500
-2.500
-3.500
0.500
1.500
2.500
3.500
4.500
5.500
6.500
7.500
8.500
(*10^2)

0916.0268



FLAC-40

0916.0269

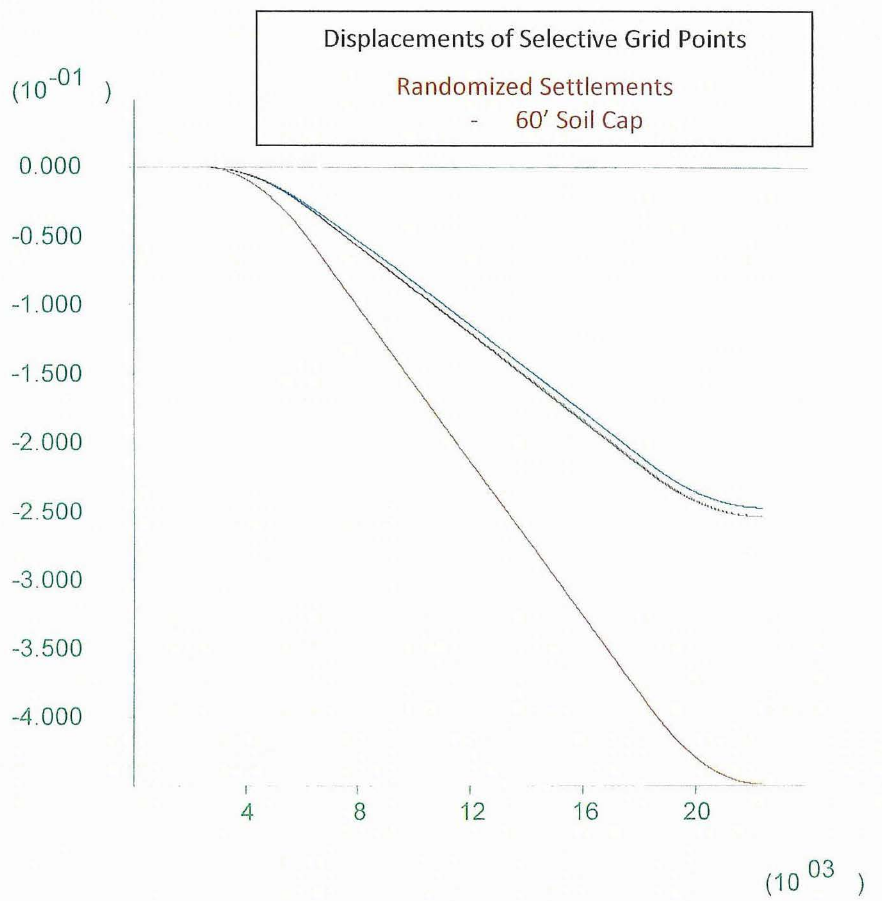
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:10
step 22384

HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 5)
5 Y displacement(162, 5)
X-axis :
Number of steps



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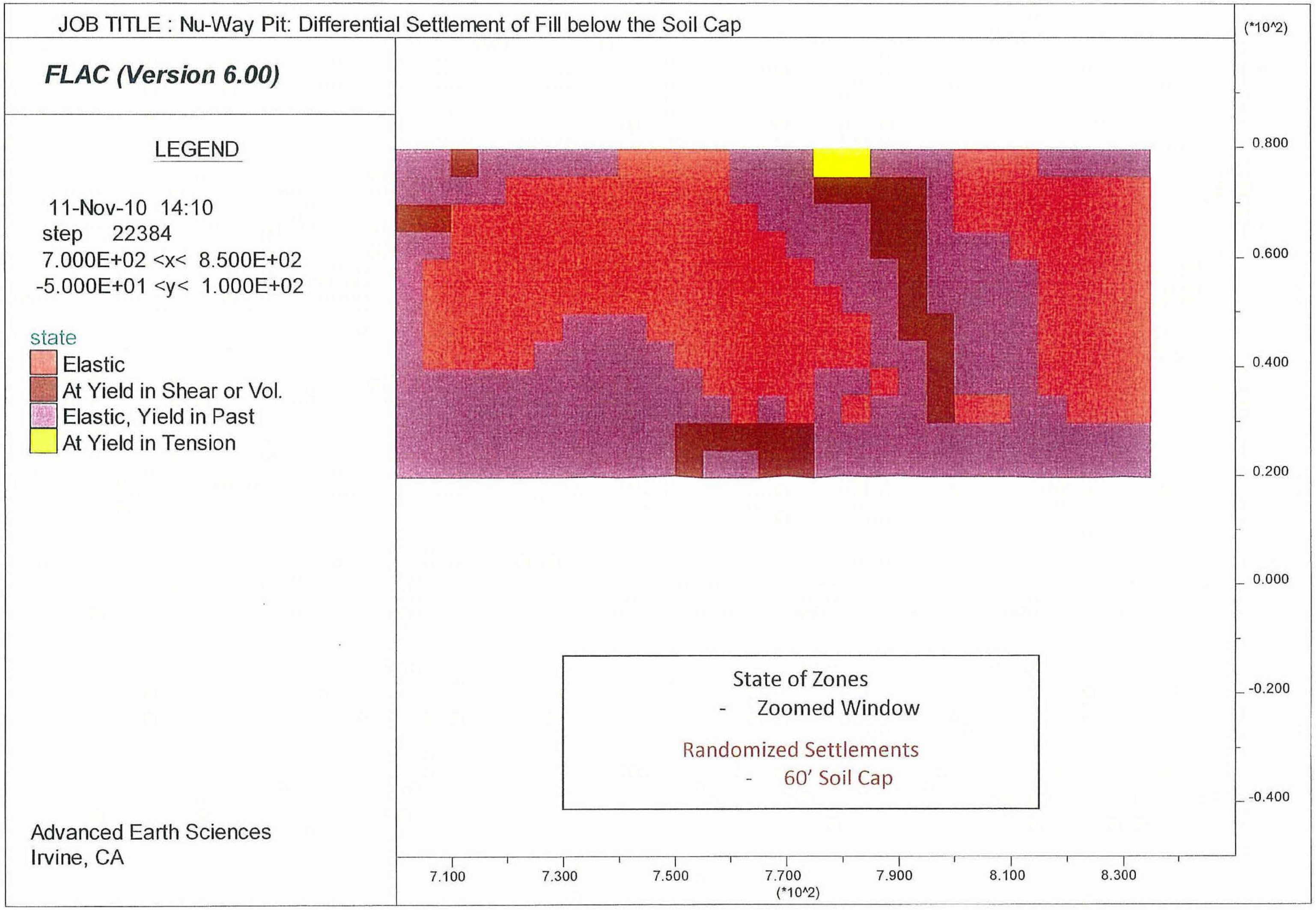
0916.0270



FLAC-42

0916.0271

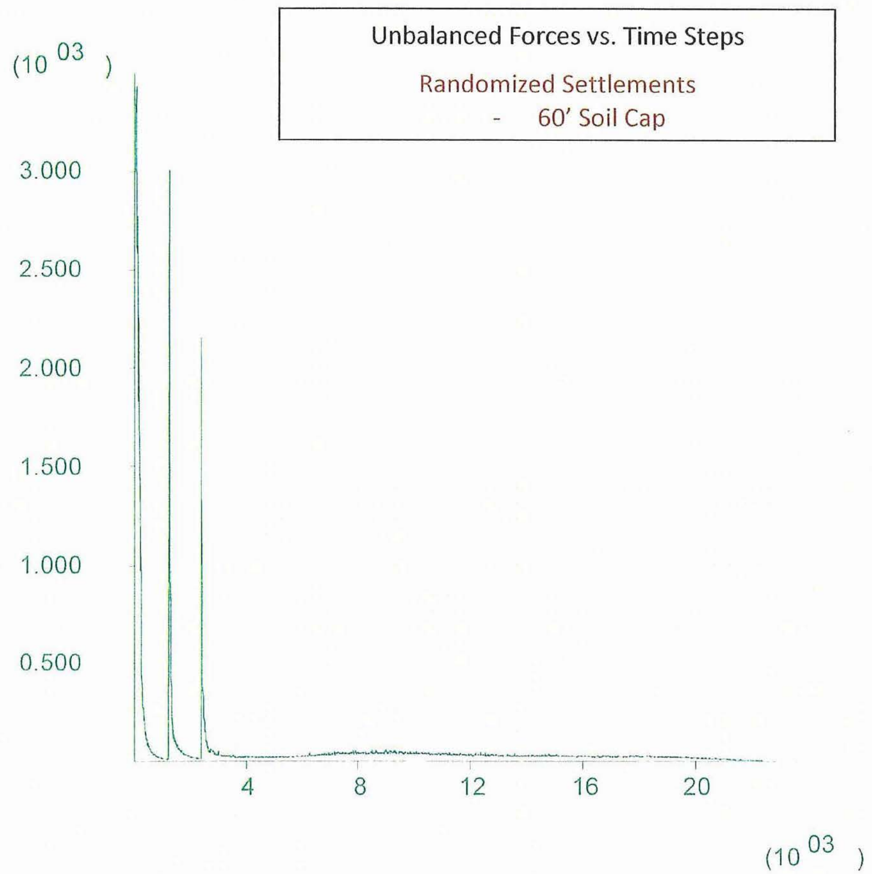
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:10
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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Irvine, CA

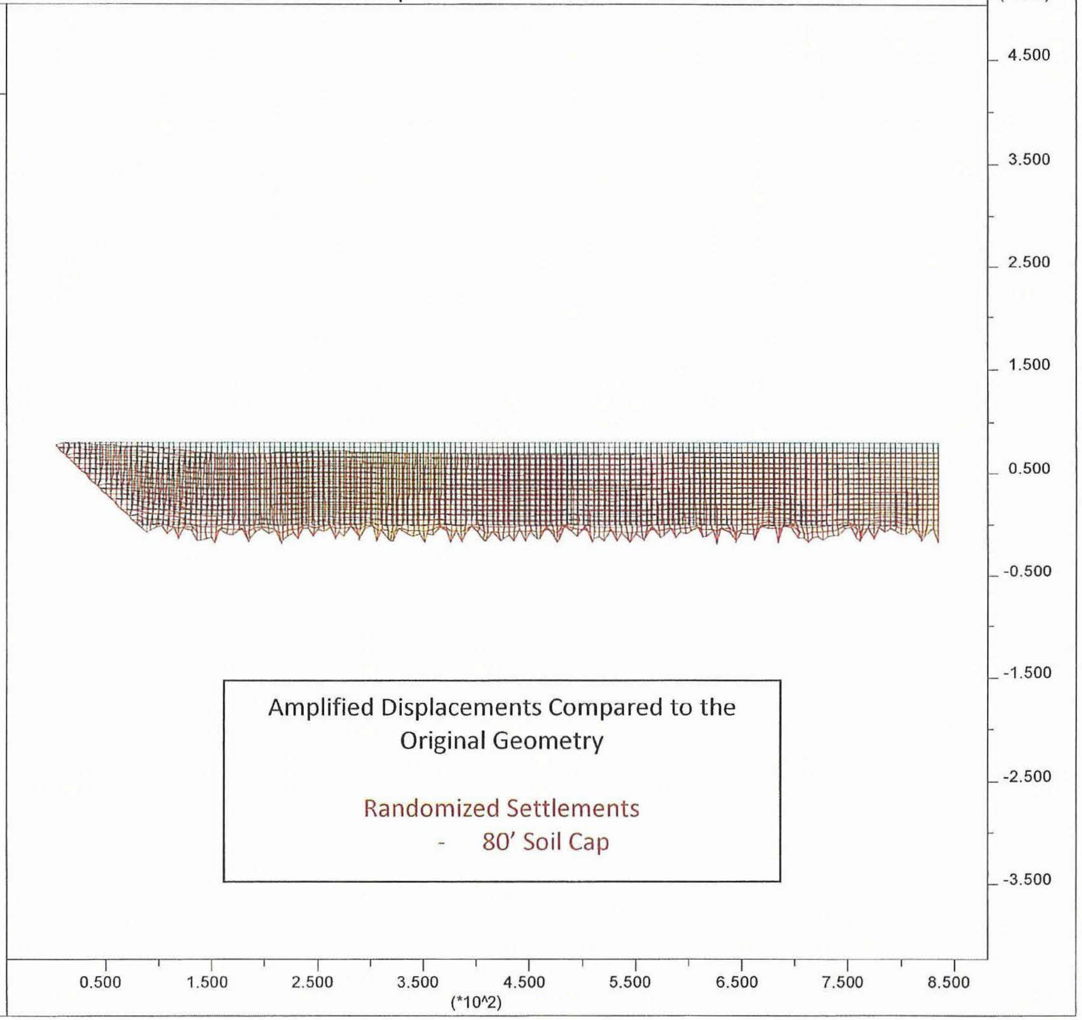
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:12
step 22388
-4.375E+01 <x< 8.812E+02
-4.225E+02 <y< 5.025E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 1.893E-01
Exaggerated Grid Distortion
Magnification = 1.000E+02
Max Disp = 1.893E-01



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Irvine, CA

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:12
step 22388
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 1.758E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 1.758E-01

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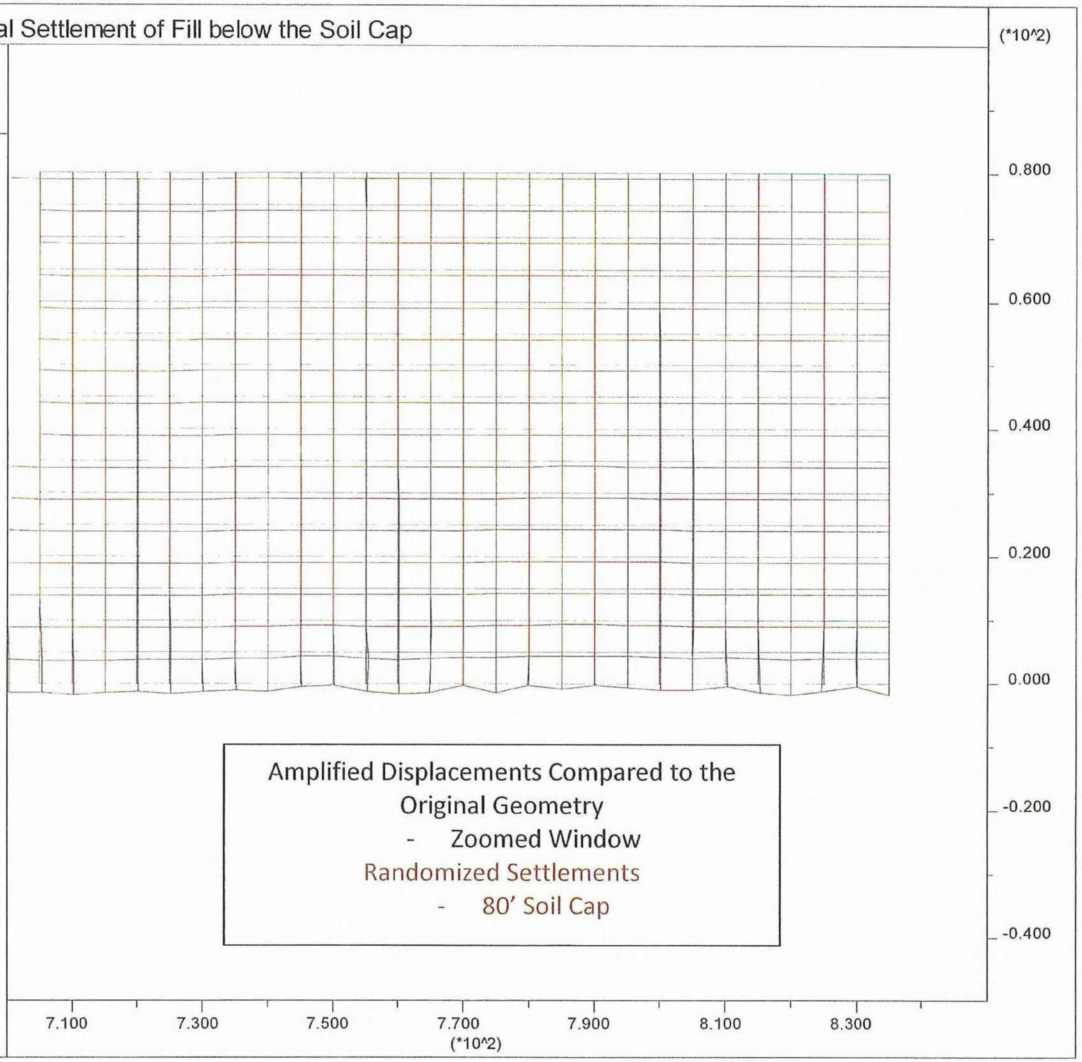

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:12
step 22388

HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 1)
5 Y displacement(162, 1)
X-axis :
Number of steps

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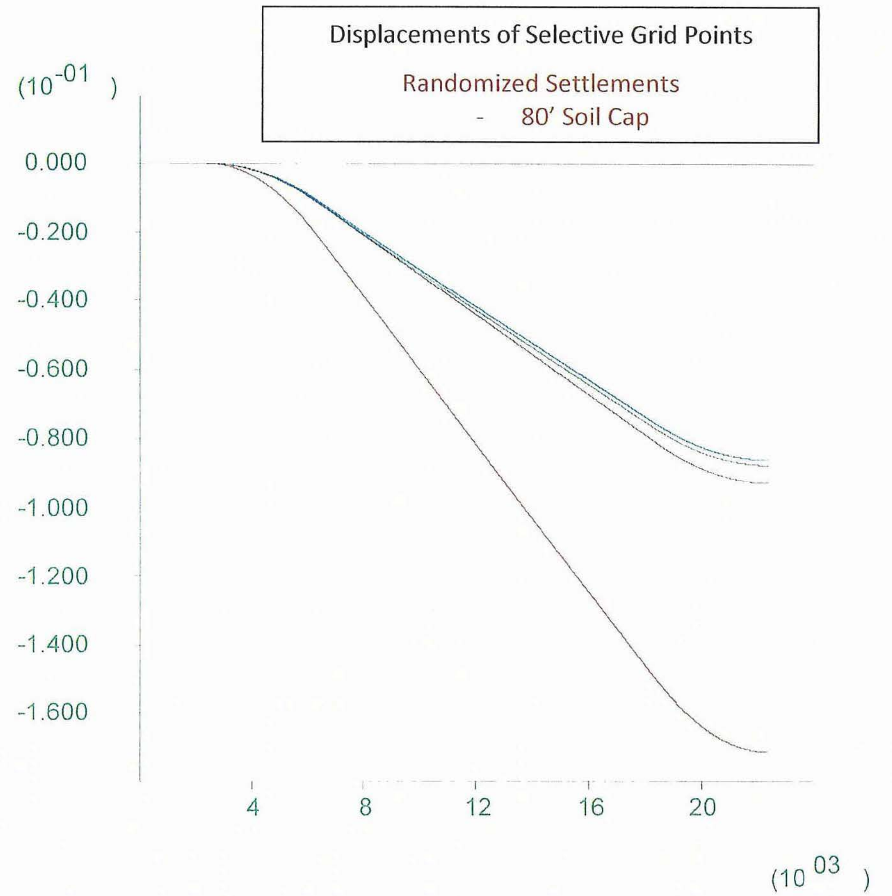


0916.0275

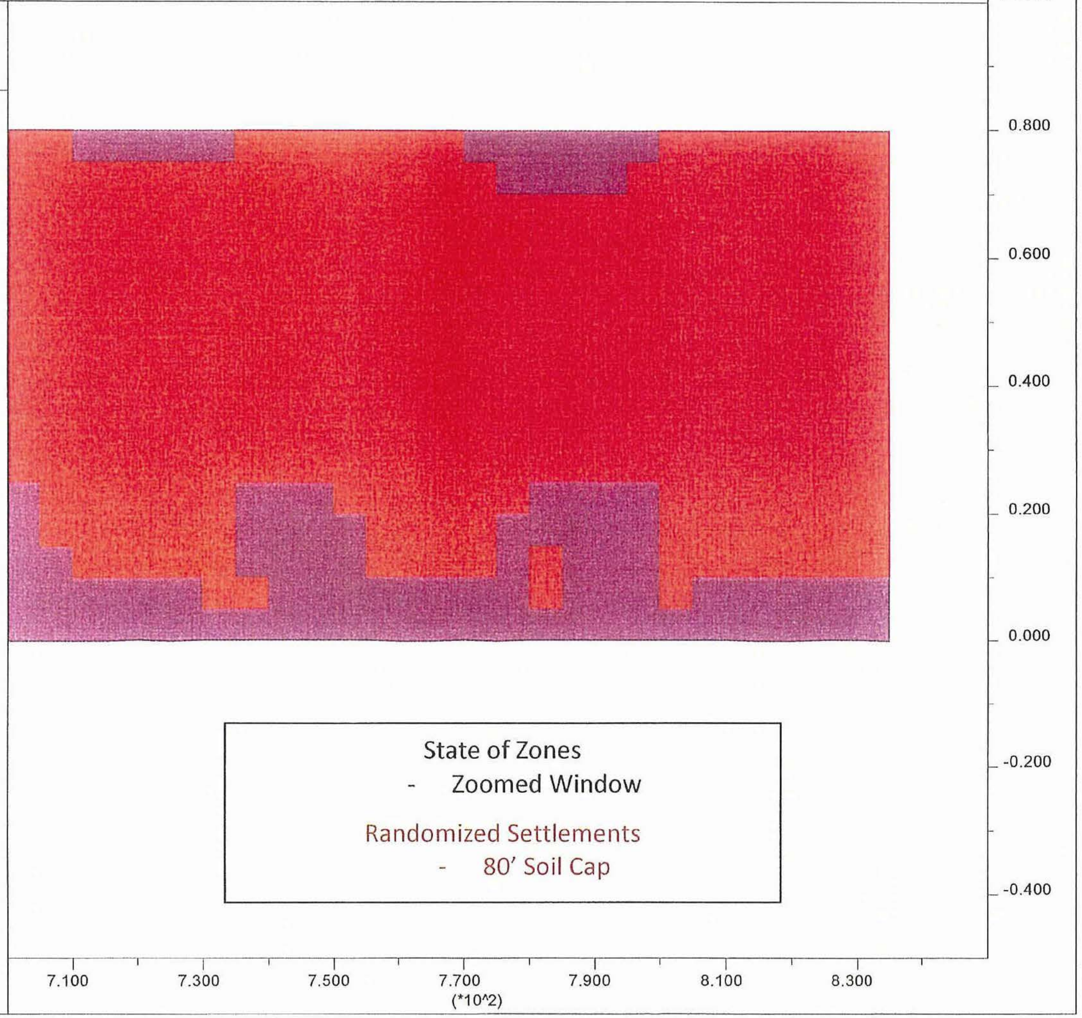
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:12
step 22388
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

state
- Elastic
- At Yield in Shear or Vol.
- Elastic, Yield in Past
- At Yield in Tension



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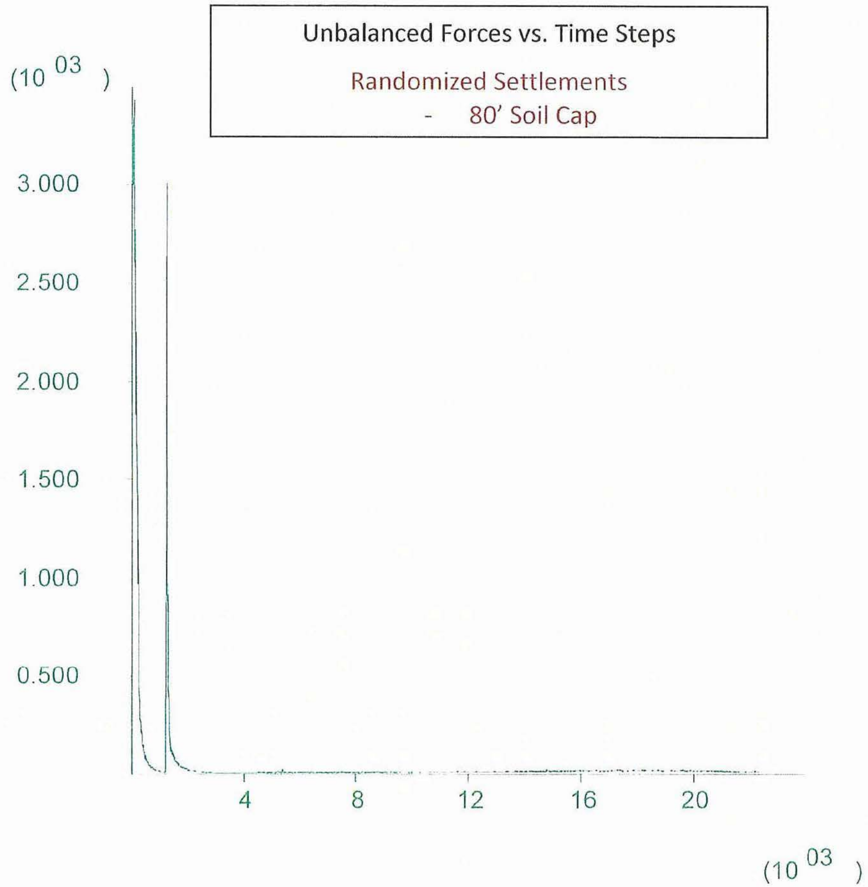
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 14:12
step 22388

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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5. Parametric Studies

5.1 Zone (Mesh) Size and Displacement Rates

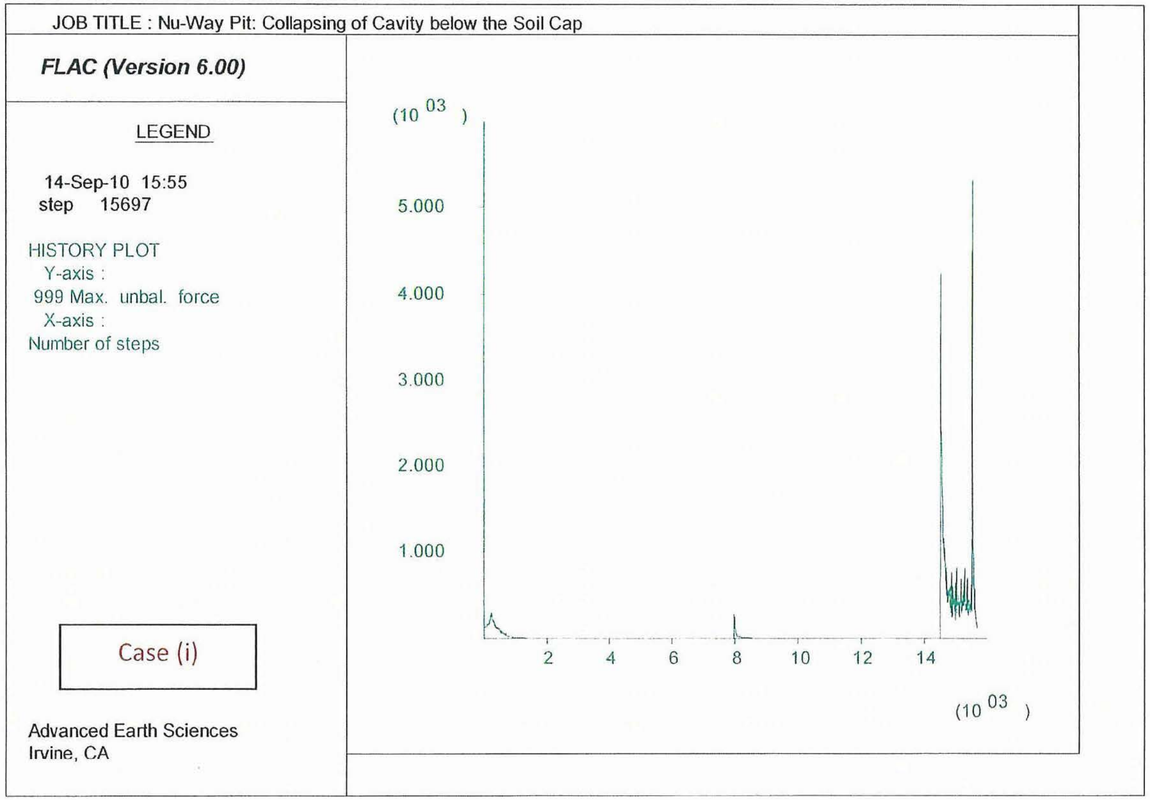
In order to evaluate the influence of zone size and displacement rates the following four sets of analyses were performed:

(i) Smaller zone size (1′ x 1′); Larger displacement rate (2.5×10^{-4} ft/step)
(ii) Larger zone size (5′ x 5′); Larger displacement rate (2.5×10^{-4} ft/step)
(iii) Smaller zone size; Smaller displacement rate (ramping)
(iv) Larger zone size; Smaller displacement rate (ramping)

An average total settlement of three inches was applied over 10 feet of the bottom of 40-foot thick soil cap. Shear modulus wasn't degraded with incremental strains. Amplified nodal (grid point) displacements at the end of simulation and total unbalanced forces during simulation were plotted. The following observations were made:

(i) For the smaller (1′ x 1′) zone size model, the displacement rate of 2.5×10^{-4} ft/step seems too large. It gives a large shock to the system and there is no enough time for the zones to recover. As a result the bottom zones (closer to where the displacements were applied) suddenly start to yield and the zones above try to catch-up. This is evidenced by high unbalanced forces (>>100 lbs) during the application of displacement rate and highly distorted bottom zones (see output plots). As a result, surface manifestation looks negligible. This is purely an artifact and the results are meaningless.
(ii) For the 5′ x 5′ zone size model, even though the unbalanced forces are very high at the beginning, they fall below 200 lbs quickly (within one-quarter of the total time steps used for applying displacement). Due to reduced number of zones (elements), the upper zones quickly catch-up, and as a result we could see the surface manifestation.
(iii) When the user-defined ramping function was used for the smaller zone size model, the unbalanced forces are kept very low (<100 lbs) and there is enough time for the upper zones to catch up. For this analysis, a total of 20,000 time steps were used for applying the same displacement as compared to 1,000 steps above. During the first 4,000 steps, the displacement rate was gradually increased from zero to 1.56×10^{-5} ft/step, and kept at this value for the next 12,000 steps before gradually decreasing to zero over the last 4,000 steps. This way, the first increment is in the order of 10^{-9} ft/step, thus minimizing the shock to the system. Surface manifestations can be accurately quantified using this approach.
(iv) For the larger zone size model, even the higher displacement rates (case ii above) gives comparable results to more rigorous analysis such as the Case iii above. However, using the ramping function gives added confidence in the results as the unbalanced forces are kept <100 lbs throughout the application of displacement rate.

The following plots show the results from the above analyses:



Explanation:

1st Peak: Application of gravity (assume linear elastic model)

2nd Peak: Application of actual material properties

3rd Peak: Start of displacement

4th Peak: End of displacement

Note the very high unbalanced forces

FLAC-50

0916.0279

JOB TITLE : Nu-Way Pit: Collapsing of Cavity below the Soil Cap

FLAC (Version 6.00)

LEGEND

14-Sep-10 15:55
step 15697
3.500E+02 <x< 4.500E+02
-5.000E+01 <y< 5.000E+01

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.500E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 2.500E-01

Case (i)

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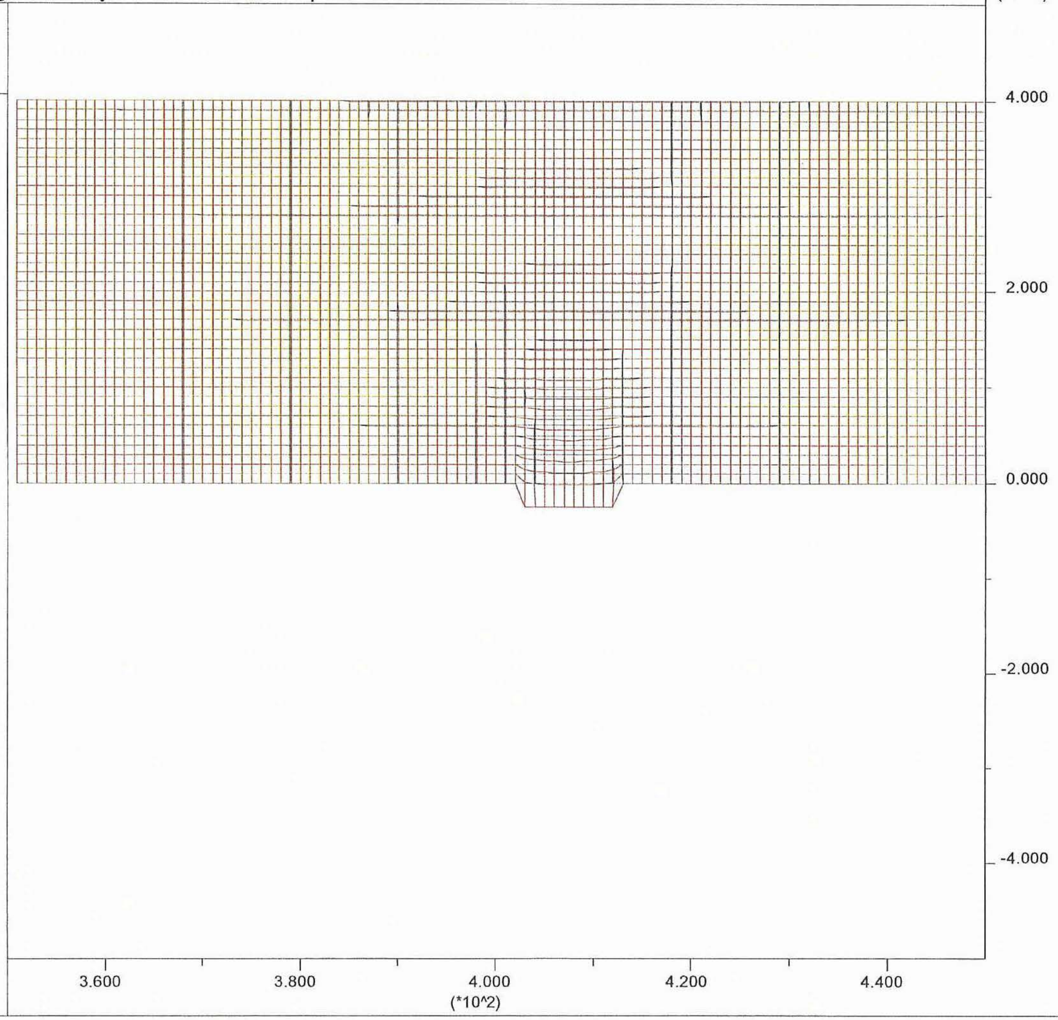
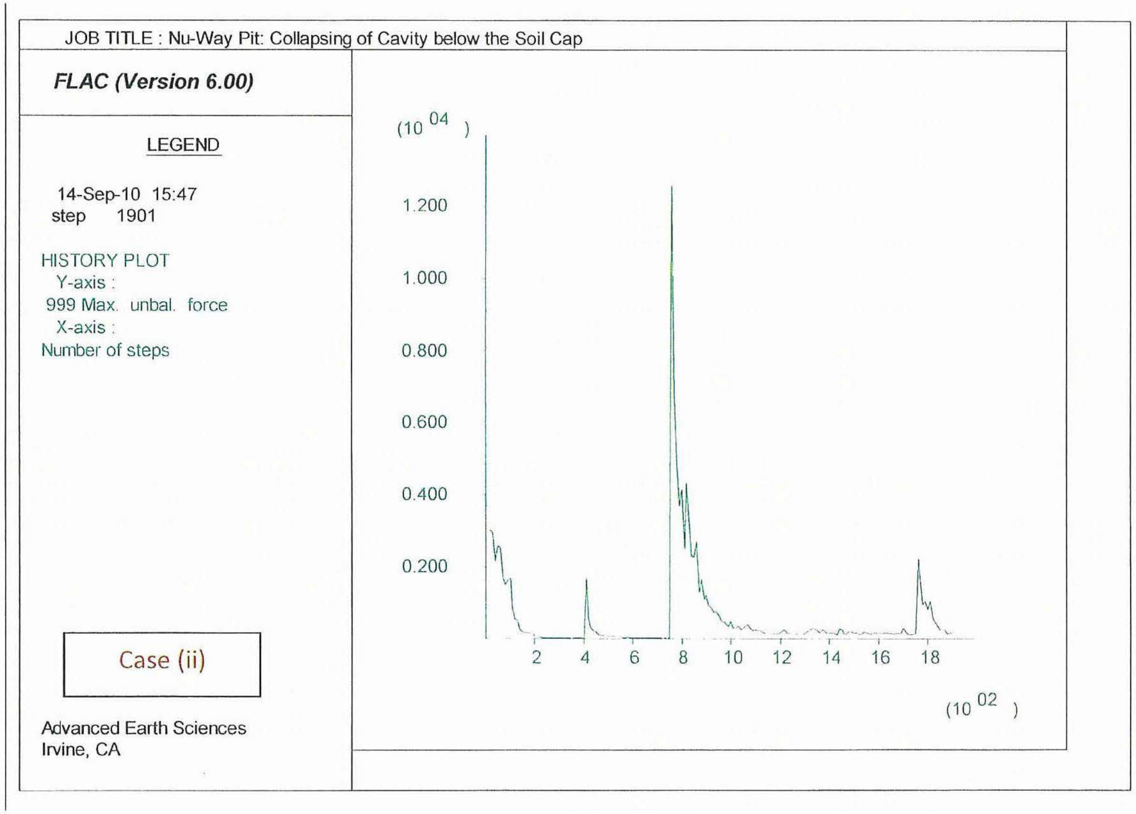


FLAC-51

0916.0280



Explanation:

1st Peak: Application of gravity (assume linear elastic model)

2nd Peak: Application of actual material properties

3rd Peak: Start of displacement

4th Peak: End of displacement

JOB TITLE : Nu-Way Pit: Collapsing of Cavity below the Soil Cap

FLAC (Version 6.00)

LEGEND

14-Sep-10 15:47
step 1901
3.500E+02 <x< 4.500E+02
-5.000E+01 <y< 5.000E+01

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.505E-01

Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 2.505E-01

Case (ii)

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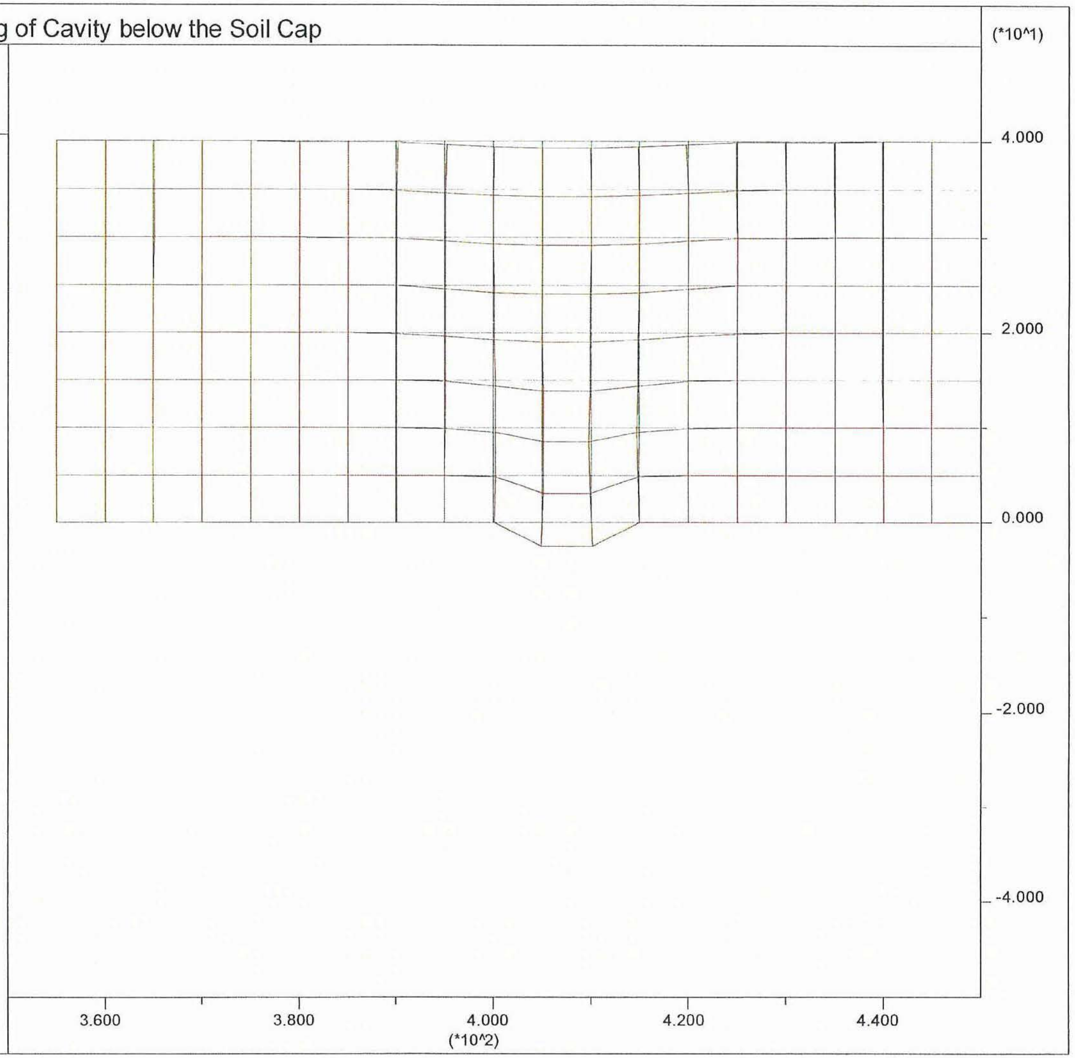
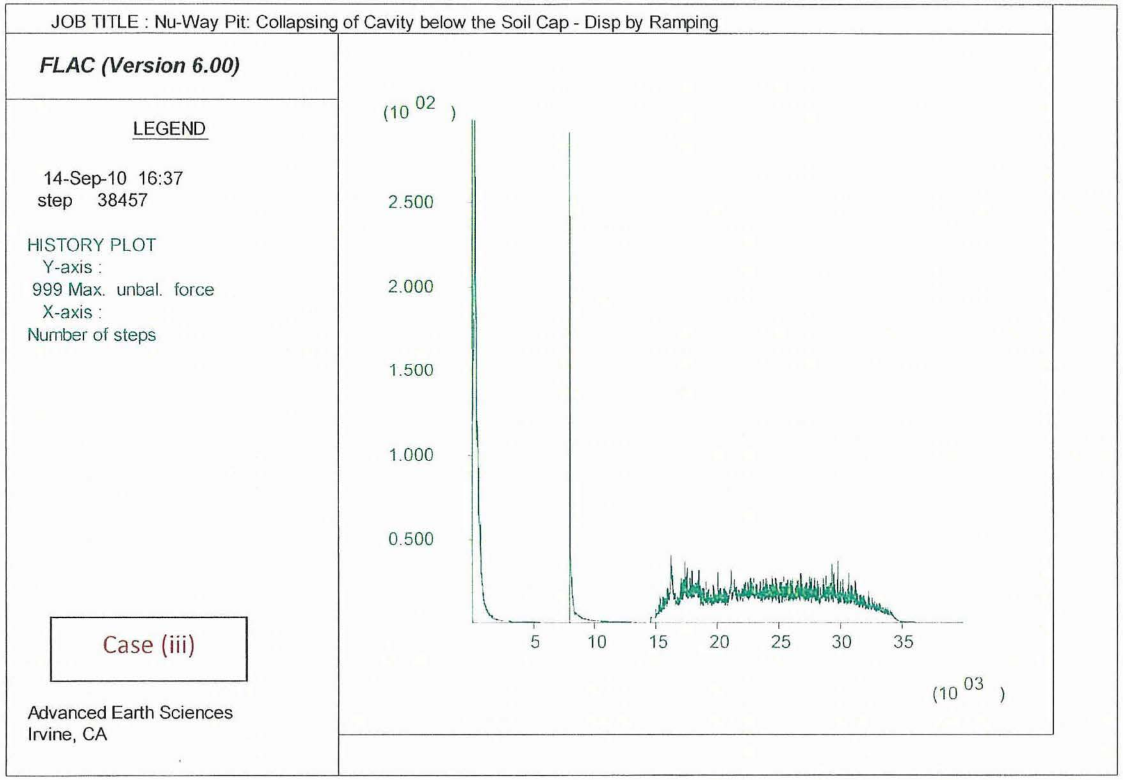


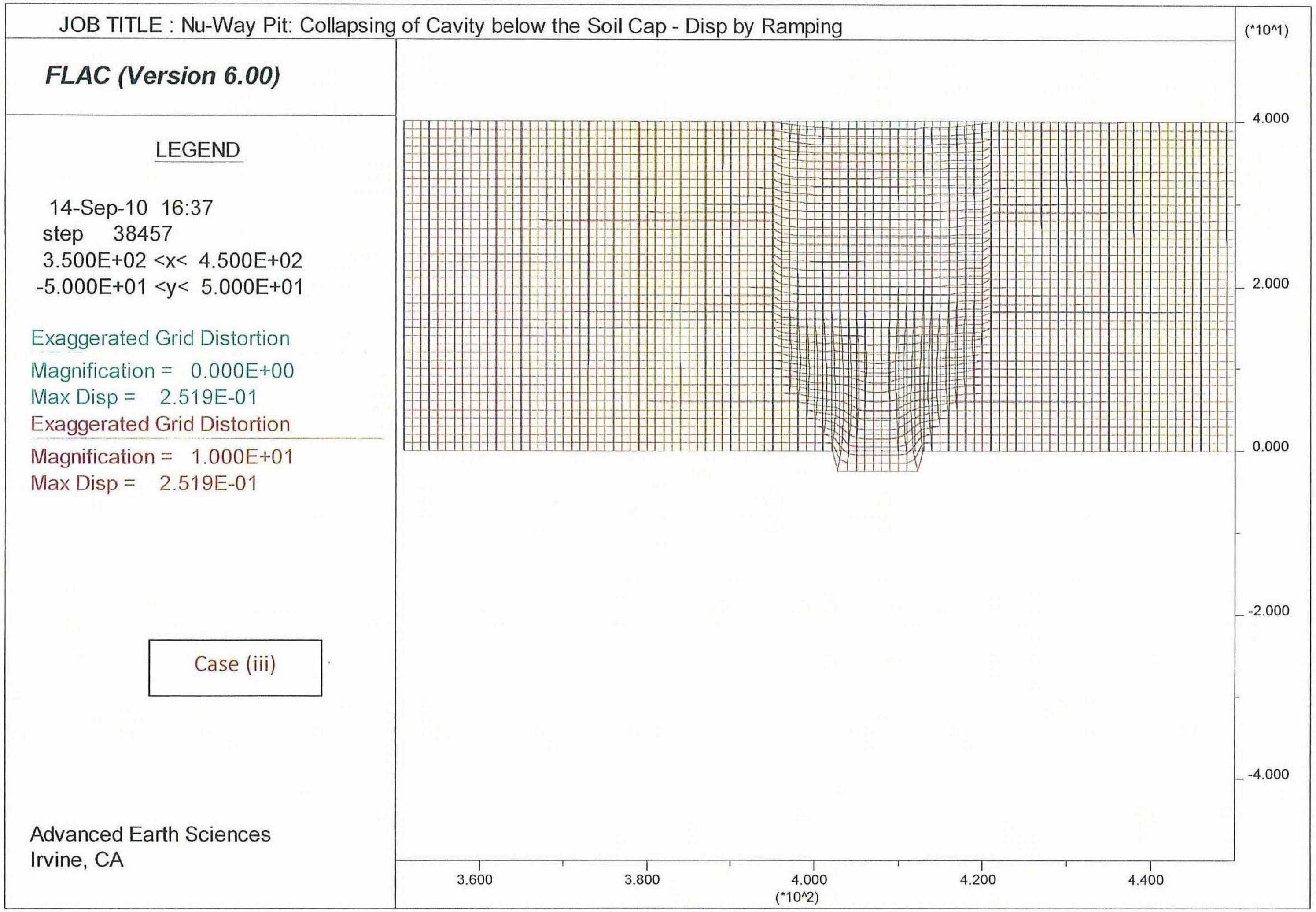


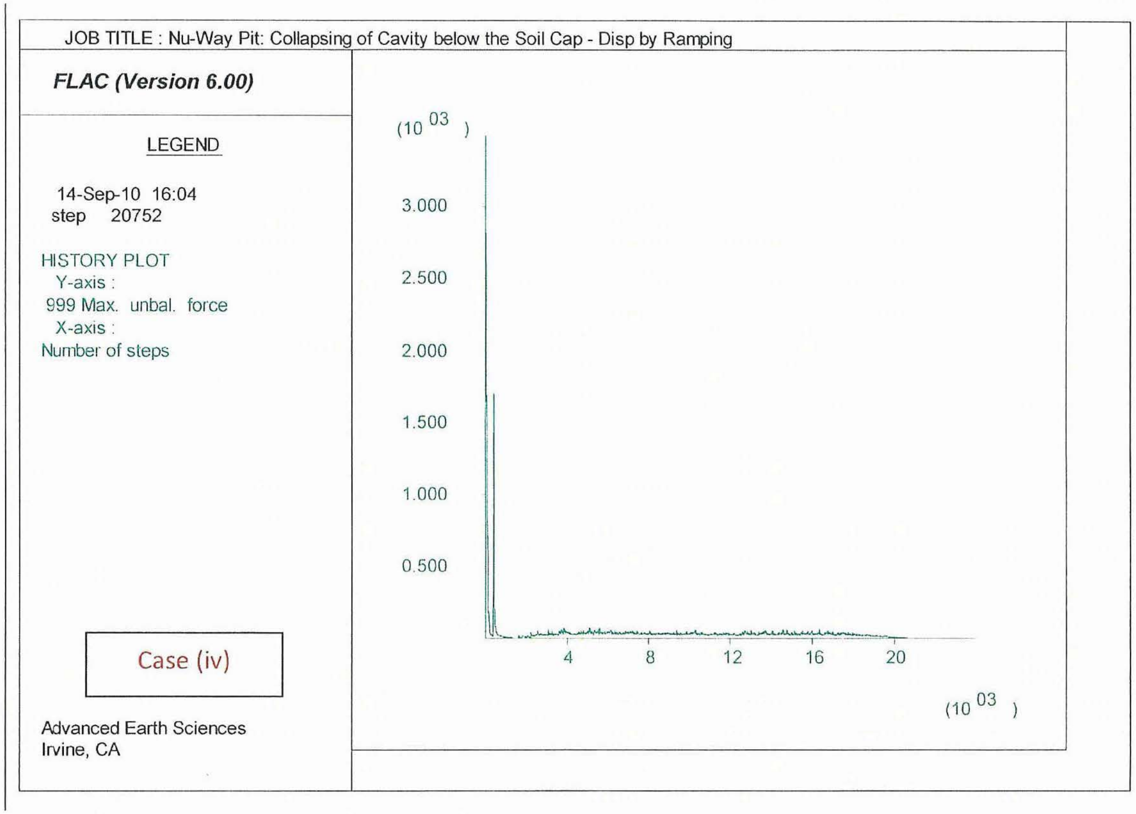
Explanation:

1st Peak: Application of gravity (assume linear elastic model)

2nd Peak: Application of actual material properties

Smaller Spikes: Application of displacements (note the very low unbalanced forces)


JOB TITLE : Nu-Way Pit: Collapsing of Cavity below the Soil Cap - Disp by Ramping
FLAC (Version 6.00)
LEGEND
14-Sep-10 16:37
step 38457
3.500E+02 <x< 4.500E+02
-5.000E+01 <y< 5.000E+01
Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.519E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 2.519E-01
Case (iii)
Advanced Earth Sciences
Irvine, CA
(*10^1)
4.000
2.000
0.000
-2.000
-4.000
3.600
3.800
4.000
(*10^2)
4.200
4.400



Explanation:

1st Peak: Application of gravity (assume linear elastic model)

2nd Peak: Application of actual material properties

Smaller Spikes: Application of displacements (note the very low unbalanced forces)

JOB TITLE : Nu-Way Pit: Collapsing of Cavity below the Soil Cap - Disp by Ramping

FLAC (Version 6.00)

LEGEND

14-Sep-10 16:04
step 20752
3.500E+02 <x< 4.500E+02
-5.000E+01 <y< 5.000E+01

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.506E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 2.506E-01

Case (iv)

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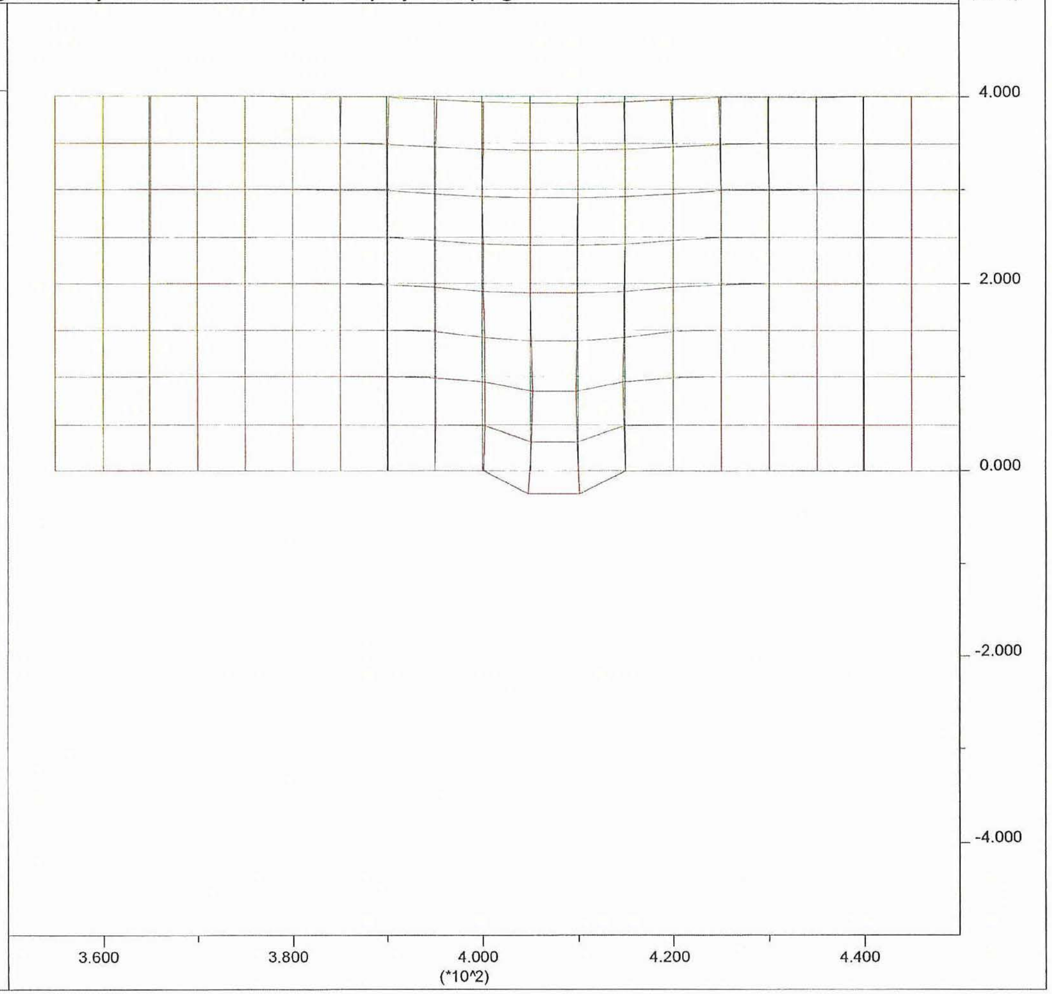
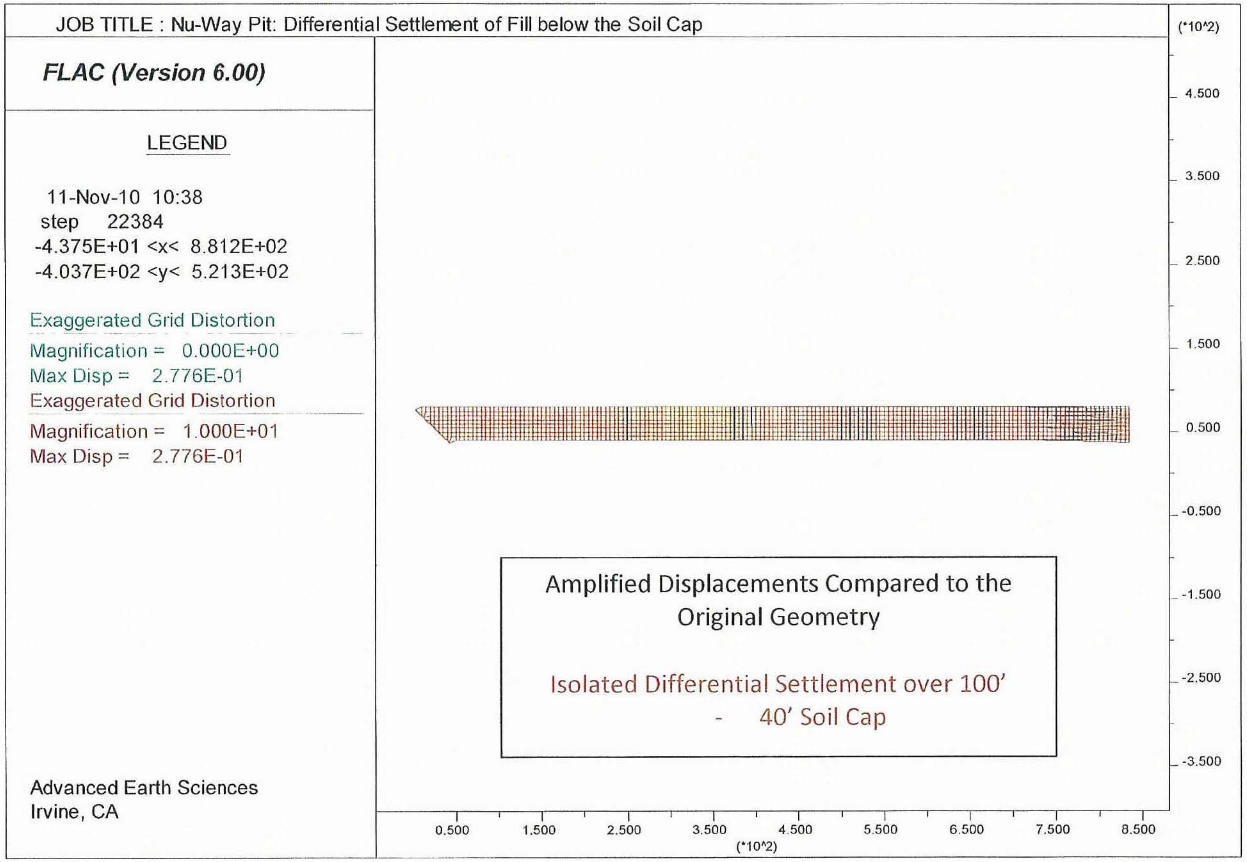
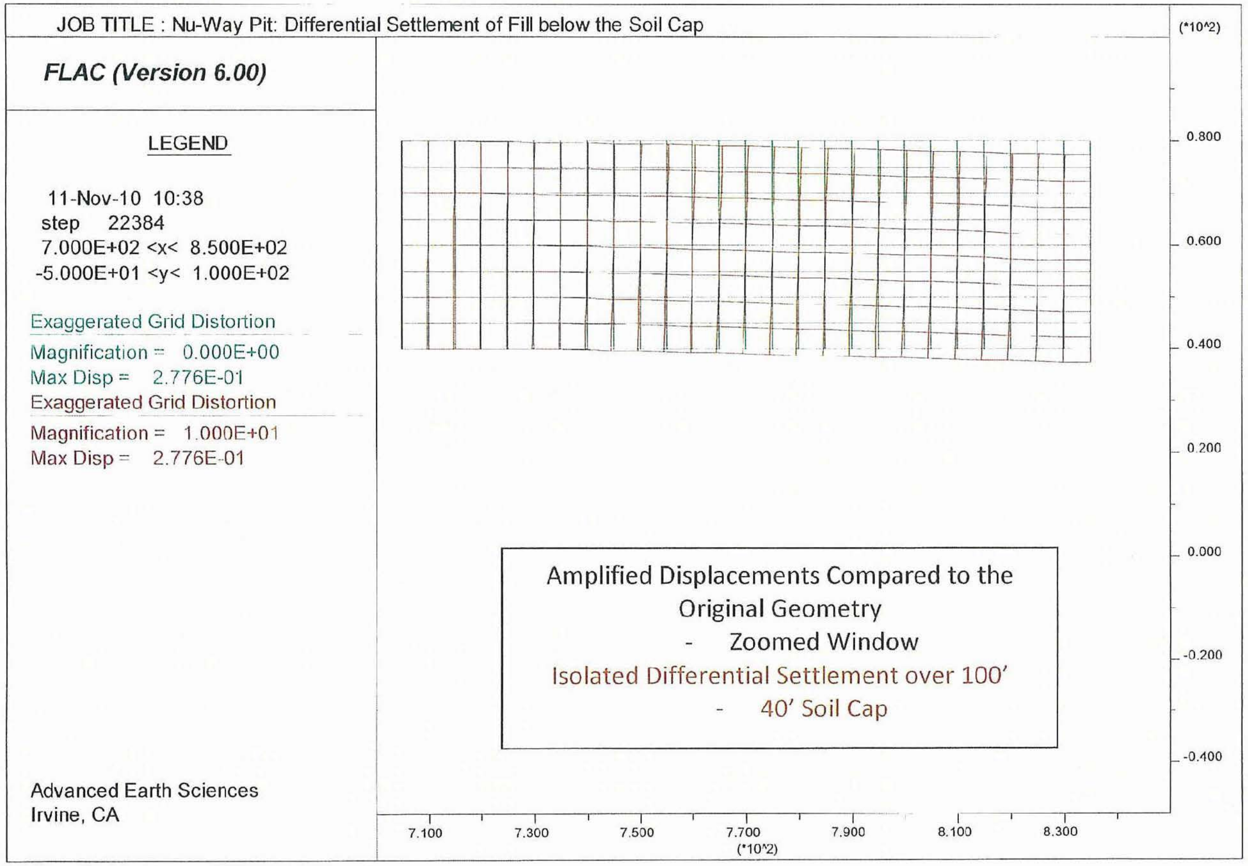


0916.0286

5.2 Isolated Differential Settlement Over 100 ft

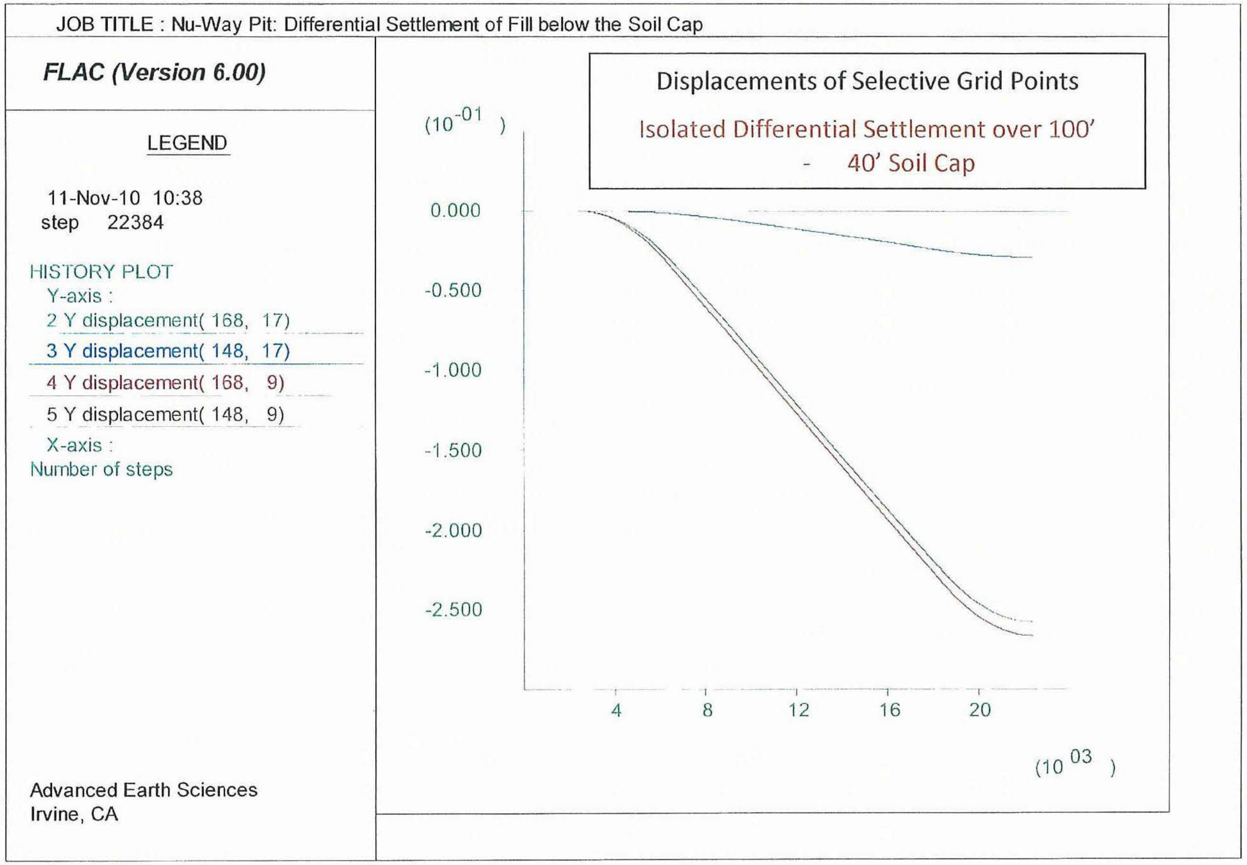
The same settlements as in Section 4.1 were applied over 100 ft instead of 30 ft in order to evaluate the influence of the distance, over which the differential settlement takes place, on the surface manifestation.

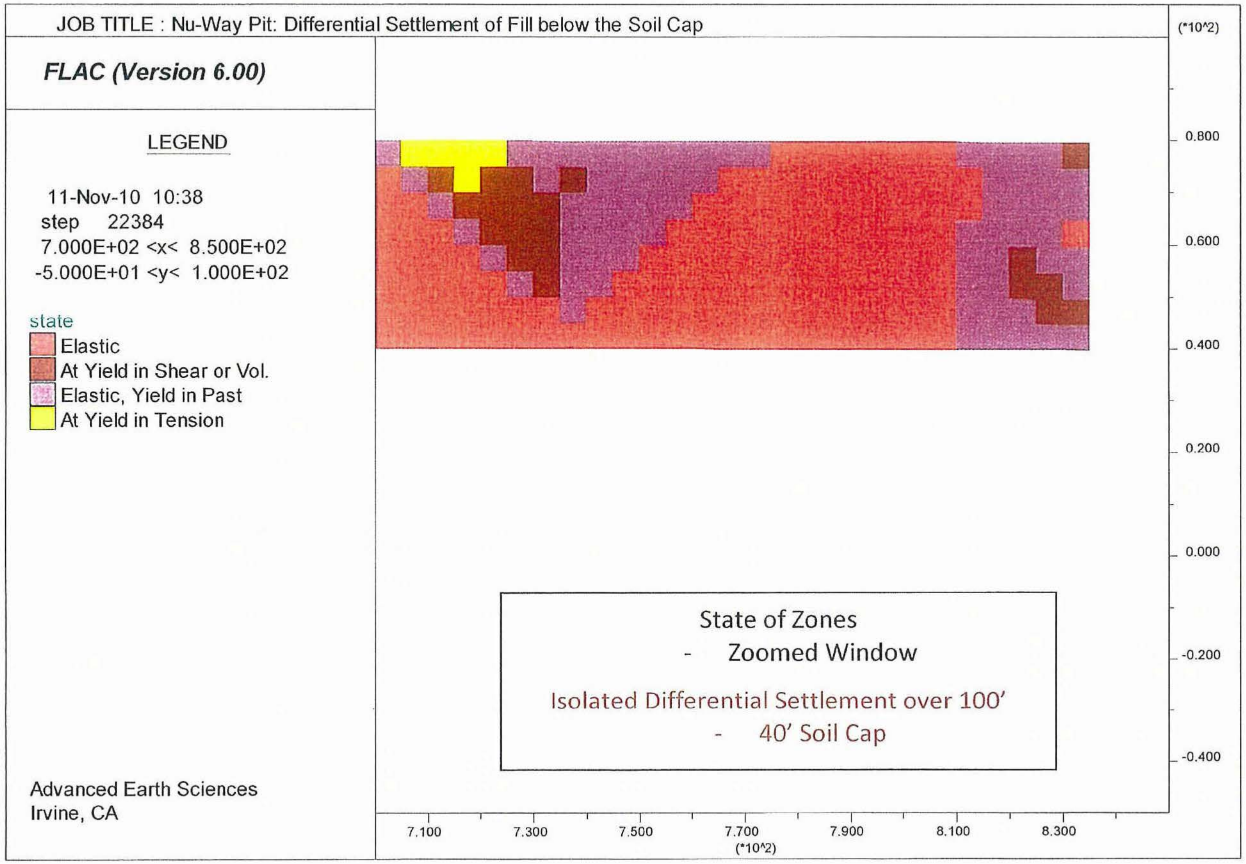




FLAC-59

0916.0288


JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 10:38
step 22384
HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(148, 17)
4 Y displacement(168, 9)
5 Y displacement(148, 9)
X-axis :
Number of steps
Advanced Earth Sciences
Irvine, CA
Displacements of Selective Grid Points
Isolated Differential Settlement over 100'
- 40' Soil Cap
(10^-01)
0.000
-0.500
-1.000
-1.500
-2.000
-2.500
4
8
12
16
20
(10^03)



FLAC-61

0916.0290

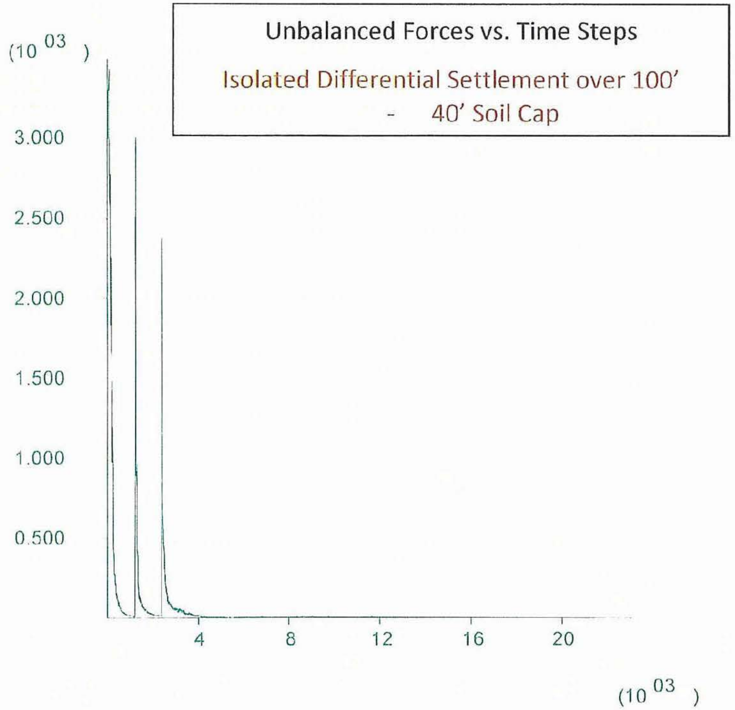
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:38
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

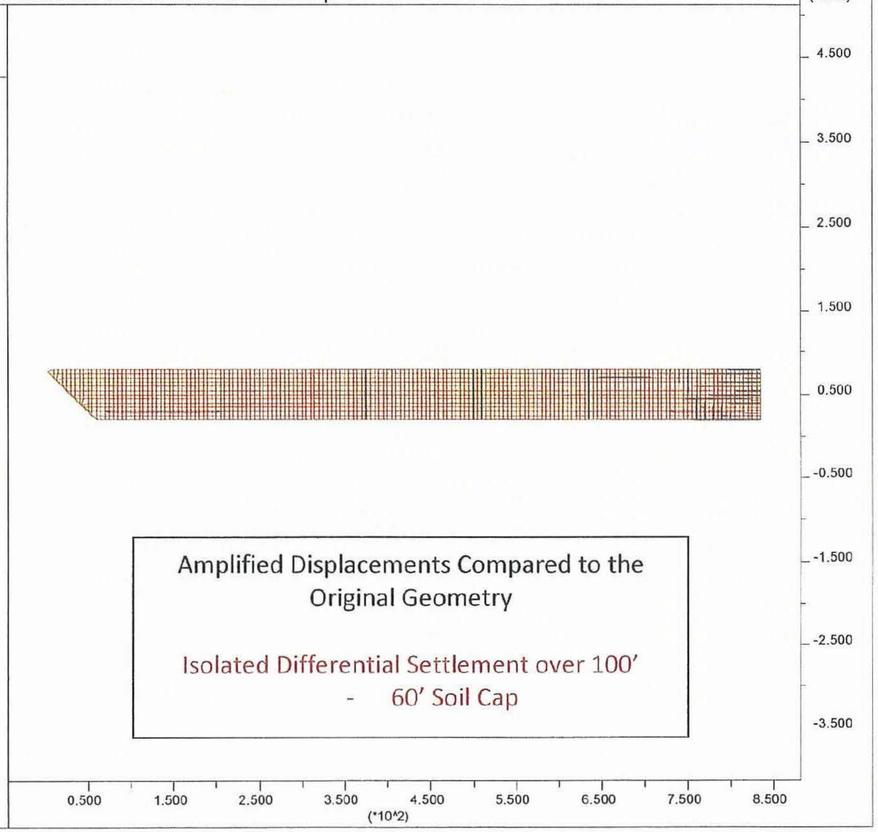
LEGEND

11-Nov-10 10:41
step 22384
-4.375E+01 <x< 8.812E+02
-4.125E+02 <y< 5.125E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 1.331E-01

Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 1.331E-01

Advanced Earth Sciences
Irvine, CA



JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:41
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 1.331E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 1.331E-01

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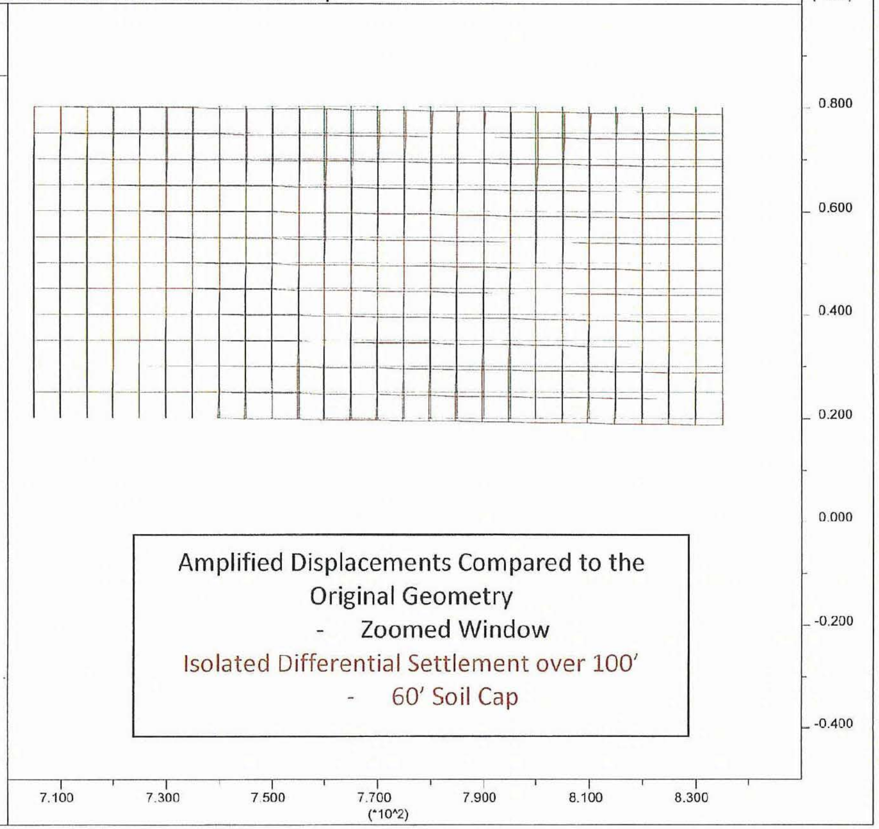

0916.0293

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:41
step 22384

HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(148, 17)
4 Y displacement(168, 5)
5 Y displacement(148, 5)
X-axis :
Number of steps

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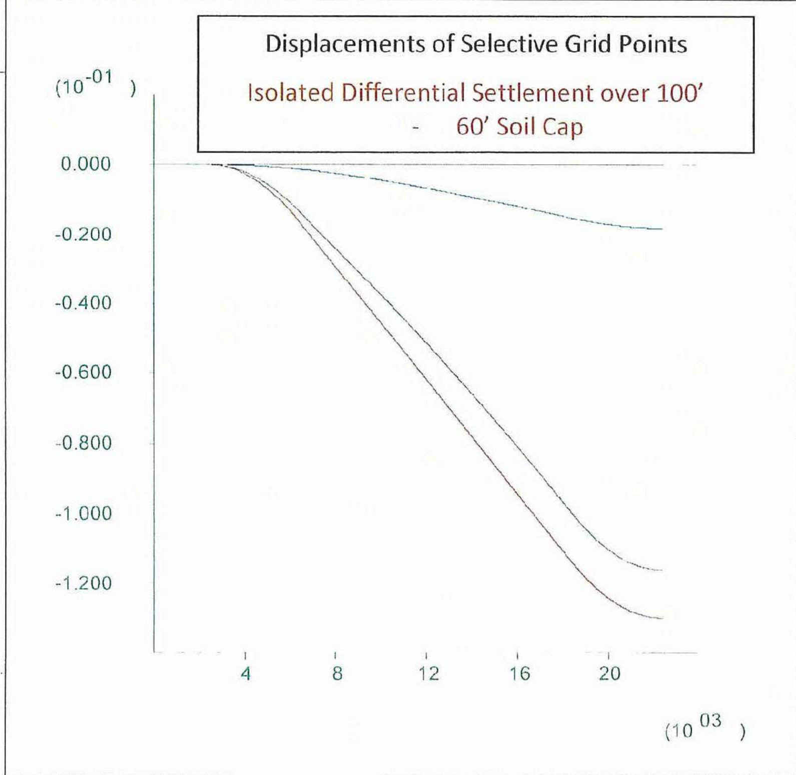
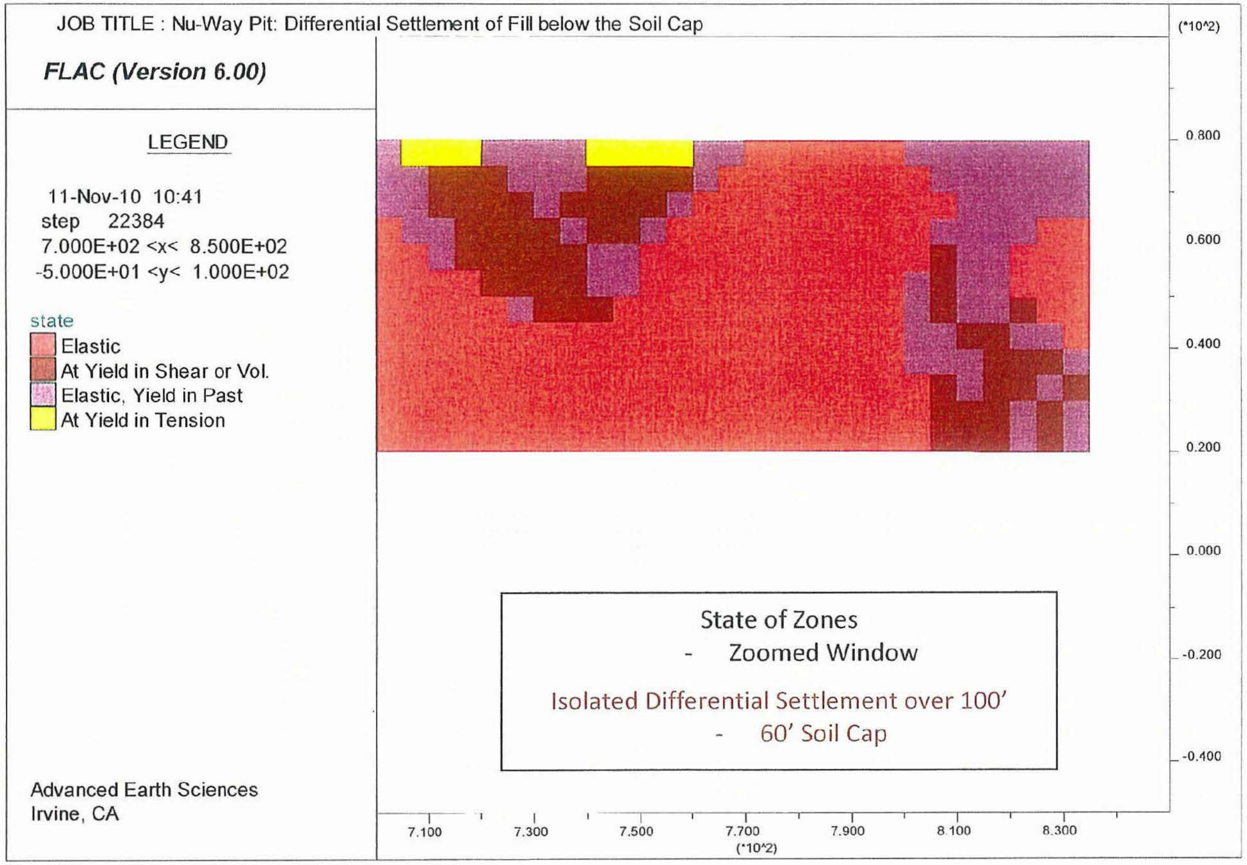


FLAC-65

0916.0294



FLAC-66

0916.0295

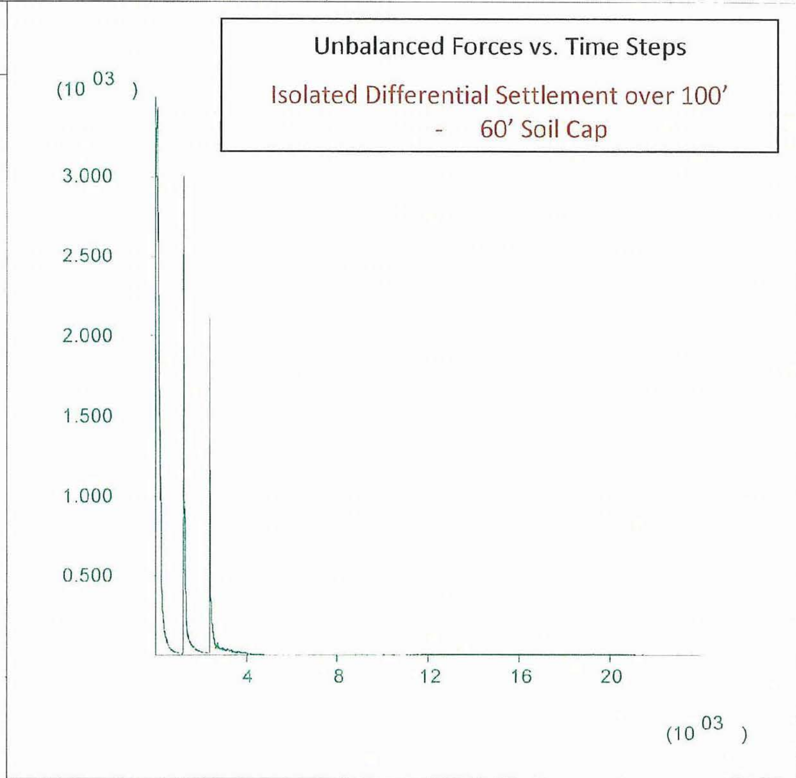
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:41
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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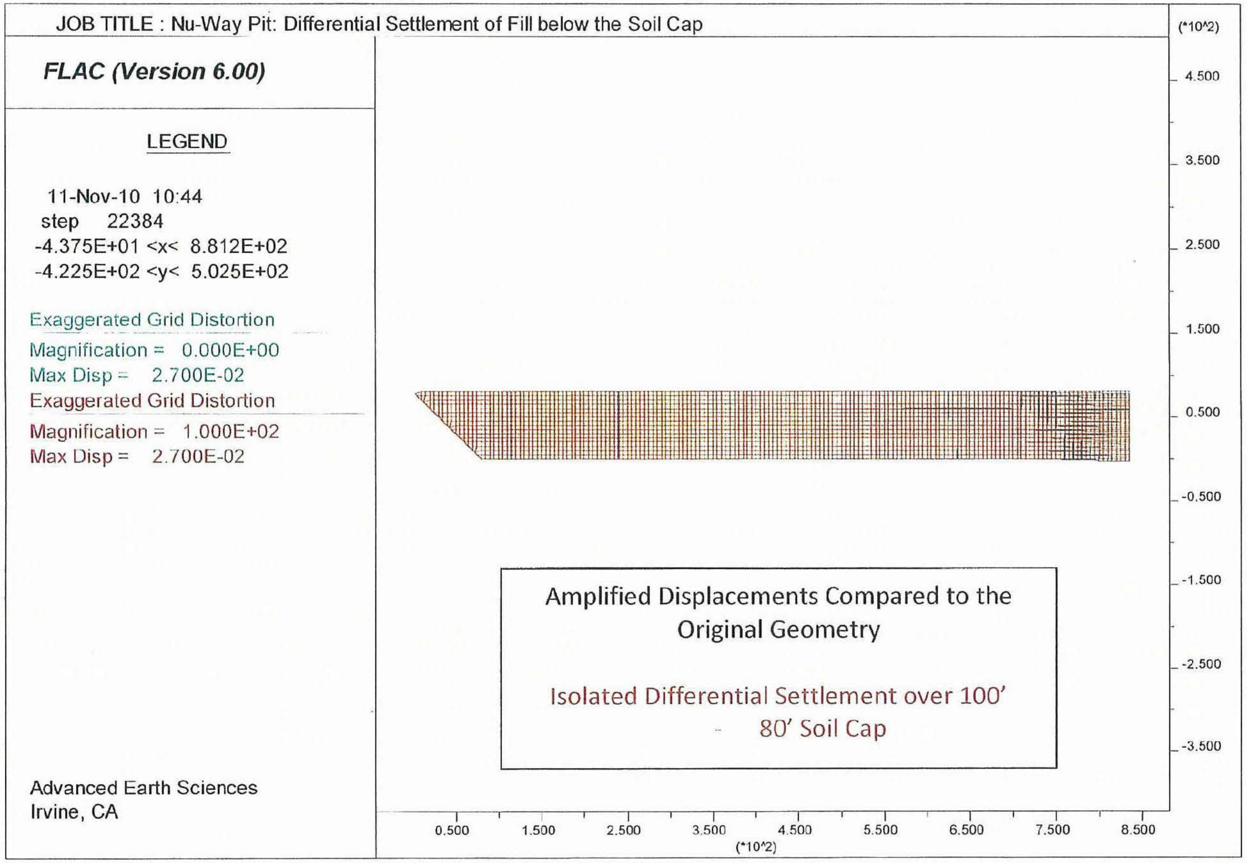
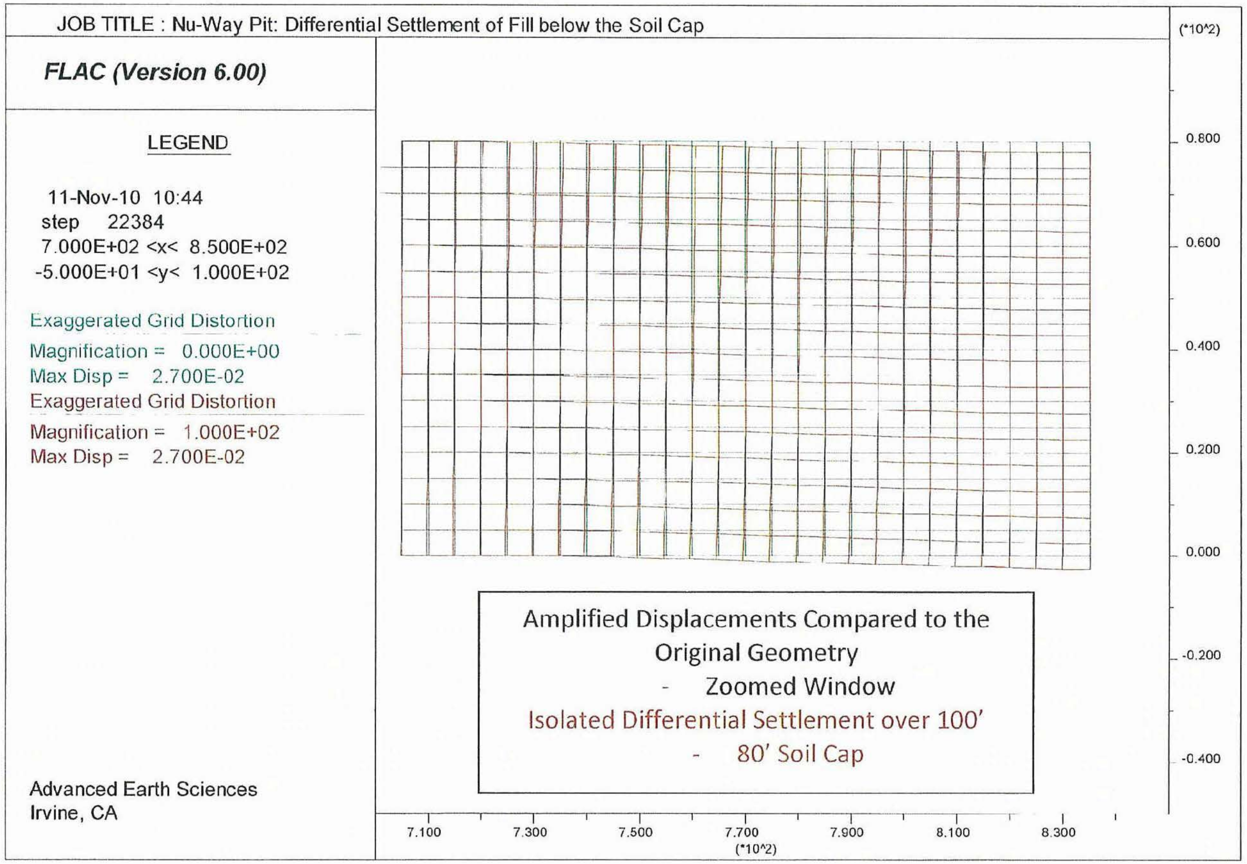
Irvine, CA

0916.0296



FLAC-68

0916.0297



FLAC-69

0916.0298

JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:44
step 22384

HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(148, 17)
4 Y displacement(168, 1)
5 Y displacement(148, 1)
X-axis :
Number of steps

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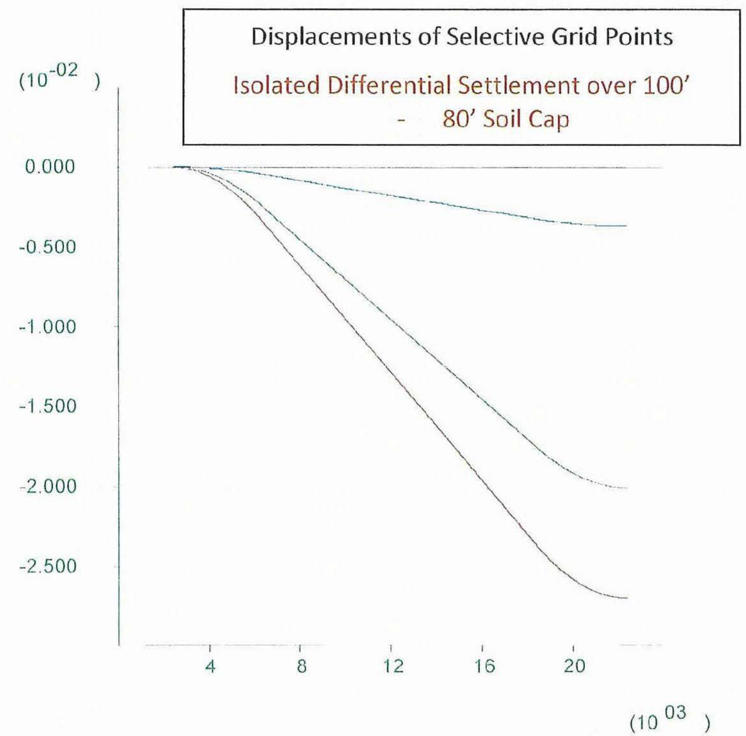
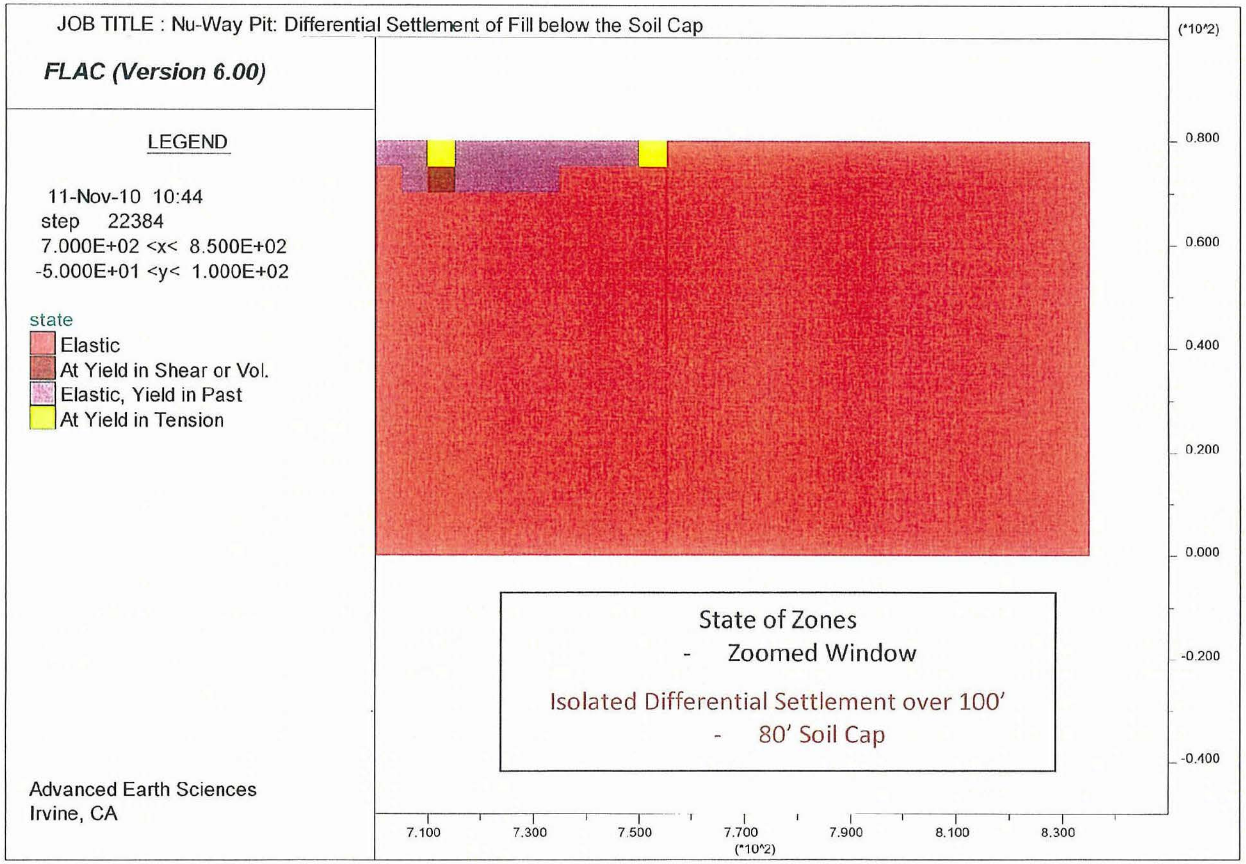
Irvine, CA



0916.0299



FLAC-71

0916.0300

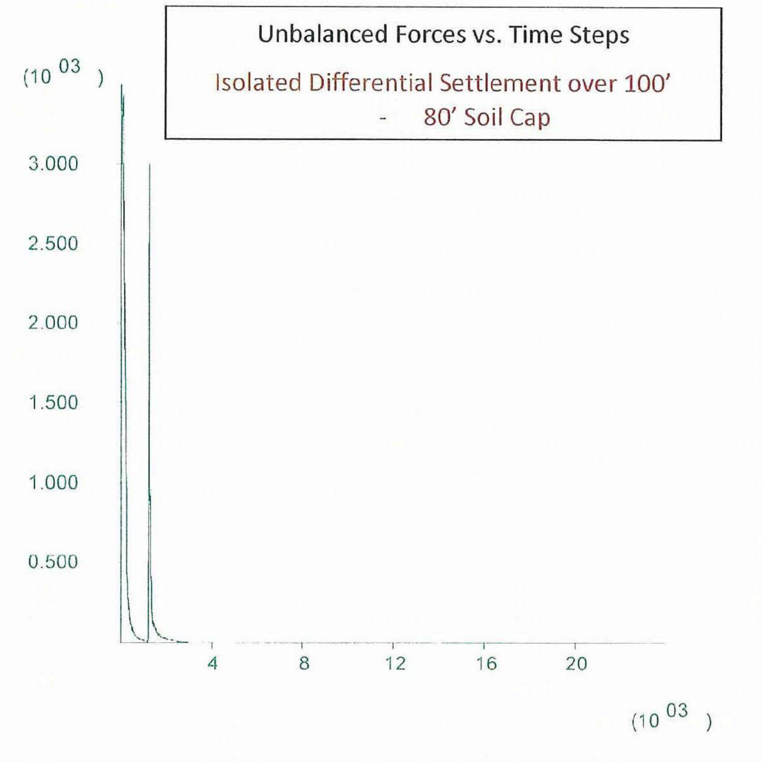
JOB TITLE : Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 10:44
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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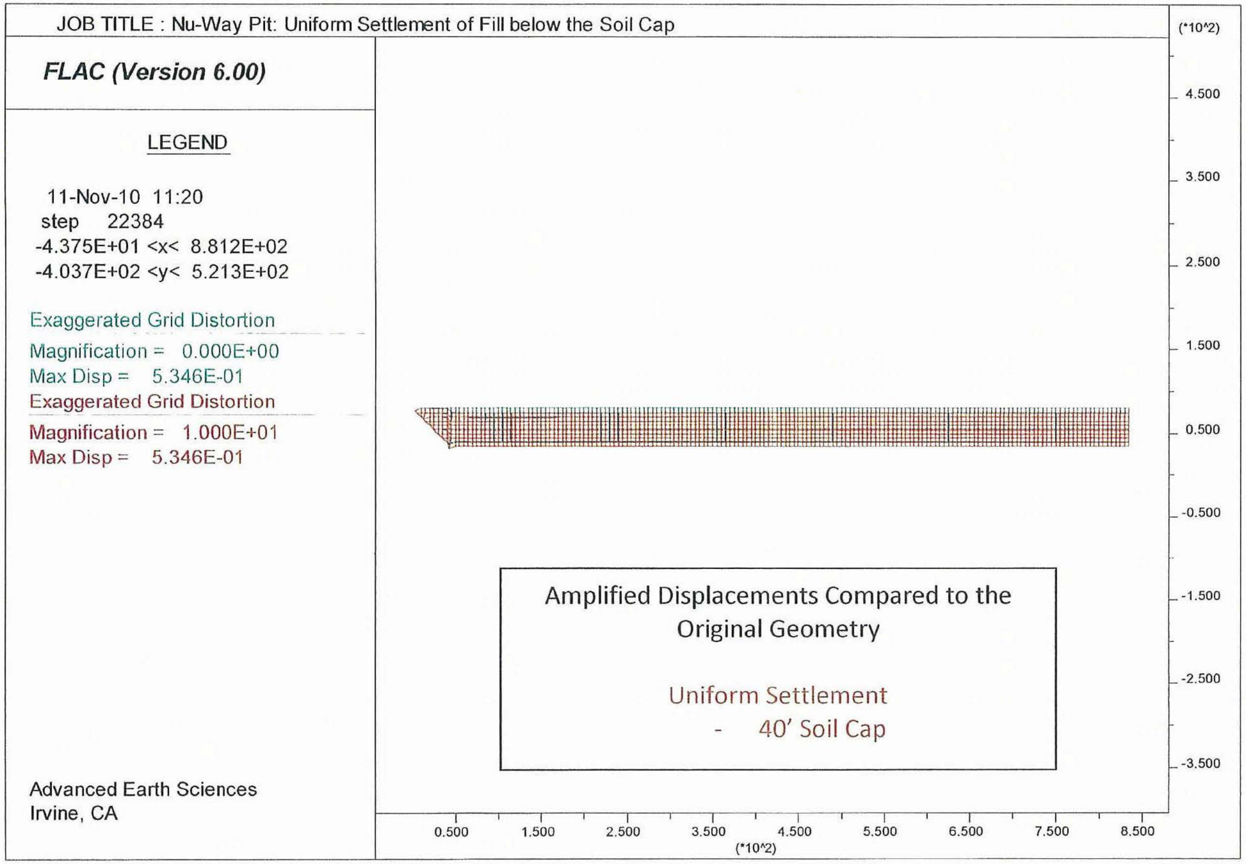
FLAC-72

0916.0301

5.3 Uniform Total Settlement

The following results are for analyses with average total settlements applied at the bottom of soil cap uniformly. Results show that most of these settlements would be transferred to the surface except directly above the side slopes of the pit and adjacent areas, where differential settlements take place.



0916.0302

JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 11:20
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 5.330E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 5.330E-01

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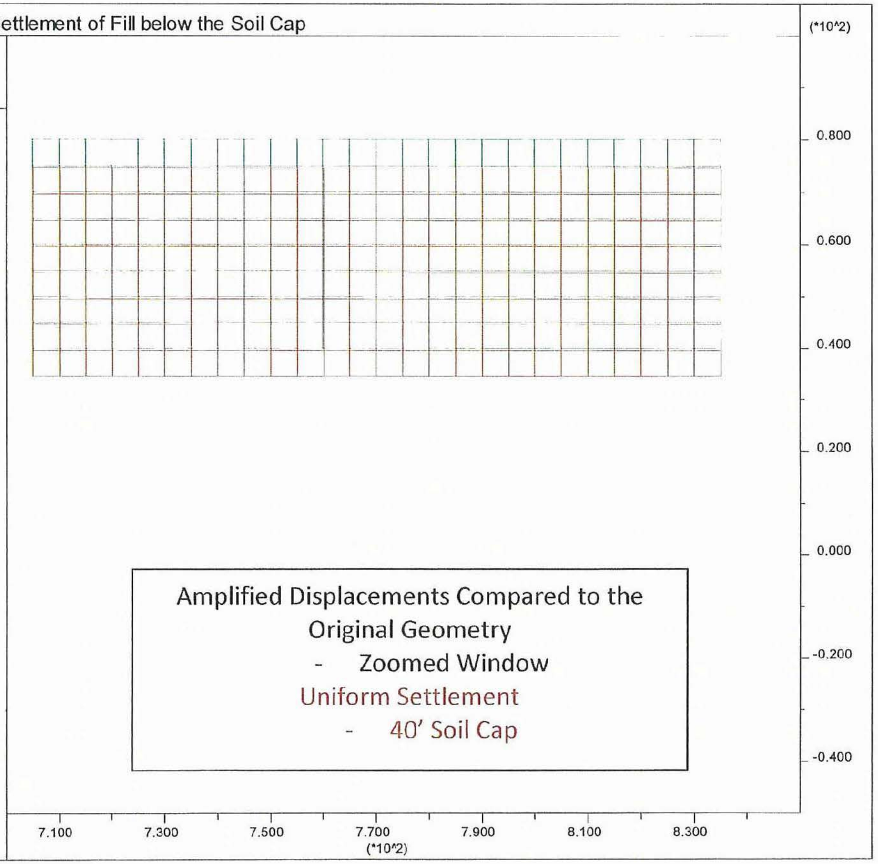
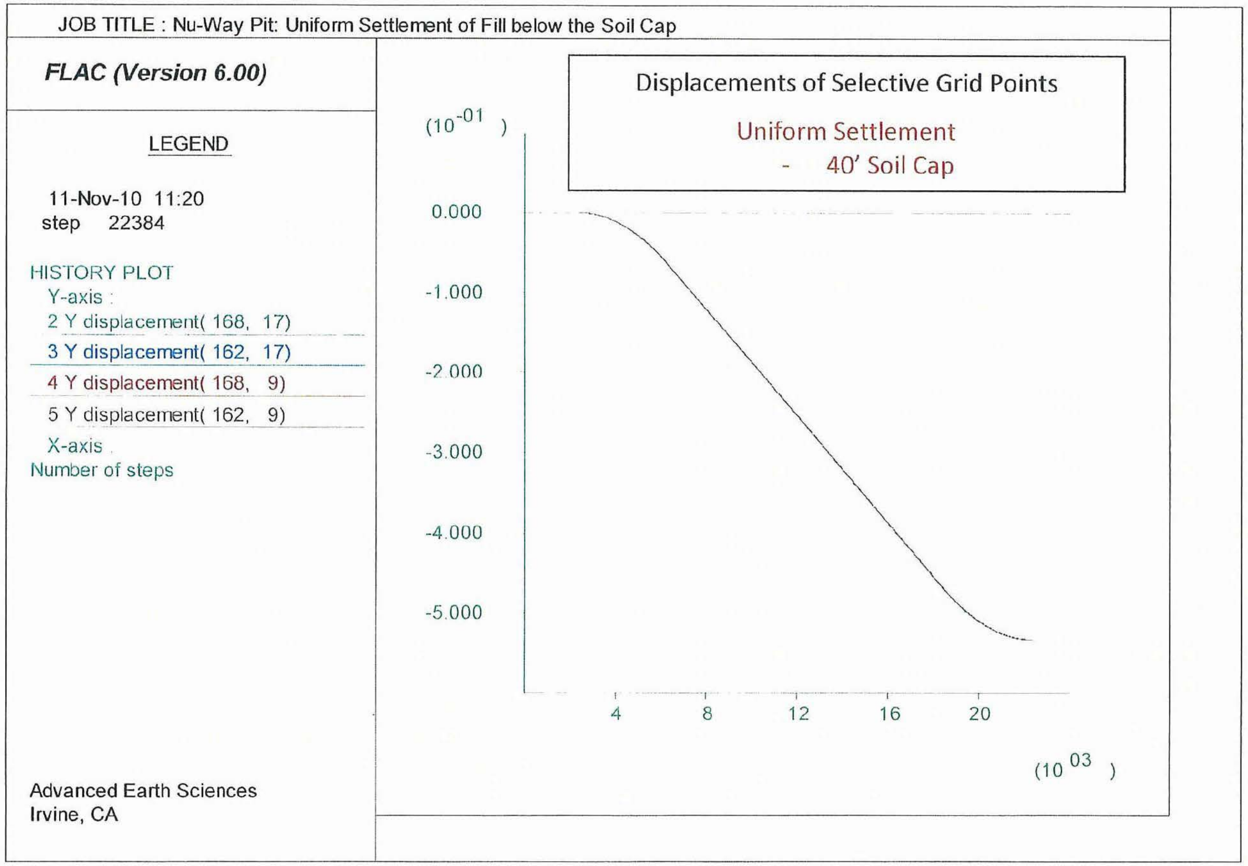


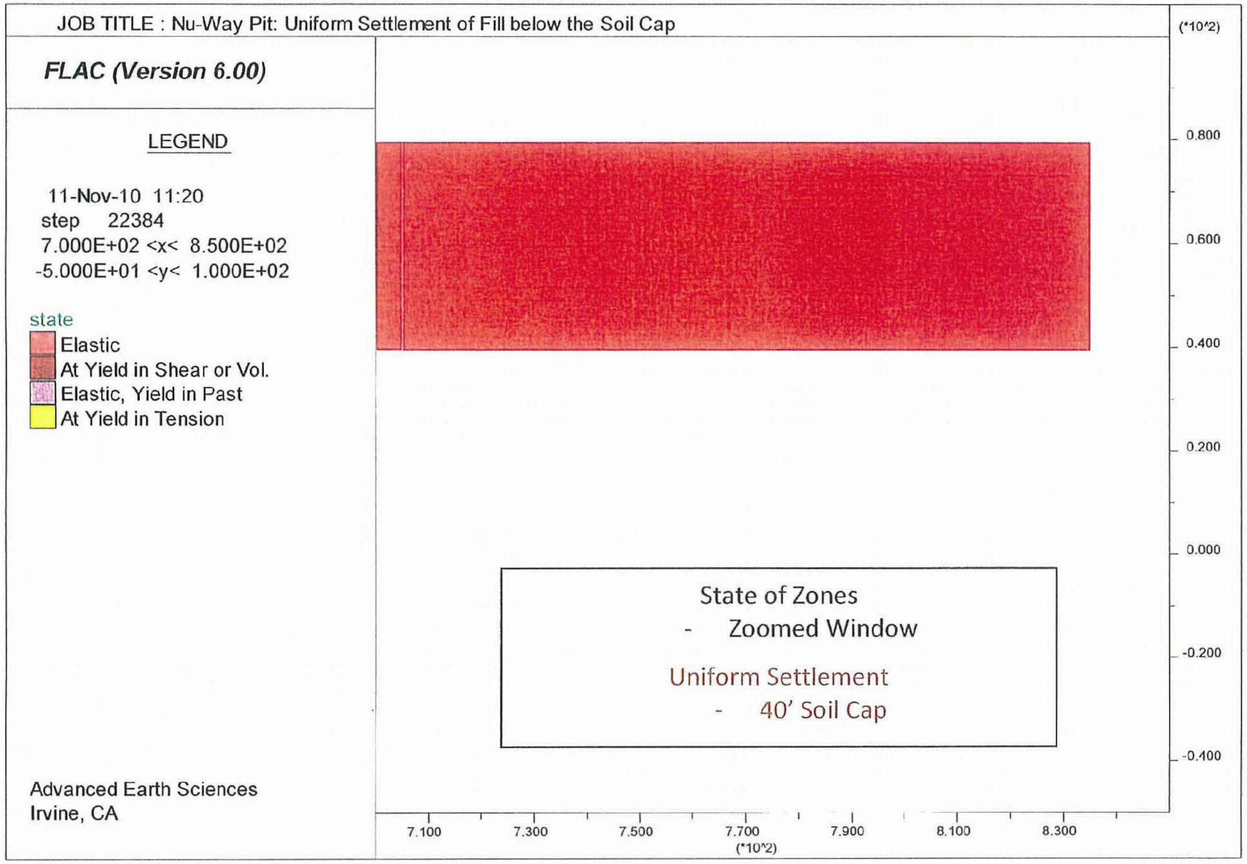
Amplified Displacements Compared to the Original Geometry
- Zoomed Window

Uniform Settlement
- 40' Soil Cap

FLAC-74

0916.0303


JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 11:20
step 22384
HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 9)
5 Y displacement(162, 9)
X-axis :
Number of steps
Advanced Earth Sciences
Irvine, CA
Displacements of Selective Grid Points
Uniform Settlement
- 40' Soil Cap
(10^-01)
0.000
-1.000
-2.000
-3.000
-4.000
-5.000
4
8
12
16
20
(10 03)



FLAC-76

0916.0305

JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 11:20
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps

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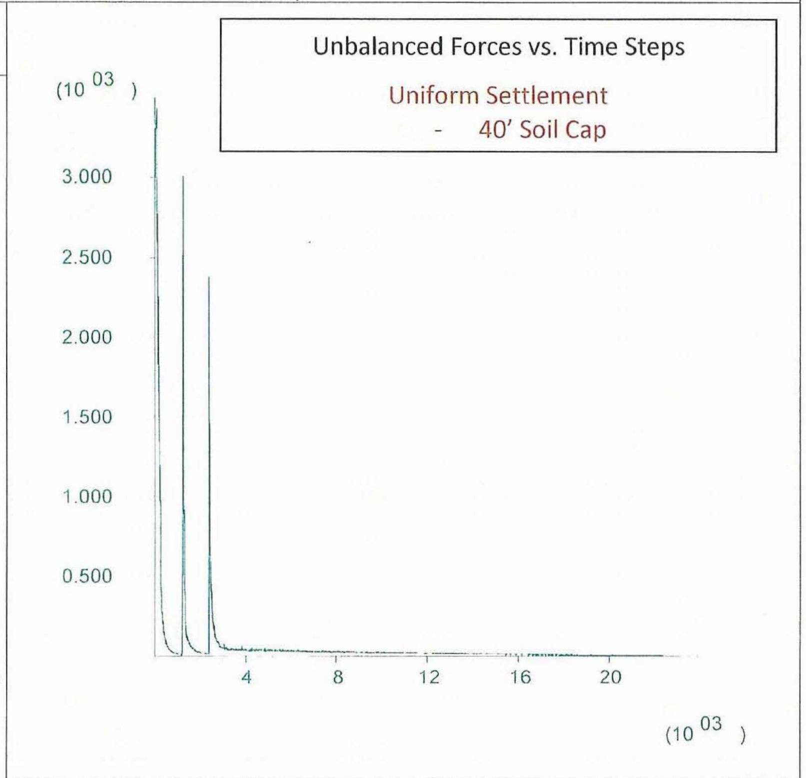
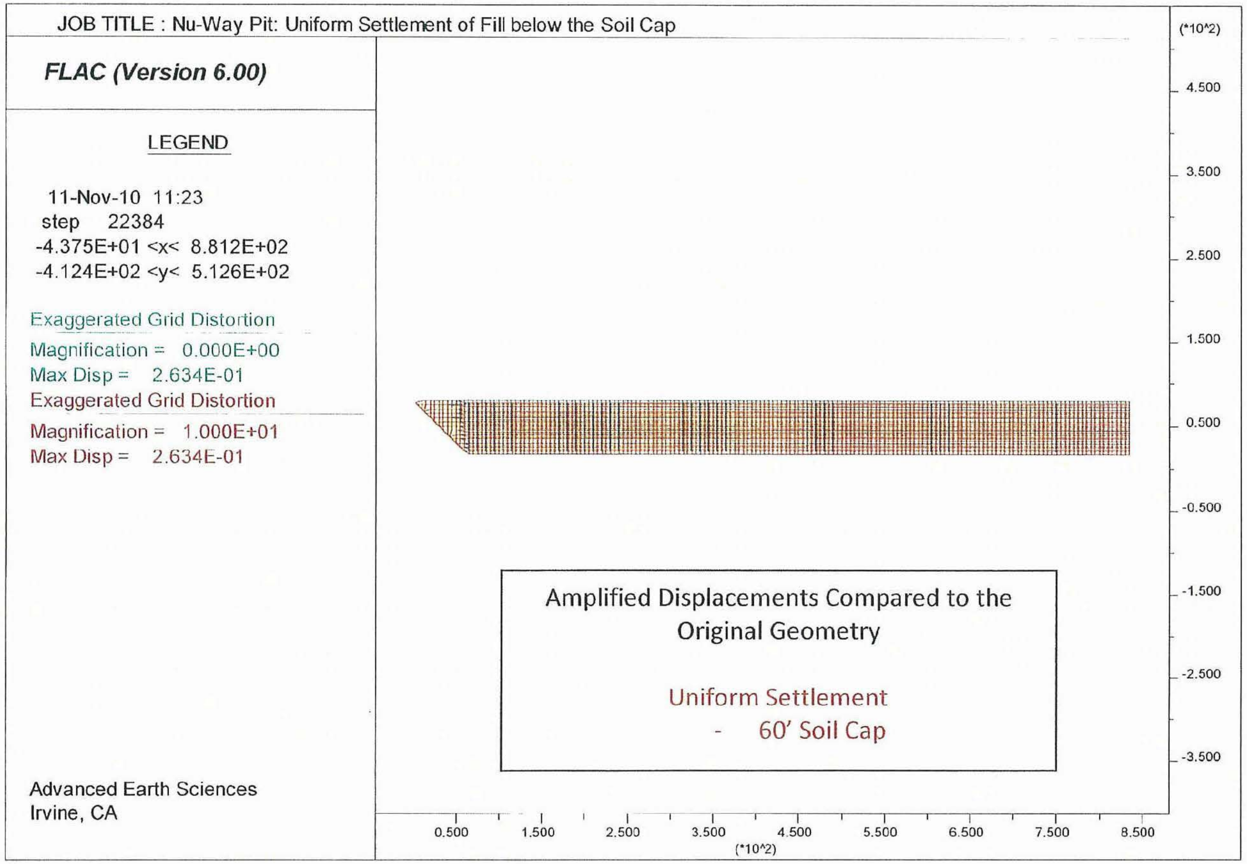


FLAC-77

0916.0306



0916.0307

FLAC-78

JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 11:23
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02

Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 2.601E-01
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 2.601E-01

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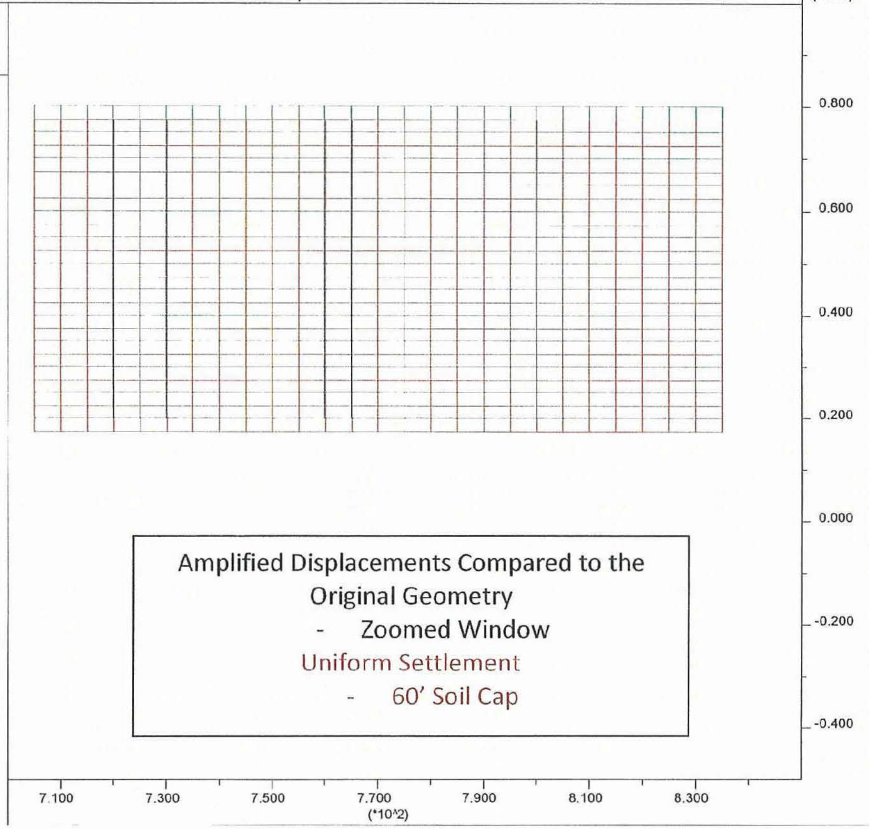
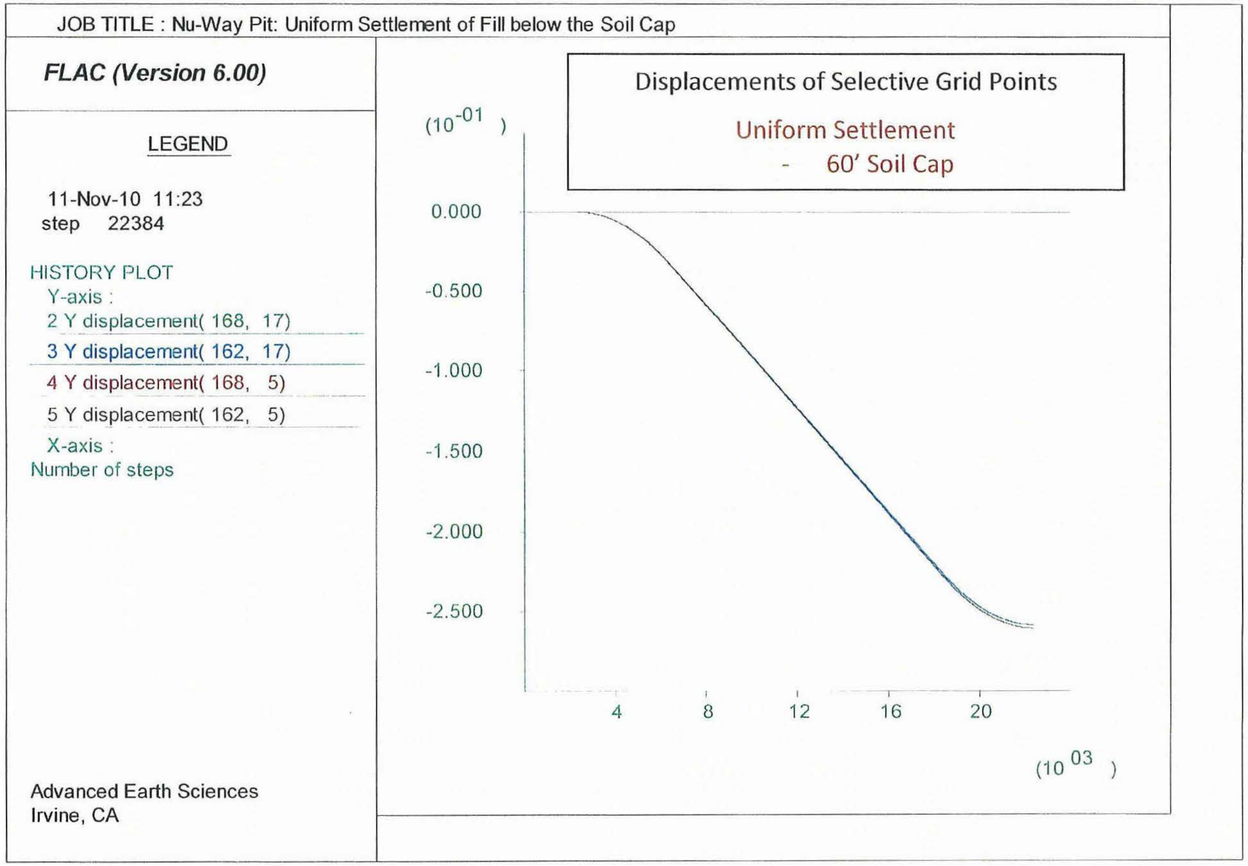
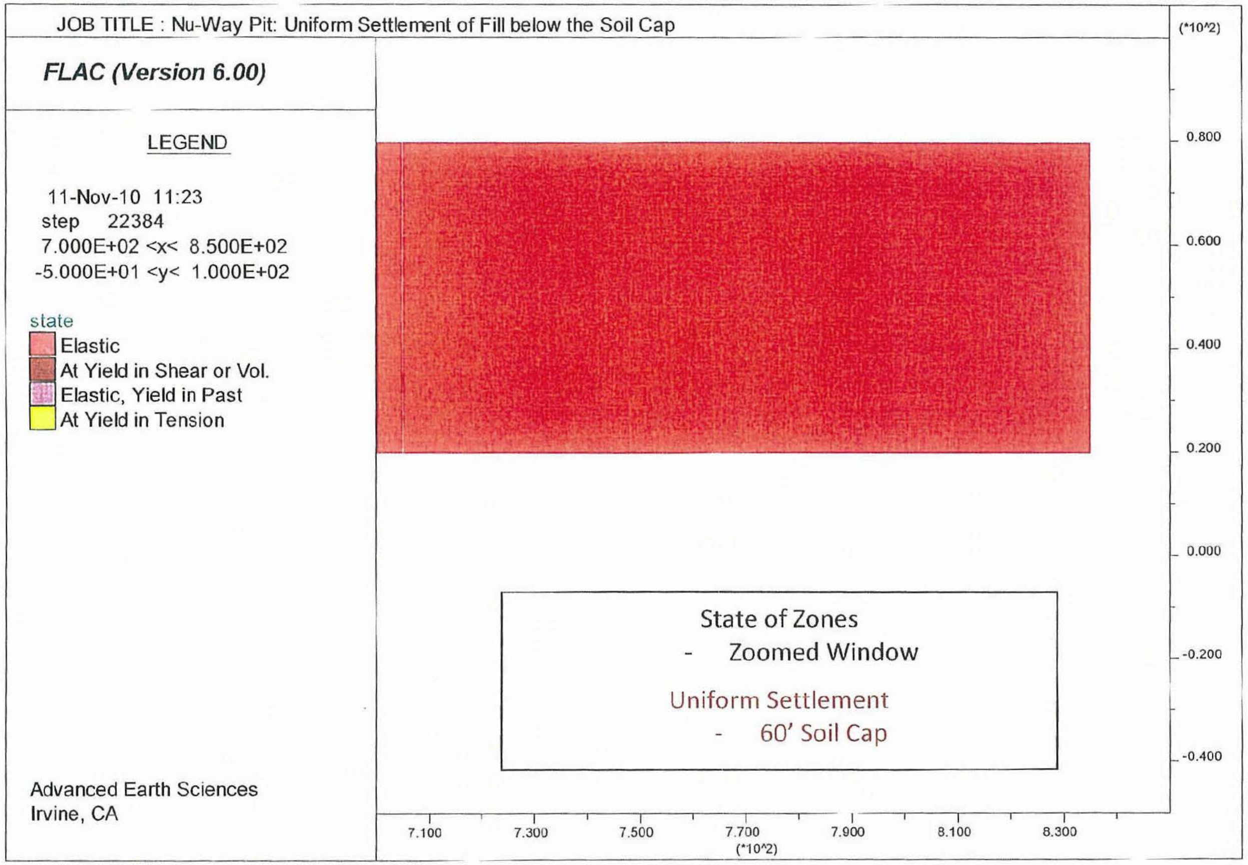


FLAC-79

0916.0308


JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 11:23
step 22384
HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 5)
5 Y displacement(162, 5)
X-axis :
Number of steps
Advanced Earth Sciences
Irvine, CA
Displacements of Selective Grid Points
Uniform Settlement
- 60' Soil Cap
(10-01)
0.000
-0.500
-1.000
-1.500
-2.000
-2.500
4
8
12
16
20
(10 03)

0916.0309


JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 11:23
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02
state
Elastic
At Yield in Shear or Vol.
Elastic, Yield in Past
At Yield in Tension
Advanced Earth Sciences
Irvine, CA
(*10^2)
0.800
0.600
0.400
0.200
0.000
-0.200
-0.400
7.100
7.300
7.500
7.700
(*10^2)
7.900
8.100
8.300
State of Zones
- Zoomed Window
Uniform Settlement
- 60' Soil Cap

0916.0310

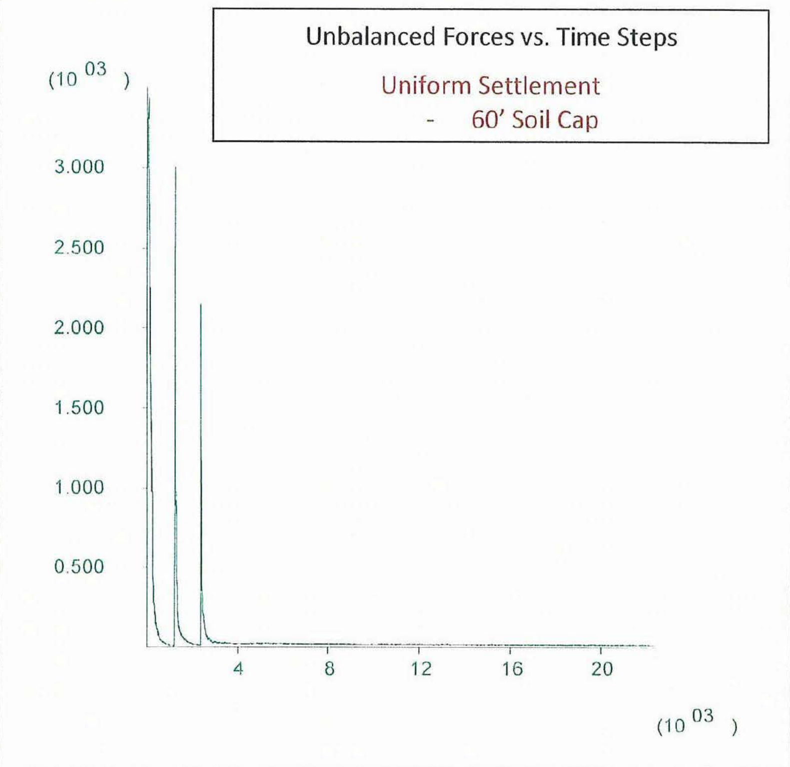
JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 11:23
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps



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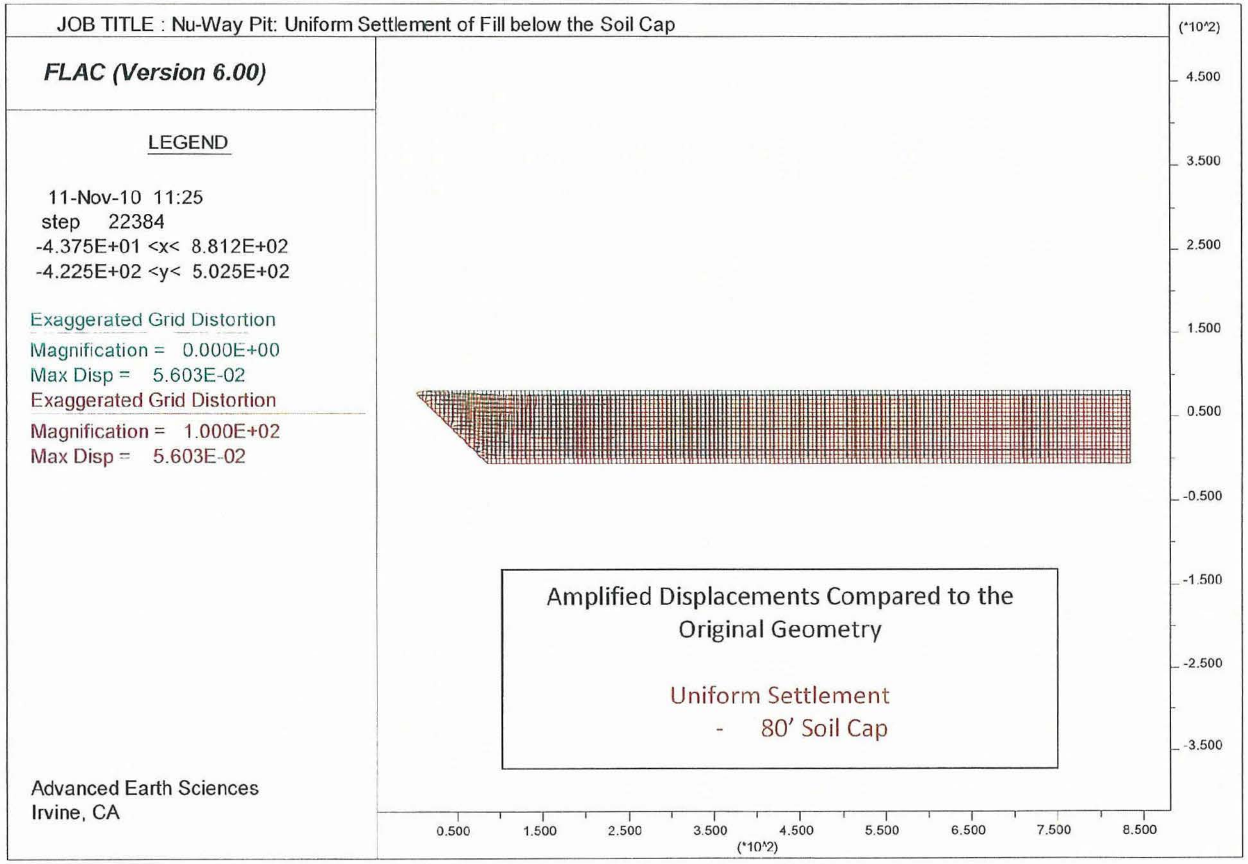
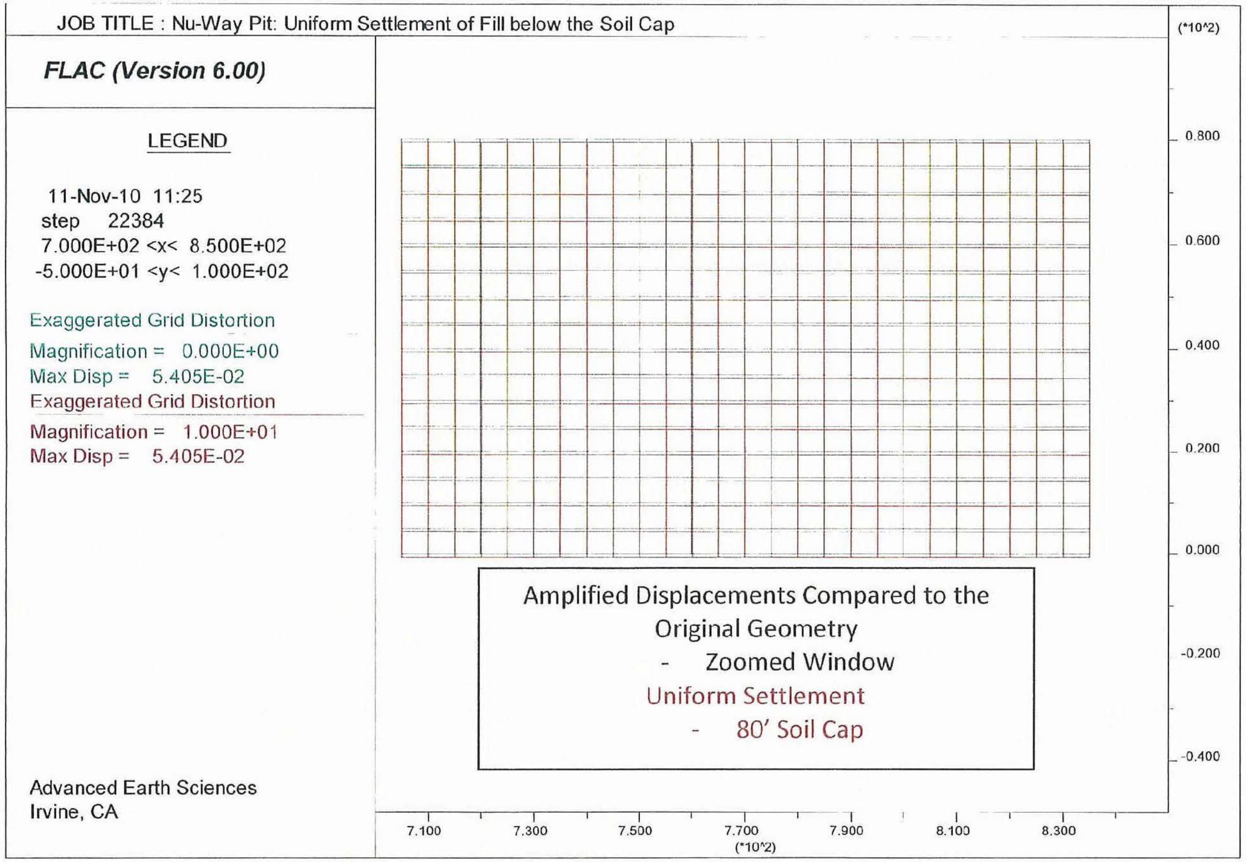
Irvine, CA


JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 11:25
step 22384
-4.375E+01 <x< 8.812E+02
-4.225E+02 <y< 5.025E+02
Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 5.603E-02
Exaggerated Grid Distortion
Magnification = 1.000E+02
Max Disp = 5.603E-02
Advanced Earth Sciences
Irvine, CA
(*10^2)
4.500
3.500
2.500
1.500
0.500
-0.500
-1.500
-2.500
-3.500
Amplified Displacements Compared to the Original Geometry
Uniform Settlement
- 80' Soil Cap
0.500
1.500
2.500
3.500
4.500
5.500
6.500
7.500
8.500
(*10^2)


JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 11:25
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02
Exaggerated Grid Distortion
Magnification = 0.000E+00
Max Disp = 5.405E-02
Exaggerated Grid Distortion
Magnification = 1.000E+01
Max Disp = 5.405E-02
Advanced Earth Sciences
Irvine, CA
Amplified Displacements Compared to the Original Geometry
- Zoomed Window
Uniform Settlement
- 80' Soil Cap
(*10^2)
0.800
0.600
0.400
0.200
0.000
-0.200
-0.400
7.100
7.300
7.500
7.700
7.900
8.100
8.300
(*10^2)

JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap

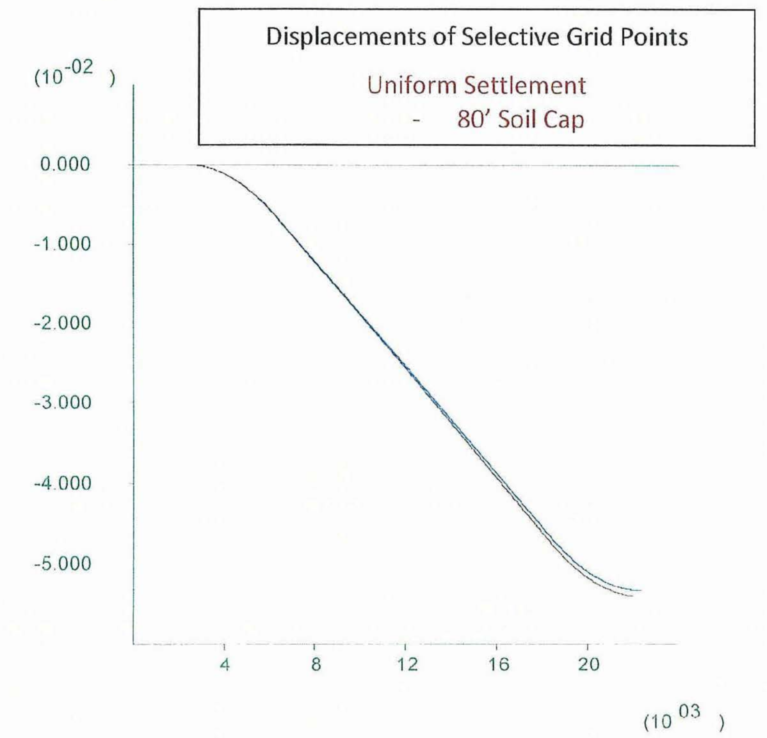
FLAC (Version 6.00)

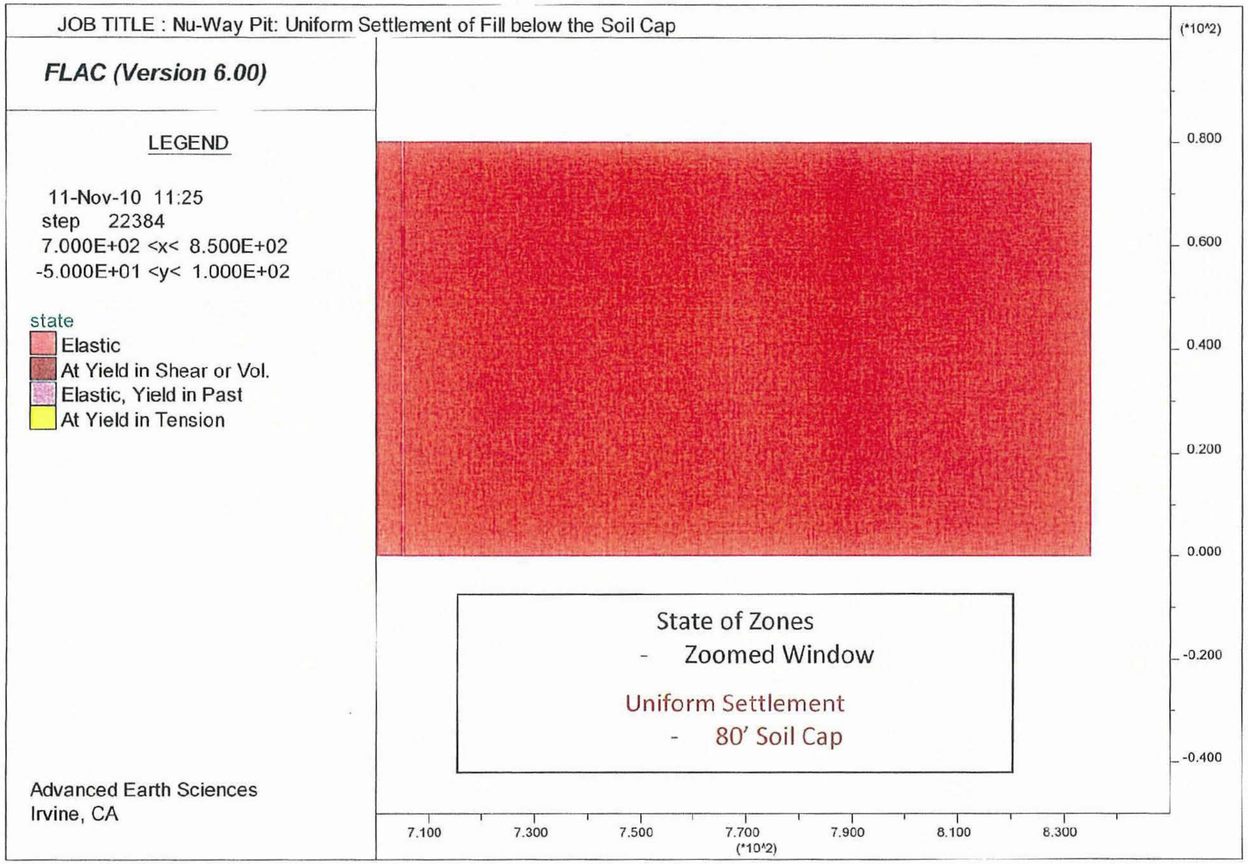
LEGEND

11-Nov-10 11:25
step 22384

HISTORY PLOT
Y-axis :
2 Y displacement(168, 17)
3 Y displacement(162, 17)
4 Y displacement(168, 1)
5 Y displacement(162, 1)
X-axis :
Number of steps

Advanced Earth Sciences
Irvine, CA




JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap
FLAC (Version 6.00)
LEGEND
11-Nov-10 11:25
step 22384
7.000E+02 <x< 8.500E+02
-5.000E+01 <y< 1.000E+02
state
Elastic
At Yield in Shear or Vol.
Elastic, Yield in Past
At Yield in Tension
Advanced Earth Sciences
Irvine, CA
State of Zones
- Zoomed Window
Uniform Settlement
- 80' Soil Cap
(*10^2)
0.800
0.600
0.400
0.200
0.000
-0.200
-0.400
7.100
7.300
7.500
7.700
7.900
8.100
8.300
(*10^2)

JOB TITLE : Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap

FLAC (Version 6.00)

LEGEND

11-Nov-10 11:25
step 22384

HISTORY PLOT
Y-axis :
999 Max. unbal. force
X-axis :
Number of steps

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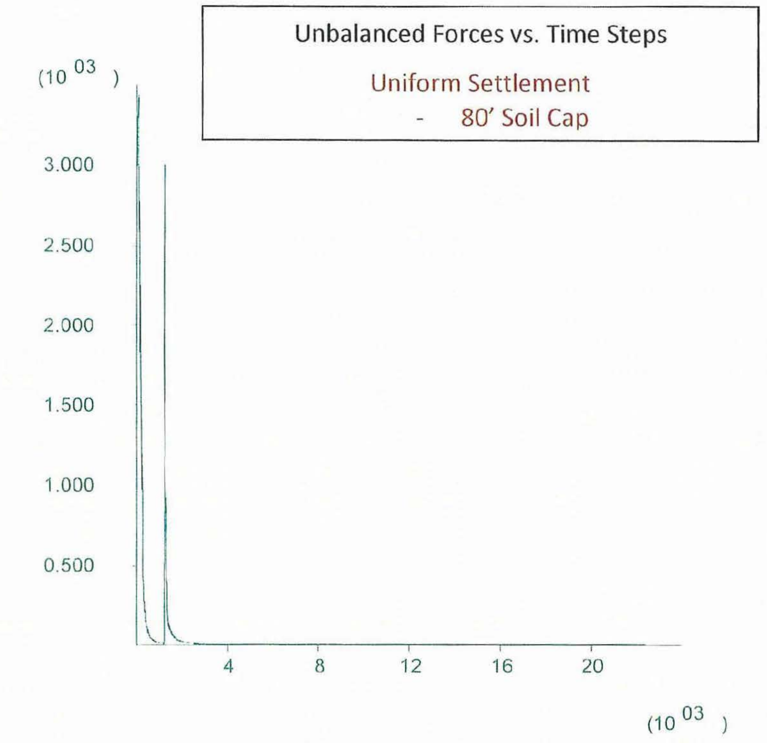
Irvine, CA



5.4 Parametric Study Input Files

(a) Section 5.1 Case (i) Input File

```
new
Title:
Nu-Way Pit: Collapsing of Cavity below the Soil Cap

;Model only one-half of the soil cap; No settlement of side
slope
;Use Modulus values consistent with previous study
;Run by: TS            Date: August 2010

config extra 2
grid 835,40
model elastic
;gen same 0. 40. 835. 40. 835. 0.
gen line 0.0,40.0 40.0,0.0

model null region 2 3
group 'null' region 2 3
group delete 'null'
group 'User:new1' notnull
model mohr notnull group 'User:new1'
prop density=3.73 bulk=1.33E7 shear=4.44e6 cohesion=250.0
friction=30.0 &
 dilation=0.0 tension=0.0 notnull group 'User:new1'
fix  x y mark
fix  y i 42 836 j 1
fix  x i 836 j 1 40
set gravity=32.18504
history 999 unbalanced
solve elastic

ini xd=0.
ini yd=0.

apply yv=-0.00025 i 404 413 j 1

def cav_coll
 loop ns (1,10000)
  if ydisp(404,1)>-0.25
   command
    step 1
   end_command
  else
   command
    apply yv=0. i 404 413 j 1
    solve f=100
   end_command
  exit
  endif
 end_loop
end

cav_coll
set plot emf
set output un_fine.emf
plot pen his 999
window 350 450 -50 50
set output def_fine.emf
plot pen grid mag 0 gr grid mag 10 red
```

(b) Section 5.1 Case (ii) Input File

```
new
Title:
Nu-Way Pit: Collapsing of Cavity below the Soil Cap

;Model only one-half of the soil cap; No settlement of side
slope
;Use Modulus values consistent with previous study
;Run by: TS            Date: August 2010

config extra 2
grid 167,8
model elastic
gen same 0. 40. 835. 40. 835. 0.
gen line 0.0,40.0 40.0,0.0

model null region 2 3
group 'null' region 2 3
group delete 'null'
group 'User:new1' notnull
model mohr notnull group 'User:new1'
prop density=3.73 bulk=1.33E7 shear=4.44e6 cohesion=250.0
friction=30.0 &
 dilation=0.0 tension=0.0 notnull group 'User:new1'
fix  x y mark
fix  y i 10 168 j 1
fix  x i 168 j 1 9
set gravity=32.18504
history 999 unbalanced
solve elastic

ini xd=0.
ini yd=0.

apply yv=-0.00025 i 82 83 j 1
```


0916.0317

```
def cav_coll
 loop ns (1,10000)
  if ydisp(82,1)>-0.25
   command
    step 1
   end_command
  else
   command
    apply yv=0. i 82 83 j 1
    solve f=100
   end_command
  exit
  endif
 end_loop
end

cav_coll
set plot emf
set output un_coarse.emf
plot pen his 999
window 350 450 -50 50
set output def_coarse.emf
plot pen grid mag 0 gr grid mag 10 red
```

(c) Section 5.1 Case (iii) Input File

```
new
Title:
Nu-Way Pit: Collapsing of Cavity below the Soil Cap - Disp by
Ramping

;Model only one-half of the soil cap; No settlement of side
slope
;Use Modulus values consistent with previous study
;Run by: TS           Date: August 2010

config extra 2
grid 835,40
model elastic
;gen same 0. 40. 835. 40. 835. 0.
gen line 0.0,40.0 40.0,0.0

model null region 2 3
group 'null' region 2 3
group delete 'null'
group 'User:new1' notnull
model mohr notnull group 'User:new1'
prop density=3.73 bulk=1.33E7 shear=4.44e6 cohesion=250.0
friction=30.0 &
 dilation=0.0 tension=0.0 notnull group 'User: new1'
fix  x y mark
fix  y i 42 836 j 1
fix  x i 836 j 1 40
set gravity=32.18504
history 999 unbalanced
solve elastic

ini xd=0.
ini yd=0.

;apply yv=-0.00025 i 404 413 j 1

;def cav_coll
; loop ns (1,10000)
;  if ydisp(404,1)>-0.25
;   command
;    step 1
;   end_command
;  else
;   command
;    apply yv=0. i 404 413 j 1
;    solve f=100
;   end_command
;  exit
;  endif
; end_loop
;end

;cav_coll

;*********************************************************
set large

def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
        udapp=dis_inc*float(ns)
else
        if ns<=(n_step-ns_ramp)
                udapp=dis_inc*float(ns_ramp)
        else
                udapp=dis_inc*float(n_step-ns)
        endif
endif

command
ini yv=udapp i 404,413 j 1
step 1
```


0916.0318

```
end_command

end_loop
end

set n_step=20000 tot_dis=-2.5e-1 ;should be negative
superstep

solve
;*********************************************************

set plot emf
set output un_fine_ra.emf
plot pen his 999
window 350 450 -50 50
set output def_fine_ra.emf
plot pen grid mag 0 gr grid mag 10 red
```

(d) Section 5.1 Case (iv) Input File

```
new
Title:
Nu-Way Pit: Collapsing of Cavity below the Soil Cap - Disp by
Ramping

;Model only one-half of the soil cap; No settlement of side
slope
;Use Modulus values consistent with previous study
;Run by: TS           Date: August 2010

config extra 2
grid 167,8
model elastic
gen same 0. 40. 835. 40. 835. 0.
gen line 0.0,40.0 40.0,0.0

model null region 2 3
group 'null' region 2 3
group delete 'null'
group 'User:new1' notnull
model mohr notnull group 'User:new1'
prop density=3.73 bulk=1.33E7 shear=4.44e6 cohesion=250.0
friction=30.0 &
 dilation=0.0 tension=0.0 notnull group 'User:new1'
fix  x y mark
fix  y i 10 168 j 1
fix  x i 168 j 1 9
set gravity=32.18504
history 999 unbalanced
solve elastic

ini xd=0.
ini yd=0.
```

```
;apply yv=-0.00025 i 82 83 j 1

;def cav_coll
; loop ns (1,10000)
;  if ydisp(82,1)>-0.25
 ;   command
;     step 1
;    end_command
;  else
;    command
;     apply yv=0. i 82 83 j 1
;     solve f=100
;    end_command
;  exit
;  endif
; end_loop
;end

;cav_coll
;*********************************************************
set large

def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
       udapp=dis_inc*float(ns)
else
       if ns<=(n_step-ns_ramp)
              udapp=dis_inc*float(ns_ramp)
       else
              udapp=dis_inc*float(n_step-ns)
       endif
endif

command
ini yv=udapp i 82,83 j 1
step 1
end_command

end_loop
end

set n_step=20000 tot_dis=-2.5e-1 ;should be negative
superstep
```


0916.0319

```
solve
;*******************************************************

set plot emf
set output un_coarse_ra.emf
plot pen his 999
window 350 450 -50 50
set output def_coarse_ra.emf
plot pen grid mag 0 gr grid mag 10 red
```

(e) Section 5.2 Input File

```
new
Title:
Nu-Way Pit: Differential Settlement of Fill below the Soil Cap

;Differential settlement applied over 100ft length for 40, 60
and 80-foot soil caps
;Model only one-half of the soil cap; No settlement of side
slope
;Use Modulus values revised using Vs consistent with downhole
measurement (880ft/s)
;Run by: TS            Date: Sept 2010

config extra 2
;Initial model for 80 ft soil cap
grid 167,16
model elastic

;Assign coordinates and cut 1:1 side slope
gen same 0. 80. 835. 80. 835. 0.
gen line 80.0,0.0 0.0,80.0
model null region 2 3
group 'null' region 2 3
group delete 'null'

group 'User:new1' notnull
model mohr notnull group 'User:new1'
prop density=3.73 bulk=8.67E6 shear=2.89e6 cohesion=250.0
friction=30.0 &
 dilation=0.0 tension=0.0 notnull group 'User:new1'

fix  x y mark
fix  y i 18 168 j 1
fix  x i 168 j 1 17
set gravity=32.18504
history 999 unbalanced

def nigp
int n_igp
n_igp=igp
end
nigp

;Solve as elastic material to initialize stresses & then apply
specified soil parameters
solve elastic

def mod_ini
command
ini xd=0.
ini yd=0.
end_command
 loop ii (1,izones)
  loop jj (1,jzones)
       if model(ii,jj) # 1 then
        ex_1(ii,jj)=shear_mod(ii,jj)
        state(ii,jj)=0
       endif
  end_loop
 end_loop
end

;Initialize displacements to zero
mod_ini

his 1 yd i=1 j=17    ;check

;G/Gmax Table for Upper Sand (Seed and Idriss, 1970; Idriss,
1990)
table 1 0.,1. 0.000001,1. 0.00000316,1. 0.00001,0.99
0.0000316,0.96 0.0001,0.85 &
0.000316,0.655 0.001,0.37 0.00316,0.19 0.01,0.085 0.1,0.0085

save elas_DNWP.sav

;40-foot thick soil cap
________________________________*************************

m n i=1,167 j=1,8
fix  y i 10 168 j 9

his 2 yd i=168 j=17
his 3 yd i=148 j=17
his 4 yd i=168 j=9
his 5 yd i=148 j=9

set large

def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
```

```
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
        udapp=dis_inc*float(ns)
else
        if ns<=(n_step-ns_ramp)
                udapp=dis_inc*float(ns_ramp)
        else
                udapp=dis_inc*float(n_step-ns)
        endif
endif

command
ini yv=0. var udapp 0. i 148,168 j 9
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
        shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
        bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 tot_dis=-2.66e-1 ;should be negative
superstep

solve
save NL_40D100.sav

;*** plot commands ***
set output pl_DNWP40_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_DNWP40_dis.emf
plot pen his 2 3 4 5
set output pl_DNWP40_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_DNWP40_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_DNWP40_st.emf
plot pen state block

set hisfile his_DNWP40.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1)
o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of
simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of
simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of
simulation'
loop ii (1,igp)
        arr1(ii)=string(x(ii,jgp))
        arr2(ii)=string(y(ii,jgp))
        arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open('Sur_40D100.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_i
gp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out

;60-foot thick soil cap
________________________________*************************

res elas_DNWP.sav

m n i=1,167 j=1,4

fix y i 14 168 j 5

his 2 yd i=168 j=17
his 3 yd i=148 j=17
his 4 yd i=168 j=5
his 5 yd i=148 j=5

set large

def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
        udapp=dis_inc*float(ns)
else
        if ns<=(n_step-ns_ramp)
                udapp=dis_inc*float(ns_ramp)
        else
                udapp=dis_inc*float(n_step-ns)
```

```
        endif
endif

command
ini yv=0. var udapp 0. i 148,168 j 5
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
        shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
        bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 tot_dis=-1.3e-1 ;should be negative
superstep

solve
save NL_60D100.sav

;*** plot commands ***
set output pl_DNWP60_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_DNWP60_dis.emf
plot pen his 2 3 4 5
set output pl_DNWP60_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_DNWP60_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_DNWP60_st.emf
plot pen state block

set hisfile his_DNWP60.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1)
o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of
simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of
simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of
simulation'
loop ii (1,igp)
        arr1(ii)=string(x(ii,jgp))
        arr2(ii)=string(y(ii,jgp))
        arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open ('Sur_60D100.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_i
gp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out

;80-foot thick soil cap
________________________________*************************

res elas_DNWP.sav

his 2 yd i=168 j=17
his 3 yd i=148 j=17
his 4 yd i=168 j=1
his 5 yd i=148 j=1

set large

def superstep
float dis_inc tot_dis
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
if ns<=ns_ramp
        udapp=dis_inc*float(ns)
else
        if ns<=(n_step-ns_ramp)
                udapp=dis_inc*float(ns_ramp)
        else
                udapp=dis_inc*float(n_step-ns)
        endif
endif

command
ini yv=0. var udapp 0. i 148,168 j 1
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
        shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
        bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop
```

```
end_loop
end

set n_step=20000 tot_dis=-0.27e-1 ;should be negative
superstep

solve
save NL_80D100.sav

;*** plot commands ***
set output pl_DNWP80_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_DNWP80_dis.emf
plot pen his 2 3 4 5
set output pl_DNWP80_def.emf
plot pen grid mag 0 gr grid mag 100 red
window 700 850 -50 100
set output pl_DNWP80_def_zo.emf
pl pen grid mag 0 gr grid mag 100 red
set output pl_DNWP80_st.emf
plot pen state block

set hisfile his_DNWP80.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1)
o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of
simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of
simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of
simulation'
loop ii (1,igp)
        arr1(ii)=string(x(ii,jgp))
        arr2(ii)=string(y(ii,jgp))
        arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open ('Sur_80D100.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_i
gp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out
ret
```

(f) Section 5.3 Input File

```
new
Title:
Nu-Way Pit: Uniform Settlement of Fill below the Soil Cap

;Uniform settlement applied over the entire length for 40, 60
and 80-foot soil caps
;Model only one-half of the soil cap; No settlement of side
slope
;Use Modulus values revised using Vs consistent with downhole
measurement (880ft/s)
;Run by: TS           Date: Sept 2010

config extra 2
;Initial model for 80 ft soil cap
grid 167,16
model elastic

;Assign coordinates and cut 1:1 side slope
gen same 0. 80. 835. 80. 835. 0.
gen line 80.0,0.0 0.0,80.0
model null region 2 3
group 'null' region 2 3
group delete 'null'

group 'User:new1' notnull
model mohr notnull group 'User:new1'
prop density=3.73 bulk=8.67E6 shear=2.89e6 cohesion=250.0
friction=30.0 &
 dilation=0.0 tension=0.0 notnull group 'User:new1'

fix  x y mark
fix  y i 18 168 j 1
fix  x i 168 j 1 17
set gravity=32.18504
history 999 unbalanced

def nigp
int n_igp
n_igp=igp
end
nigp

;ret
;Solve as elastic material to initialize stresses & then apply
specified soil parameters
solve elastic

def mod_ini
command
ini xd=0
ini yd=0
end_command
```

```
 loop ii (1,izones)
  loop jj (1,jzones)
       if model(ii,jj) # 1 then
        ex_1(ii,jj)=shear_mod(ii,jj)
        state(ii,jj)=0
       endif
  end_loop
 end_loop
end

;Initialize displacements to zero
mod_ini

his 1 yd i=1 j=17    ;check

;G/Gmax Table for Upper Sand (Seed and Idriss, 1970; Idriss,
1990)
table 1 0.,1. 0.000001,1. 0.00000316,1. 0.00001,0.99
0.0000316,0.96 0.0001,0.85 &
0.000316,0.655 0.001,0.37 0.00316,0.19 0.01,0.085 0.1,0.0085

save el_UNWP.sav

;40-foot thick soil cap
________________________________*************************

m n i=1,167 j=1,8
fix  y i 10 168 j 9

his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=9
his 5 yd i=162 j=9

set large

def superstep
float dis_inc udapp
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
 if ns<=ns_ramp
       udapp=float(ns)
 else
       if ns<=(n_step-ns_ramp)
              udapp=float(ns_ramp)
       else
              udapp=float(n_step-ns)
       endif
 endif

              uda=dis_inc*udapp
       loop ii(10,igp)
              yvel(ii,9)=uda
       end_loop
command
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
       shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
       bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 tot_dis=-5.33e-1 ;should be negative
superstep

solve
save NL_40U.sav

;*** plot commands ***
set output pl_UNWP40_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_UNWP40_dis.emf
plot pen his 2 3 4 5
set output pl_UNWP40_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_UNWP40_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_UNWP40_st.emf
plot pen state block

set hisfile his_UNWP40.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1)
o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of
simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of
simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of
simulation'
```


0916.0324

```
loop ii (1,igp)
        arr1(ii)=string(x(ii,jgp))
        arr2(ii)=string(y(ii,jgp))
        arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open('Sur_40U.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_i
gp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out
;ret

;60-foot thick soil cap
________________________________**************************

res el_UNWP.sav

m n i=1,167 j=1,4

fix y i 14 168 j 5

his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=5
his 5 yd i=162 j=5

set large

def superstep
float dis_inc udapp
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
 if ns<=ns_ramp
        udapp=float(ns)
 else
        if ns<=(n_step-ns_ramp)
                udapp=float(ns_ramp)
        else
                udapp=float(n_step-ns)
        endif
 endif

                uda=dis_inc*udapp
        loop ii(14,igp)
                yvel(ii, 5)=uda
        end_loop
command
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
        shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
        bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop

end_loop
end

set n_step=20000 tot_dis=-2.6e-1 ;should be negative
superstep

solve
save NL_60U.sav

;*** plot commands ***
set output pl_UNWP60_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_UNWP60_dis.emf
plot pen his 2 3 4 5
set output pl_UNWP60_def.emf
plot pen grid mag 0 gr grid mag 10 red
window 700 850 -50 100
set output pl_UNWP60_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_UNWP60_st.emf
plot pen state block

set hisfile his_UNWP60.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1)
o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of
simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of
simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of
simulation'
loop ii (1,igp)
        arr1(ii)=string(x(ii,jgp))
        arr2(ii)=string(y(ii,jgp))
        arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open('Sur_60U.out',1,1)
```

```
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_i
gp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out

;80-foot thick soil cap
________________________________*************************

res el_UNWP.sav

his 2 yd i=168 j=17
his 3 yd i=162 j=17
his 4 yd i=168 j=1
his 5 yd i=162 j=1

set large

def superstep
float dis_inc udapp
Int ns_ramp ramp_acc ramp_dec Q
ns_ramp=n_step/5
ramp_acc=ns_ramp*(ns_ramp+1)/2
ramp_dec=ns_ramp*(ns_ramp-1)/2
Q=ramp_acc+ns_ramp*(n_step-2*ns_ramp)+ramp_dec
dis_inc=tot_dis/Q

loop ns (1,n_step)
 if ns<=ns_ramp
        udapp=float(ns)
 else
        if ns<=(n_step-ns_ramp)
                udapp=float(ns_ramp)
        else
                udapp=float(n_step-ns)
        endif
 endif

                uda=dis_inc*udapp
        loop ii(18,igp)
                yvel(ii,1)=uda
        end_loop
command
step 1
end_command

 loop ii (1,izones)
  loop jj (1,jzones)
        shear_mod(ii,jj)=ex_1(ii,jj)*table(1,ssi(ii,jj))
        bulk_mod(ii,jj)=3.*shear_mod(ii,jj)
  end_loop
 end_loop
end_loop
end

set n_step=20000 tot_dis=-0.54e-1 ;should be negative
superstep

solve
save NL_80U.sav

;*** plot commands ***
set output pl_UNWP80_un.emf
set plot emf

;plot name: Unbalanced force
plot pen history 999
set output pl_UNWP80_dis.emf
plot pen his 2 3 4 5
set output pl_UNWP80_def.emf
plot pen grid mag 0 gr grid mag 100 red
window 700 850 -50 100
set output pl_UNWP80_def_zo.emf
pl pen grid mag 0 gr grid mag 10 red
set output pl_UNWP80_st.emf
plot pen state block

set hisfile his_UNWP80.his
his write 1 2 3 5 vs 4 skip 10

def Sur_out
array arr1(n_igp) arr2(n_igp) arr3(n_igp) o_h1(1) o_h2(1)
o_h3(1)
o_h1(1)='X-Coordinate of surface nodes at the end of
simulation'
o_h2(1)='Y-Coordinate of surface nodes at the end of
simulation'
o_h3(1)='Y-Displacement of surface nodes at the end of
simulation'
loop ii (1,igp)
        arr1(ii)=string(x(ii,jgp))
        arr2(ii)=string(y(ii,jgp))
        arr3(ii)=string(ydisp(ii,jgp))
end_loop
oo=open ('Sur_80U.out',1,1)
oo=write(o_h1,1),write(arr1,n_igp),write(o_h2,1),write(arr2,n_i
gp)
oo=write(o_h3,1),write(arr3,n_igp)
oo=close
end
sur_out
ret
```

END

0916.0326

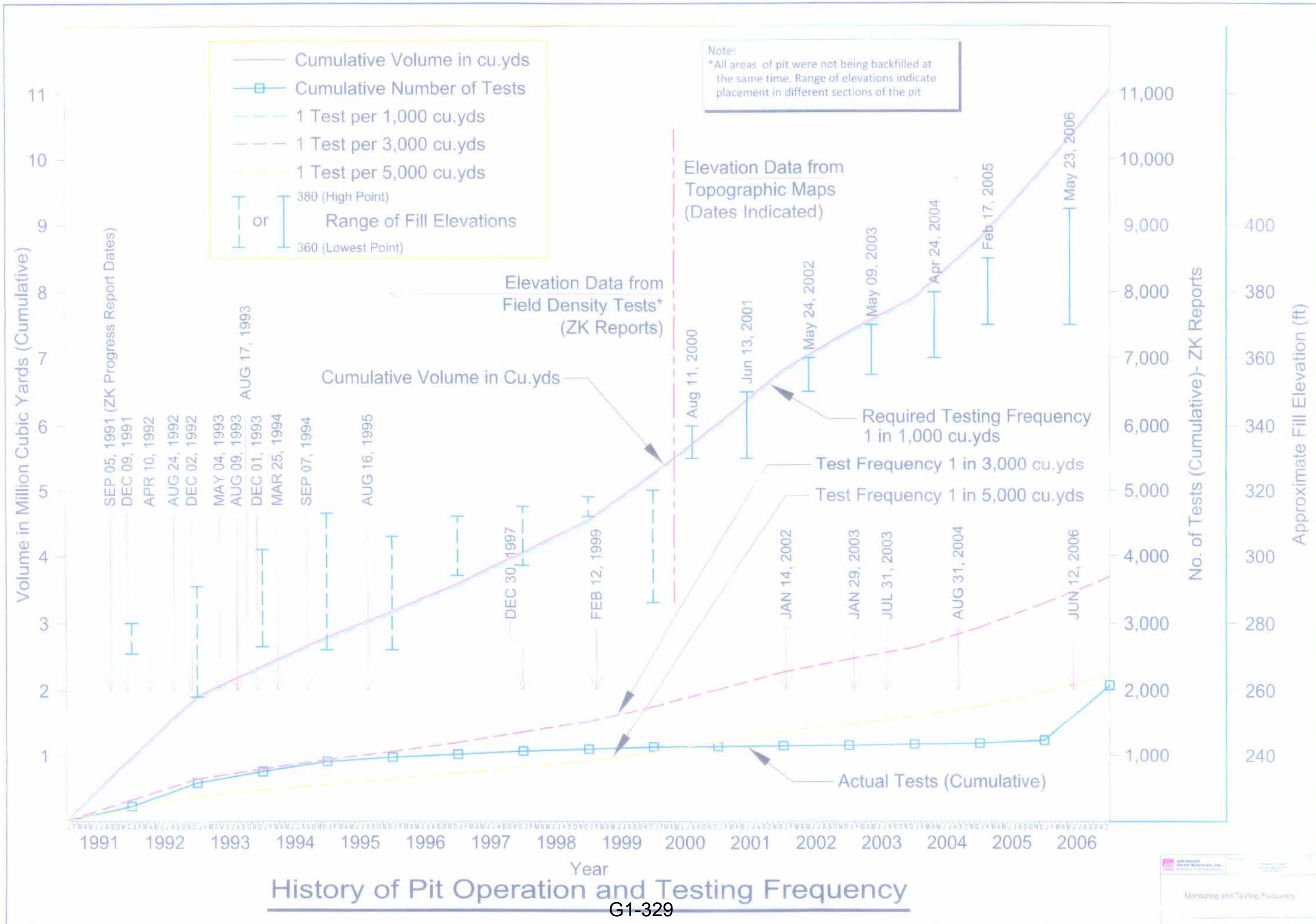
PCL XL error

Subsystem: GE_VECTOR

Error: GEEmptyClipPath Warning: IllegalMediaSize


VISITS 35 1991 TESTS 224
969,186
VISITS 53 1992 TESTS 353
936,463
VISITS 47 1993 TESTS 171
455,103
VISITS 25 1994 TESTS 152
423,698
VISITS 16 1995 TESTS 66
383,582
VISITS 16 1996 TESTS 43
405,624
VISITS 11 1997 TESTS 49
474,021
VISITS 9 1998 TESTS 36
494,909
VISITS 4 1999 TESTS 35
701,599
VISITS 1 2000 TESTS 5
786,123
VISITS 5 2001 TESTS 12
782,786
VISITS 6 2002 TESTS 6
580,909
VISITS 10 2003 TESTS 18
520,451
VISITS 8 2004 TESTS 15
877,326
VISITS 45 2005 TESTS 44
1,083,006
VISITS 265 (ZK 21; HAI 244) 2006 TESTS 844
LEGEND
Notes:
G1-328
Advanced Earth Sciences, Inc.
Pit Backfilling History
Figure 2-1

0916.0328



History of Pit Operation and Testing Frequency

0916.0329

APPENDIX II
EXCEPTS FROM CITY OF IRWINDALE GRADING REQUIREMENTS

(v) Prepare a report evaluating the fill in terms of its ability to support future development of the site with retail, commercial and/or industrial building ranging up to about four stories in height. The report shall specifically address the relative compaction and potential for settlement, voids within the fill, long term hydraulic transportation of fines within the fill and its susceptibility to seismic densification during major earthquakes.

Existing fills found to have been properly compacted and otherwise appropriately placed to reduce settlement and other geotechnical risks to acceptable levels such that it can support the planned site development objectives noted above may remain in place. Existing fills that are found not to be properly compacted or placed, or are otherwise unsuitable to support these development objectives shall either be remediated to meet the development objectives or shall be restricted from any form of development that could be adversely affected by the condition of the existing fill. In any event, any fill placed on the existing fill must be in compliance with these Guidelines.

6.0 BACKFILL MATERIALS

6.1 Composition

The materials used to backfill the pits must have adequate engineering properties to provide suitable structural support for the planned land uses (i.e. buildings and associated site improvements). In addition, the backfill materials must comply with the Inert Debris Engineered Fill Operations (IDEF) permit.

The primary source of backfill materials is expected to be inert construction and demolition debris, such as concrete and soil. Silts and fine sands derived from the local aggregate processing operations also may be a significant component of the backfill. The remainder will probably consist of silts from desilting basins and other miscellaneous sources.

Acceptable fill materials include concrete, tile, masonry brick or block, concrete block, rock, gravel, sand, silt, clay, clay products, glass, ceramics, metals embedded in concrete, and other similar inert materials. Asphaltic concrete may be placed above the high groundwater table as stated in the IDEF permit.

Unacceptable fill materials include those that are hazardous, contaminated, or biodegradable. These include (but are not limited to) organic materials, wood, vegetation, paper, rubber (including tires) plastic, metals not encased in concrete, plaster, wallboard, liquid wastes, and trash. Permits from local water quality control agencies and others may place additional restrictions on materials that may be placed in the fill.

Appropriate quality control measures must be in place to keep unacceptable materials out of the fill. Recommendations for such measures are discussed in Section 9.

Incoming loads of organic materials must be rejected. However, very small amounts of non-hazardous organic materials that happen to be contained in otherwise inorganic loads may be incorporated into the fill. However, even in such cases, there should be active "hand-picking" to remove larger organic materials such as wood and branches. Any organics that are incorporated into the fill may not occupy more than 0.5 percent of any lift by volume.

6.2 Particle Size and Material Processing

Unlike conventional soil fills, the incoming fill materials are expected to be very heterogeneous and contain a large fraction of oversize particles (>12 inches). In addition, some of the incoming material will probably have odd shapes or contain protruding rebar. Thus, material processing will almost certainly be required in order to produce a fill material with acceptable particle sizes which then will be suitable for placement and compaction.

Much of the incoming material will consist of reinforced concrete from demolition projects. This will include slabs, columns, beams, and other structural members, as well as sidewalks, curb and gutter, and other reinforced or unreinforced concrete. Reinforcing steel that is embedded within these concrete objects may be placed in the fill. However, reinforcing steel that is not fully embedded must be removed and hauled off the site.

Occasional large objects, that cannot be crushed or broken, may be buried in the fill as described below, but most oversize materials will need to be broken down to a size that is suitable for placement and compaction in horizontal lifts. Thus, the particle sizes for fills to be placed in lifts shall not exceed the following:

- For flat and elongated particles (aspect ratio >3): 18 inches
- For all other particles: 12 inches

Larger particles must be broken down to meet these criteria, or placed as oversize materials as discussed below.

In order to adequately fill the voids between the larger particles, the fill shall have the following characteristics:

- 30–100 percent by weight is smaller than ¾ inch
- All particles larger than 3 inches are spread apart (i.e. not nested)

The incoming fill materials shall be processed in order to remove the unacceptable materials and to achieve the required particle sizes, particle shapes and blend of materials. The operator shall control these processing operations in order to produce fills that meet these recommendations. Processing shall include particle size reduction through crushing and breaking of oversize particles, removal of deleterious materials, blending of fill materials to achieve acceptable ranges of particle sizes, and moisture conditioning prior to fill placement. Examples of processing are presented in Figure 3.

Embedded reinforcing steel that is not removed from concrete during the processing operations may be placed in the fill so long as it is completely encased in concrete and any exposed bars are cut flush with the concrete.

6.3 Oversize Materials

Occasional sound, inert objects, such as boulders or concrete, that cannot be reduced to the maximum sizes defined in Section 6.2 may be placed in the fill at depths more than 20 feet below finish grade in accordance with Section 7.6.

7.0 BACKFILL PLACEMENT AND COMPACTION

The processed fill materials are then to be placed in horizontal lifts, as described in this section. Oversize materials shall be specially placed using as described in Section 7.6.

7.1 Fill Placement

The processed fill, other than oversize materials, is to be placed as follows:

All fill should be scarified, plowed, disked, ripped and/or bladed until it is uniform in consistency, and free of large unbroken chunks or clods of soil or inert material. Prior to compacting, each lift shall be uniformly moisture conditioned to a moisture content above optimum. The uncompacted lift thickness shall not be greater than 1.5 times the maximum particle size. However, the uncompacted lift thickness need not be less than 8 inches. Each prepared fill lift shall be mechanically compacted to satisfy the requirements of Section 7.3.

Dedicated compaction equipment shall make multiple passes over the entire fill surface as necessary to obtain the required compaction before another lift of fill is placed thereon.

All fill to be placed deeper than 40 feet from the future finished grade shall be compacted to at least 93% relative compaction as described in Section 7.3. Within the upper 40 feet from future finished grade, the fill shall be compacted to at least 90% relative compaction. Fills compacted to less than the specified relative compaction shall be reworked to achieve at least the required minimum relative compaction

7.2 Moisture Control

Moisture contents in equal to or in excess of optimum moisture shall be maintained throughout each fill lift and material type. Materials that are too dry shall be moisture-conditioned by adding water at the time of blending and during compaction. Materials containing excessive moisture shall be dried by aerating, blading, disking, ripping or harrowing to achieve the specified moisture content. Because of the relatively thick lifts, moisture conditioning after the lift is placed will not be sufficient to adequately moisten the entire lift, and thus is not an acceptable procedure.

Silty and clayey soils (including silts from aggregate processing or debris basins) may possess excess moisture content when delivered to the site. Soils that are unworkable because of excessive moisture shall be segregated from materials that do not require special processing. These soils shall be stockpiled in a designated area not within the fill placement area and processed separately to achieve drying under the observation of the geotechnical engineer.

For purposes of these guidelines, all moisture content tests shall be performed on the – ¾ inch fraction of the fill material, and all moisture content specifications are likewise based on the – ¾ inch fraction. This specification is based on the assumption that larger particles are sufficiently moist not to absorb moisture from the fines fraction and thereby lower its moisture content below optimum.

7.3 Compaction Standards

The definition of relative compaction and the associated test methods for measuring it depend on the quantity of large particles in the fill. Thus, the recommended methods for soil fills differ from those for blended rubble fills, as described below. The recommended testing frequency is described in Section 9. In addition to these quantitative tests, the quality assessment efforts shall include monitoring of fill placement and compaction procedures and periodic test pits as described in Section 9.

7.3.1 Soil Fills

Soil fills are those that contain no more than 30% particles greater than ¾-inch in maximum dimension. The relative compaction in such fills is defined as the ratio of the in-place dry density to the maximum dry density determined using ASTM D1557. The in-place dry density shall be measured using ASTM D1556, D4914, or D5030. When applicable, oversize material correction factors shall be applied using ASTM D4718.

7.3.2 Blended Rubble Fills

Blended rubble fills are those that have at least 30 percent of the particles larger than ¾ inch. Such materials cannot be tested using ASTM D1557, so an alternative method is necessary for assessing relative compaction. The recommended methodology is as follows:

1. The relative compaction for blended rubble fills is defined as the ratio of the in-place bulk density to the maximum achievable bulk density.
2. The in-place bulk density shall be determined using ASTM D4914, ASTM D5030, or the procedure described in Appendix B.
3. The maximum achievable bulk density for each fill material shall be determined by constructing one or more field test pads, as described in Appendix A.

7.4 Fill Slopes

The compaction standards shall apply to any final fill slope face. Fill slopes shall be no steeper than 2:1 (horizontal:vertical), and must be safe for the intended use. Temporary or interim fill slopes will be cut to remove loose material to expose fully compacted fill through the benching for future fills placed against the former fill slope.

7.5 Benching

Horizontal benches shall be cut into existing slopes to expose competent native materials or properly compacted fill.

7.6 Placement of Oversized Material

Oversize material, as defined in Section 6.3, may be placed in a trench to form windrows. These windrows shall be as wide and deep as necessary to accommodate the oversize material in a single layer (i.e. individual particles shall not be stacked vertically in the trench). Granular soils with a sand equivalent (SE) of at least 30 shall be jetted or otherwise placed between the oversize materials to fill the voids. Details on these windrows are shown in Figure 4.

8.0 INSTRUMENTATION

Settlement monuments shall be installed within the fill as it is being placed and as the fill elevation advances. Settlement monuments shall be installed in accordance with a design prepared by the geotechnical engineer, or as provided in Appendix C. Settlement monuments destroyed by grading operations shall be reestablished to their condition and depth prior to damage. Conventional settlement plates are not recommended because of their susceptibility to base damage during the inert fill operation. A more durable alternative is shown in Appendix C.

The settlement monuments shall be surveyed by a licensed land surveyor, to an accuracy of 0.01 ft horizontally and vertically initially, and to 0.10 ft. horizontally and 0.01 ft. vertically thereafter. The frequency of such surveys will be such that the vertical difference between any two successive surveys for each monument is not more than a few one-hundredths of a foot, once each three months, or at such time as the monument pipe needs to be extended, whichever occurs first. See Appendix C for more information on settlement monuments and survey data recordation.

9.0 QUALITY CONTROL AND QUALITY ASSURANCE

The landfill owner is responsible for placing the fill in conformance with these Guidelines and must have appropriate quality control procedures in place to promote uniform compliance. In some cases this responsibility might be delegated to the operator. The owner also is responsible for retaining a geotechnical engineer, who provides verification through a quality assurance process.

9.1 Quality Control

Quality Control for the day-to-day landfill operations to maintain compliance with these Guidelines is the responsibility of the landfill operator. It is recommended that the operator investigate multiple methods and procedures to accomplish the necessary processing, crushing, sorting, removal of deleterious materials from fill materials, blending, moisture conditioning and compaction of the fill materials to meet these Guidelines. Once such procedures are found that achieve the intended goals and their relative cost and ease of execution are known, then the consistent application of such procedures through the day-to-day processing of the fill is likely to achieve the highest level of Quality Control. In almost all instances a high level of Quality Control will result in a companion lower level of "failures" during the Quality Assurance program conducted by the geotechnical engineer. Therefore, well planned and systematic operations will likely have the highest level of Quality Assurance results, and the lowest incidents of reworking large volumes of fill that fail to meet the provisions of these Guidelines. It should be noted that in an attempt to minimize the cost of the Quality Assurance program conducted under the direction of the geotechnical engineer, the frequency of testing relative to the inert rubble fill has been reduced considerably in comparison to conventional earthwork QA programs. Therefore the volume of material placed as compacted fill which is represented by each QA test is significantly larger than would be the case for conventional earthwork fills. Consequently, the volume of fill that will have to be reworked by the Operator when QA test indicate noncompliance may be significantly higher than in conventional earthwork situations. Therefore, the cost of the rework will be proportionately higher as well. It is then in the Operator's interest to develop and maintain an effective Quality Control program at all time during active landfill operations.

9.2 Quality Assurance

The geotechnical engineer is responsible for the Quality Assurance program for the fill work. Field and laboratory testing in combination with field observations, surveys and other data collection by the geotechnical engineer will be required to document compliance with these Guidelines and to form the basis of the certification of the fill in accordance with the California Professional Engineers Act (BPC §6735.5). The recommended schedule of quality assurance activities is presented in Table 1. The horizontal and vertical locations of all tests and observations shall be accurately determined and recorded. This effort will require vertical and horizontal control by a licensed surveyor.

When the quality assurance activities indicate the fill is not in conformance with these Guidelines, the fill shall be reworked by re-blending and recompacting or other work as needed, then retesting to demonstrate conformance before additional fill is placed thereon.

TABLE 1
SCHEDULE OF QUALITY ASSURANCE ACTIVITIES

Field Test or Observation	Test Method	Minimum Testing Frequency	
		Soil Fill Note 1	Blended Rubble Fill Note 2
Observation of Filling Process	N.A.	Sufficient to provide the basis for an opinion on conformance of the filling procedures with these Guidelines	
Test Pits or Trenches	Visual assessment of fill materials and voids in accordance with Section 9.2.4	N.A.	One per 5,000 cy, two per lift, or one per week, whichever is more frequent
Field Density and Compaction	See Note 3	One per 1,000 cy or one per each two lifts	One per 20,000 cy, one per ea. 2 weeks, or per 2 lifts
Gradation	See Note 4	One per 100,000 cy or one per month	One per 100,000 cy or one per month
Moisture Content	Shall be determined only on sizes up to and including ¾ inch	One per field density test, and as needed to guide fill placement	One per 5,000 cy or one per week

Notes:

1. Soil fill has no more than 30% particles larger than ¾ inch.
2. Blended rubble fill has more than 30% particles larger than ¾ inch.
3. Field density tests on soil fill may include sand cone (ASTM D1556) or nuclear gauge (ASTM D 2922). Maximum densities for soil fill shall be determined by ASTM D1557. Field density tests for blended rubble fill shall be determined by large ring tests conducted in accordance with ASTM D4914, ASTM D 5030 or the provisions of Appendix B. Maximum achievable density data for blended rubble fill shall be determined from test pad fills in accordance with Appendix A.
4. Gradation for soil fills and blended rubble fills shall be established by the methods given in ASTM D422, and the Field Bulk Gradation Method described in Appendix B, respectively. See also Section 7.3.

9.2.1 Regular Site Visits, Observations and Testing

The geotechnical engineer shall visit the fill site regularly both on a scheduled and unscheduled basis. The City may accompany the geotechnical engineer on these visits, or may conduct independent visits. During such site visits the geotechnical engineer shall, at a minimum, conduct the following activities:

- Review the site inspection process which checks incoming truck loads for acceptability, and determine the current running average rate of import material entering the site. The rate of import so determined will assist in scheduling the frequency of site visitations.
- Record the area within the site where active processing and filling activities are occurring (horizontally and vertically).
- Observe the placement of import material, before processing, for any indications of materials not allowed within the fill and observe material processing, blending and moisture conditioning procedures and the resultant fill material for compliance with these Guidelines, including estimations of the combined gradation of the blended fill material.
- Inspect all instrumentation for proper maintenance and protection from operational equipment, obtain technical readings and data as appropriate and record same.

- Record all site visit observations, data, information and recommendations and communicate same to the operator before departing the site.

The geotechnical engineer shall be on-site for sufficient time and frequency to provide the information needed to render professional opinions on conformance with these Guidelines. At a minimum, this level of effort is expected to require site visits once per week or with sufficient frequency to conduct the tests and observations described in Table 1, whichever is more frequent. In some cases, more frequent visits or full-time observation and testing may be necessary. The geotechnical engineer shall confirm that all materials represented by one such test have been processed, placed and compacted in the same way as the material actually tested. This confirmation shall include timely observation of the fill process, observation of test trenches within multiple areas of the represented fill and observation of complete coverage of the fill area with appropriate compaction equipment

The Committee has established the field test and observations frequencies in Table 1 above based on its collective best experience and judgment relative to the operations being considered. The field test and observation frequencies presented in the above, may be increased or decreased based on the technical review, assessment and recommendations of the geotechnical engineer, however any such change will require the review and approval of the City of Irwindale and the Irwindale Backfilling Committee.

Representative samples of the blended material shall be tested for gradation to determine the overall material "type" in comparison to characteristics determined per Appendix A. Similarly, additional observations in trenches or excavations within the compacted fill shall be conducted to evaluate the characteristics of the completed fill. Where the fill is deemed to be poorly-graded, or if voids are present, or if minimum compaction is not achieved, the fill shall be reworked by re-blending and recompacting it in-place before additional fill is placed thereon.

9.2.2 Test Pits

Periodic test pits shall be excavated into compacted blended rubble fills as indicated in Table 1, and the geotechnical engineer shall visually assess exposed fill in these test pits. This assessment shall include checking for the following:

- The presence or absence of discernable voids between rubble particles. The geotechnical engineer should not consider as voids any inclusions that have been created around rubble particles that have been disturbed or dislodged by the excavation process.
- Conformance with the material composition requirements of Section 6.1.
- The percentage of organics or other deleterious materials (Section 6.1), the maximum particle size, and the percentage smaller than ¾ inch.
- The presence or absence of nesting.

The results of these efforts must be documented in writing and with photographs of the pit walls.

The geotechnical engineer also shall perform similar visual assessments in all pits excavated for large-scale field density tests.

9.2.3 Field Density and Compaction Testing

The field density and compaction shall be tested in accordance with Section 7.3. In addition, sufficient observations and grain-size tests shall be performed in order to properly select the maximum achievable density.

9.2.4 Gradation Testing

The grain-size distribution for soil fills shall be determined using the following test methods:

- For soil fills: ASTM D422
- For blended rubble fills: Bulk field gradation test, as described in Appendix B

9.2.5 Moisture Content Testing

Moisture content tests shall be conducted in accordance with Section 7.2.

9.2.6 Reporting

All project records shall be preserved in a long-term record retention system as described in more detail in Section 11. The intent is to produce a comprehensive history of the construction of the fills. This shall include a record of observations and test results, as well as summaries that permit conclusions to be drawn regarding compliance with or deviations from these Guidelines. In general, the data and records produced during the filling process may be categorized as follows:

- **Operator's Records** representing the import of inert material to the site to be used as fill, including rate of import such as cubic yards/day, month and year, material visual classification upon arrival at the entry inspection station such as material composition, and other documents such as site photos of work in progress, aerial photos, topographic maps, other field surveys and all quality control records.

- **Quality Assurance Records** representing the physical characteristics of the imported material to be used as fill, including all gradation and density test results, observations, recommendations, assessments of processing and blending procedures, corrective measures, instrumentation data, records of regular site visits, site photos of work in progress, interim reports and all other data produced and/or recorded by the geotechnical engineer in conjunction with the filling process. Much of the material to be used as fill will have a variable composition and particle sizes. The anticipated high percentage of larger sizes will dictate extensive observation to support the test data. Reporting shall include a summary of field classifications on the basis of material categories defined in Section 6. In addition, reports shall include detailed description information or the location and character of oversized materials placed in windrows within the fill, and observational data on the composition and condition of the fills.

- **Geotechnical Engineer's Records and Reports** representing the assimilation of the above data and records of the operator and the QA processes. The geotechnical engineer's records will include all data and analysis derived from field tests, settlement monuments, and all other technical evaluations and analysis. These records will also include all interim, periodic and final geotechnical reports covering the subject fill.

The above records and data shall be presented in formal reports categorized as follows with the information and intent as described:

Quarterly Report: These reports will be prepared by the geotechnical engineer based on the accumulated data in the Owner's and Operator's records and geotechnical engineer's QA and site visitation records created since any preceding quarterly report. The quarterly report's main functions will be to state that the fill covered by the report was or was not placed in accordance with the Guidelines recommendations; discuss remedies employed to correct any non-compliant fill and identify and discuss any unexpected data or fill performance, or any problematic issues associated with fill conditions and update any such issues cited in the previous quarterly report. The quarterly reports will also serve as the primary tool to compile basic data, test results and observations into complete and self-explanatory long-term records. The reports will be signed by the geotechnical engineer and delivered to the operator on a quarterly basis.

Photographs, observations, and test data shall be included in appendices. The text of the reports shall contain a description of the fill placed during the current quarter along with summary information on the following:

1. Vertical and horizontal extent of fill.
2. Range and average lift thickness.
3. Fill composition and grain size.
4. Placement moisture content of soil and blended rubble fill gradations.
5. Range and average field density values, listed by test method.
6. Discussion of material categories.
7. Location, dimensions and composition of windrows.
8. Settlement.
9. Check list of incomplete work and a schedule for completion.

10. Identification of the location of all field tests, trenches, including any test not in compliance with these guidelines, and records of lift thickness.

Data from items 2, 3, 4, and 5 above should also be shown on histograms or other graphs illustrating the statistical distribution of test results as well as in conventional tabular and location map forms.

Annual Report: These reports, which will be in lieu of the last quarterly report of any year, will be prepared by the geotechnical engineer, based on the information presented in the previous quarterly reports and the new information since the last quarterly report. The annual reports will draw conclusion as to the compliance of the fill with the Guidelines recommendations, identify any condition that should be investigated further, and assess the current status of the fill and its performance to date. It shall include sections on fill characteristics and settlement analysis. The annual report shall provide a summary of the previous quarterly reports, but will also contain the quarterly reports in the appendix of the annual report. Annual reports shall include a discussion section to address the perceived success or shortcomings of the backfill to date. The annual reports shall also review and update the site specific Fill Plan described in Section 5 to assess if any modifications are appropriate. Additionally, in regard to creating and maintaining long-term records, the annual reports shall include all accumulated data, records, test results and other information to date in an orderly and self explanatory presentation. The annual reports will contain all such date in written form as well as in the current state of the art electronic format. However, no basic information or data will be retained only in electronic form. The annual report shall be signed and certified by the geotechnical engineer that all fill has been placed in accordance with the Site Specific Fill Plan and these Guidelines and shall be delivered to the owner. The owner shall provide a copy of the annual report to the City within 30 days of its receipt of the report from the geotechnical engineer.

Milestone and Final Reports: These reports will be developed and presented to the owner and the City at significant milestone achievements in the backfilling process and at the completion of the fill. These reports will be a technical accumulation of all previous annual reports, including all base data and quarterly reports. In addition to the subjects and information provided in the annual reports, these reports will also include the analysis of the entire above water and underwater fills with respect to the intended end use of the property and shall make technical predictions of performance of the fill, based on all records and reports above, etc. The milestone and final reports shall contain a certification (see California Professional Engineers Act) by the geotechnical engineer that the placement of the fill is in accordance with the Guidelines. The report shall be delivered to the owner, who will provide a copy to the City within 30 days thereafter. The scope of work and schedule for the milestone and/or final reports shall be agreed upon by the owner and the City before the geotechnical engineer commences such work.

The owner, operator, geotechnical engineer and City shall each maintain all of the above reports in a secured long term record retention system that shall survive the completion of the fill and the completion of all initial developments as presented in more detail in Section 11.

tress fills designed pursuant to the provisions of Section 1819 or Section 7015 of this Code.

Exception: Where special conditions warrant, the Department may approve slopes steeper than the bedding planes if the applicant shows through investigation, subsurface exploration, analysis and report by both a soils engineer and an engineering geologist, to the Department's satisfaction, that the slopes will have a factor of safety against sliding of not less than 1.5 for static loads.

Whenever grading at the top of any natural or manufactured slope exposes soil or bedrock material that will allow the infiltration of water in a manner that would adversely affect the stability of the slope, the exposed area shall be capped with a relatively impervious compacted soil blanket seal having a minimum thickness of 2 feet (610 mm). The soils engineer shall certify in writing that the blanket seal is adequate to reduce water infiltration to permissible levels.

7010.3 Top of cut slope. The top of cut slopes shall not be made nearer to a site boundary line than one fifth of the vertical height of cut with a minimum of 2 feet (610 mm) and a maximum horizontal distance of 10 feet (3048 mm). The setback may need to be increased for any required interceptor drains. Setback dimensions shall be horizontal distances measured perpendicular to the site boundary. Setback dimensions shall be as shown in Figure E of this chapter.

SECTION 7011
FILLS

7011.1 Height. No fill slope shall exceed a vertical height of 100 feet (30 480 mm) unless horizontal benches with a minimum width of 20 feet (6096 mm), as shown in Figure D are installed at each 100 feet (30 480 mm) of vertical height.

7011.2 Slope. No fill shall be made which creates an exposed surface steeper than one unit vertical in two units horizontal (50-percent slope). The fill slopes abutting and above public property shall be placed so that no portion of the fill lies above a plane through a public property line extending upward at a slope of one unit vertical in two units horizontal (50-percent slope).

Exception: The Department or the Board in case an appeal is made to it under Section 105 may permit a fill to be made which creates an exposed surface steeper in slope than one unit vertical in two units horizontal (50-percent slope), provided:

1. The use of the steeper slope is determined to be necessary due to special design limitations on the site,
2. The gradient does not exceed one unit vertical in one and one-half units horizontal (66.7-percent slope) and
3. The applicant shows through investigation, subsurface exploration, analysis and report by both a soils engineer and an engineering geologist, to the Department's satisfaction, that the fill to be used and the underlying bedrock or soil supporting the fill have strength characteristics sufficient to produce a stable slope with a minimum factor of safety not less than 1.5 for static loads. The soils engineer shall verify by necessary testing and observation and shall certify attainment of the required strength characteristics in the fill materials as specified in the approved report.

7011.3 Compaction. All manufactured fills shall be placed on natural undisturbed material or approved compacted fill. Fills shall be compacted throughout their full extent to a minimum relative compaction of 90 percent of maximum dry density within 40 feet (1219 mm) below finish grade and 93 percent of maximum dry density deeper than 40 feet (1219 mm) below finish grade, unless a lower relative compaction (not less than 90 percent of maximum dry density) is justified by the soils engineer. The relative compaction shall be determined by ASTM soil D 1557. Every manufactured fill shall be tested for relative compaction by a soil testing agency approved by the Department. A compaction report including a Certificate of Compliance setting forth densities so determined shall be submitted to the Department for review before approval of any fill is given. For slopes to be constructed with an exposed slope surface steeper than two horizontal to one vertical, compaction at the exposed surface of the slope shall be obtained either by overfilling and cutting back the slope surface until the compacted inner core is exposed, or by compacting the outer horizontal 10 feet (3048 mm) of the slope at least 92 percent of relative compaction.

Prior to permitting building on deep fills, the Department may require the determination of the settlement characteristics of the fills to establish that any movements have substantially ceased. In those cases, a system of benchmarks shall be installed at critical points on the fill and accurate measurement of both horizontal and vertical movements shall be taken for a period of time sufficient to define the settlement behavior. In no case shall the period of time be less than 1 year, with at least four consecutive checks made at intervals of 3 months.

Exceptions:

1. The Department may approve uncompacted fill in self-contained areas where the fills are not to be used to support buildings or structures and no hazard will be created.
2. Fill material placed in areas within cemeteries used or to be used for internment sites shall be compacted to a minimum of 80 percent, unless the fill is placed on a slope steeper than three horizontal to one vertical, or placed on slopes adjacent to public properties or private properties in separate ownership, or is to be used to support buildings or structures, in which cases it shall be compacted to a minimum of 90 percent.
3. Compaction report is not required for gravel backfill behind retaining walls provided the following conditions are met:
   - A. The retaining wall does not exceed 10 feet (3048 mm) in height.
   - B. The maximum distance between the retaining wall and the backcut shall not exceed 24 inches (610 mm).
   - C. The gravel backfill shall be mechanically compacted and covered with concrete pavement or be capped with a 24-inch thick soil blanket

APPENDIX III
REPORT BY GEOVISION

GEOVision
geophysical services

REPORT

GEOPHYSICAL INVESTIGATION

Nu-Way Reclaimed Aggregate Mine
Irwindale, California

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Report 7629-01

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1 INTRODUCTION

In-situ seismic measurements using active and passive surface wave techniques were performed at the Nu-Way Reclaimed Aggregate Mine located in Irwindale, California on January 2 and 15-18, 2008 and February 8, 2008. The purpose of this investigation was to characterize shear (S) wave velocity structure to a minimum depth of 60 meters (200 ft), to be used for site characterization. Of particular interest, was the lateral variability of subsurface velocity across the site and presence of thick, low velocity silt layers in subsurface sediments.

The active and passive surface wave techniques utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) and refraction (linear array) microtremor methods, respectively. A total of 15 surface wave soundings were made along three profiles and a borehole on site, as shown in Figure 1.

The lateral variability of subsurface velocity structure was evaluated by direct comparison of shear (S) wave velocity models and comparison of the average S-wave velocity of the upper 30 m (V_S30) and 60 m (V_S60). V_S30 is also used in the Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design . The average shear wave velocity of the upper 100ft is used in the International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – $V_S30 > 1500$ m/s (UBC) or $V_S100 > 5{,}000$fps (IBC)
Class B – rock – $760 < V_S30 \leq 1500$ m/s (UBC) or $2{,}500 < V_S100 \leq 5{,}000$fps (IBC)
Class C – very dense soil and soft rock – $360 < V_S30 \leq 760$ m/s (UBC)
or $1{,}200 < V_S100 \leq 2{,}500$fps (IBC)
Class D – stiff soil – $180 < V_S30 \leq 360$ m/s (UBC) or $600 < V_S100 \leq 1{,}200$fps (IBC)
Class E – soft soil – $V_S30 < 180$ m/s (UBC) or $V_S100 < 600$fps (IBC)
Class F – soils requiring site-specific evaluation

This report contains the results of the surface wave investigation conducted at the site. An overview of the surface wave method is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Interpretation is presented in Section 5 and Section 6 presents our conclusions. References and our professional certification are presented in Sections 7 and 8, respectively.

2 METHODOLOGY

Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the refraction and array microtremor methods. The active and passive surface wave techniques utilized during this investigation consisted of the MASW and refraction (linear array) microtremor methods. A discussion of active and passive surface wave methods is provided in the technical note included as Appendix A.

The basis of surface wave methods is the dispersive characteristic of Rayleigh waves when propagating in a layered medium. The phase velocity, V_R, depends primarily on the material properties (V_S, mass density and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. Waves of different wavelengths, λ, (or frequencies, f) sample different depths. As a result of the variance in the shear stiffness of the layers, waves with different wavelengths travel at different phase velocities; hence, dispersion. A surface wave dispersion curve, or dispersion curve for short, is the variation of V_R with λ or f.

The MASW method is an in-situ seismic method for determining shear wave velocity (V_S) profiles (Park et al., 1999a and 1999b, Foti, 2000). Surface wave techniques are non-invasive and non-destructive, with all testing performed on the ground surface at strain levels in the soil in the elastic range (< 0.001%). MASW testing consists of collecting multi-channel seismic data in the field and applying a wavefield transform to obtain the dispersion curve and data modeling.

A detailed description of the MASW method is given by Park, 1999a and 1999b. Ground motions are recorded by 24, or more, geophones spaced 1 to 2 m apart and aligned in a linear array and connected to a seismograph. A wavefield transform, such as the f-k or τ-p transform, is applied to the time history data to isolate the surface wave dispersion curve. The software packages Pickwin95/WavEq, developed by Oyo Corporation, or Surfseis, developed by the Kansas Geological Survey, are typically used to process the MASW data and obtain the dispersion curve.

The refraction microtremor technique is a passive surface wave technique developed by Dr. John Louie at University of Nevada, Reno. A detailed description of this technique can be found in Louie, 2001. The refraction microtremor method differs from the more established array microtremor technique in that it uses a linear receiver array rather than a triangular or circular array. Unlike the MASW method, which uses an active energy source (i.e. hammer), the microtremor technique records background noise emanating from ocean wave activity, traffic, industrial activity, construction, etc. Refraction microtremor field procedures consist of laying out a linear array of 24, 4.5 to 8 Hz geophones and recording 10, or more, 15 to 60 second noise records. These noise records are reduced using the software package SeisOpt® ReMi™ v2.0 by Optim™ Software and Data Services. This package is used to generate and combine the slowness (p) – frequency (f) transform of the noise records. The surface wave dispersion curve is picked at the lower envelope of the surface wave energy identified in the p-f spectrum. The surface wave dispersion curve can also be estimated using the spatial autocorrelation (SPAC) technique (Okada, 2003), as implemented in the Pickwin95 software package, or equivalent.

The active and passive surface wave techniques compliment one another as outlined below:

- SASW/MASW techniques image the shallow velocity structure which cannot be imaged by the microtremor technique.
- Microtremor techniques work best in noisy environments where SASW/MASW depth investigation may be limited.
- In a noisy environment the microtremor technique will usually extend the depth of an SASW/MASW sounding.
- The degree of fit in the overlapping portion of the dispersion curves from the two techniques provides a level of confidence in the results.

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled. The software packages WinSASW V1 or V2, originally developed at the University of Texas, Austin, Surfseis, or WavEq are used to model the data, whereby through iterative forward and/or inverse modeling, a V_S profile is found whose theoretical dispersion curve is a close fit to the field data.

The final model profile is assumed to represent actual site conditions. Several options exist for forward modeling: a formulation that takes into account only fundamental-mode Rayleigh wave motion (called the 2-D solution), and one that includes all stress waves and incorporates receiver geometry (3-D solution) [Roesset et al., 1991].

The theoretical model used to interpret the dispersion assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good "global" estimate of the material properties along the array. The results may be more representative of the site than a borehole "point" estimate.

3 FIELD PROCEDURES

3.1 Survey Control

Three seismic lines (Lines A to C) were established and surveyed by **GEO***Vision* using a Nikon total station system and Sokkia Axis 3 GPS system with OmniStar differential corrections. The locations of the seismic profiles are shown on Figure 1. Surface wave station centers were established at 72 m (236 ft) intervals along each line. All geophone and shot point locations were measured using a 100-meter tape measure. The endpoints of each seismic line were surveyed using a Sokkia Axis 3 sub-meter GPS system. Relative elevations of each seismic line were surveyed using the total station system and converted to approximate true elevation using elevations using previously staked elevation control on site.

3.2 Surface Wave Survey

A typical MASW field layout is shown in Appendix A. MASW equipment used during this investigation consisted of Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones mounted at 1 m intervals on a landstreamer, seismic cable with 1 m takeouts, a 3 lb hammer, 10 lb sledge hammer and aluminum plate and an accelerated weight drop (AWD). MASW data was acquired along a linear array of 48 geophones spaced 1 m (3.3 ft) apart. Shot points were typically located 1, 3 and 10 m (3.3, 9.8 and 32.8 ft) from the end geophone locations. The 3-lb hammer, 10 lb sledge hammer and AWD were used for the 1, 3, and 10 m offset source locations, respectively. Data from the transient impacts (hammers and AWD) were averaged 5 to 10 times to improve the signal-to-noise ratio. Surface waves were monitored by 48 Oyo Geospace 4.5 Hz geophones and recorded by a Geometrics Geode signal enhancement seismograph. Photographs of typical MASW equipment are presented in Figure 2 and Appendix A. All field data was saved to hard disk and documented in a field notebook.

Refraction microtremor (linear array passive surface wave) measurements were made along a linear array of 24, 4.5 Hz geophones with a 6 m (20 ft) geophone spacing centered at each MASW station. The 72 m (236 ft) spacing between consecutive measurement stations, allowed the passive surface wave array to move from station to station by moving (rolling) 12 geophones. A typical field layout is shown in Appendix A. A Geometrics Geode, 24 bit, 24-channel seismic recording system was used to record twenty 30.96 s noise records using a 2 ms sample rate. Photographs of typical refraction microtremor equipment are presented in Figure 2 and Appendix A. All field data were stored on a laptop computer for later processing.

Prior to acquiring surface wave data along the three profiles, MASW and refraction microtremor (linear array passive surface wave) data were collected adjacent to Borehole B-4. The location of this array (SW-B4) is shown on Figure 1. During preliminary acquisition of this array, it was observed that a large noise source was originating from the asphalt plant north of the site. Inquiries revealed that the noise source was a rock crusher that typically operated daily until 1 pm. All field data was, therefore, acquired after 1 pm, when the rock crusher was not in operation.

4 DATA REDUCTION AND MODELING

The MASW data were reduced using the software PICKWIN95 developed by Oyo Corporation and the following steps:

- Input seismic record into software.
- Enter receiver spacing, geometry and wavelength restrictions, as necessary.
- Apply wavefield transform to seismic record to convert the data to phase velocity – frequency space.
- Identify and pick dispersion curve.
- Repeat for all shot records and merge dispersion curves.
- Convert dispersion curves to WinSASW format for modeling.

The refraction microtremor data were reduced using the Optim™ Software and Data Services SeisOpt® ReMi™ v2.0 data analysis package. Data reduction steps included the following:

- Conversion of SEG-2 format field files to SEG-Y format.
- Data preprocessing which includes trace-equalization gaining and DC offset removal.
- Erasing receiver geometry present in the file header.
- Computing the velocity spectrum of each record by p-f transformation.
- Combining the individual p-f transforms into one image.
- Picking and saving the velocity spectrum image.
- Conversion of the dispersion curve to WinSASW format.

The refraction microtremor data also reduced using the software PICKWIN95 developed by Oyo Corporation and the following steps:

- Input all seismic records into software.
- Enter receiver spacing, geometry and wavelength restrictions, as necessary.
- Calculate the SPAC function for each seismic record and average.
- For each frequency calculate the degree of fit of a first-order Bessel function to the SPAC function for a multitude of phase velocities.
- Identify and pick dispersion curve as the best fit of the Bessel function for each frequency.
- Convert dispersion curves to WinSASW format for modeling

The surface wave dispersion curves from the active and passive surface wave data were combined and an iterative forward modeling process was used to generate an S-wave velocity model for the sounding. During this process an initial velocity model was generated based on general characteristics of the dispersion curve. The theoretical dispersion curve was then generated using the 2-D modeling algorithm (fundamental mode Rayleigh wave dispersion module) and compared to the field dispersion curve. Adjustments are then made to the thickness and velocities of each layer and the process repeated until an acceptable fit to the field data is obtained.

Constant mass density values of 1.9 to 2.1 g/cc were used in the profile. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible effect on surface wave dispersion. During modeling the compression wave velocity, V_P, was estimated using a Poisson's ratio, ν, of 0.33 and the relationship:

$$V_P = V_S\,[(2(1-\nu))/(1-2\nu)]^{0.5}.$$

5 INTERPRETATION

5.1 *Surface Wave Array SW-B4*

An active and passive surface wave sounding was conducted near Borehole B-4 (SW-B4 on Figure 1) to constrain modeling of surface wave data acquired along Lines A to C. Uncorrected Becker blow count data, which is expected to have some correlation to S-wave velocity, was available to help constrain modeling of the surface wave data.

The fit of the theoretical surface wave dispersion curve to the experimental data collected along this array and the modeled S-wave velocity profile are presented as Figure 3 and in Appendix B as Figure B-1. The tabulated S-wave velocity model for this array is presented in Tables 1. The resolution decreases gradually with depth, because of loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The S-wave velocity model is expected to be valid to a depth of 75 to 100 m (246 to 328 ft). The surface wave phase velocities from the passive surface wave measurements are generally in good agreement with those from the MASW data in the region of overlapping wavelength. Some difference in the active and passive surface wave dispersion curves is expected because the passive surface wave data is averaged over the 138 m (453 ft) geophone array, whereas the MASW dispersion curve is averaged over a 47 m (154 ft) geophone array at the center of the passive array.

A zone of lower blow counts was observed from a depth of 164 to 190 ft while drilling Borehole B-4. Lower S-wave velocities would be expected in this zone and this constraint was used while modeling the surface wave data. The nature of the surface wave dispersion curve indicates that a low velocity zone, if present, is relatively thin relative to its depth. The evidence in the surface wave dispersion curve of a thin velocity inversion is the change in curvature of the dispersion curve between wavelengths of 60 and 120 m (200 and 400 ft). A 10 m (33 ft) thick layer with approximate V_S of 350 m/s (1,148 ft/s) between layers with V_S of 550 m/s (1,805 ft/s) is used to model the dispersion curve through this area. The low velocity layer is too thin, relative to its depth, to accurately model and models with a thinner, lower velocity layer or thicker, higher velocity layer could equally well fit the dispersion data.

V_S30 and V_S60 for this array are 377 m/s (1,238 ft/s) and 423 m/s (1,387 ft/s), respectively. According to the UBC and IBC, the area in the vicinity of this array is classified as C, very stiff soil and soft rock.

5.2 *Line A*

Active and passive surface wave soundings were conducted at five (5) locations along Line A as shown in Figure 1. These 1-D surface wave soundings were centered at Stations 226, 463, 699, 935 and 1171 ft. MASW data could not be acquired at Station 1171 ft due to ponded water, therefore the MASW data was acquired further to the north at Station 1340 ft.

The fit of the theoretical surface wave dispersion curve to the experimental data collected along this line and the modeled S-wave velocity profiles are presented in Appendix B as Figures B-2 to B-6. Tabulated S-wave velocity models for these arrays are presented in Tables 2 to 7, respectively. The S-wave velocity models are expected to be valid to a depth of 75 to 100 m (246 to 328 ft). A potential velocity inversion identified in Borehole B-4 blow count data and modeled in surface wave array SW-B4 was added to surface wave models where supported by the characteristics of the dispersion curve. This velocity inversion is poorly constrained because the layer has small thickness relative to its depth, and appears to pinch out or become too thin to image south of Station 699 ft.

Except for Station 1171/1340 ft, the surface wave phase velocities from the passive surface wave measurements are in good agreement with those from the MASW data in the region of overlapping wavelength. The MASW data collected at Station 1340 ft yielded lower surface wave phase velocities over the 30 to 60 m wavelength range than the passive surface wave data centered at Station 1171 ft. The active and passive surface wave data collected in the same area on Line B yielded similar dispersion data. To better understand this problem an additional passive surface wave sounding was acquired through Station 1340 ft on Lines A and B. The passive surface wave sounding was conducted in the early evening when operations at the asphalt plant to the north were at a minimum. Although, not very reliable due to highly directional noise sources from the north, the dispersion curve at this location supports the MASW dispersion data collected at Station 1340 ft. There are three explanations for the difference in the dispersion curves between Stations 1171 and 1340 ft: lateral velocity variation; higher mode surface waves caused by velocity inversion in the passive data collected at Station 1171 ft, or data degradation due to high noise levels from the asphalt plant. For discussion purposes, we assume that the dispersion curve differences are the result of lateral velocity variation, and two models were generated for sounding. Model 1 fits the MASW and passive surface wave data at Station 1340 ft and longer wavelengths of the passive data at Station 1171 ft. A velocity inversion at depth is not needed to fit this dispersion curve but is placed in the model to remain consistent with Model 2. Model 2 fits the small wavelength data at Station 1340 ft and passive data at Station 1171 ft. Models 1 and 2 are assumed to be representative of the average subsurface velocity structure at Stations 1340 and 1171 ft, respectively.

A comparison of the surface wave velocity models along Line A, both on a single plot and in cross section format, are presented as Figures 4 and 5. The models exhibit similar velocity structure except for the velocity inversion near an elevation of 76 m(249 ft), which pinches out south of Station 699 ft and the possible lower velocity sediments between an elevation of 98 and 111 m (322 and 365 ft). V_S30 and V_S60 range from 337 to 374 m/s (1,107 to 1,228 ft/s) and 411 to 457 m/s (1,347 to 1,499 ft/s), respectively.

5.3 Line B

Active and passive surface wave soundings were conducted at five (5) locations along Line B as shown in Figure 1. These 1-D surface wave soundings were centered at Stations 226, 463, 699, 935 and 1171 ft. MASW data could not be acquired at Station 1171 ft due to ponded water, therefore the MASW data was acquired further to the north at Station 1340 ft.

The fit of the theoretical surface wave dispersion curve to the experimental data collected along this line and the modeled S-wave velocity profiles are presented in Appendix B as Figures B-7 to B-11. Tabulated S-wave velocity models for these arrays are presented in Tables 8 to 13, respectively. The S-wave velocity models are expected to be valid to a depth of 75 to 100 m

(246 to 328 ft). A potential velocity inversion identified in Borehole B-4 blow count data and modeled in surface wave array SW-B4 was added to surface wave models where supported by the characteristics of the dispersion curve. This velocity inversion is poorly constrained because the layer has small thickness relative to its depth, and appears to pinch out or become too thin to image south of Station 463 ft.

Except for Station 1171/1340 ft and to a lesser degree Station 935 ft, the surface wave phase velocities from the passive surface wave measurements are in good agreement with those from the MASW data in the region of overlapping wavelength. Station 1171/1340 ft has the same dispersion curve differences as discussed above for Line A. A similar modeling approach was used for this surface wave sounding with Models 1 and 2 assumed to reflect subsurface velocity structure at Stations 1340 and 1171 ft, respectively.

Comparisons of the surface wave velocity models along Line B, both on a single plot and in cross section format, are presented as Figures 6 and 7. The models exhibit similar velocity structure except for the velocity inversion near an elevation of 76 m(249 ft), which pinches out south of Station 463 ft and the possible lower velocity sediments between an elevation of 98 and 111 m (322 and 365 ft). V_S30 and V_S60 range from 335 to 379 m/s (1,100 to 1,242 ft/s) and 418 to 465 m/s (1,372 to 1,525 ft/s), respectively.

5.4 Line C

Active and passive surface wave soundings were conducted at four (4) locations along Line C as shown in Figure 1. These 1-D surface wave soundings were centered at Stations 226, 463, 699 and 935 ft.

The fit of the theoretical surface wave dispersion curve to the experimental data collected along this line and the modeled S-wave velocity profiles are presented in Appendix B as Figures B-12 to B-15. Tabulated S-wave velocity models for these arrays are presented in Tables 14 to 17, respectively. The S-wave velocity models are expected to be valid to a depth of 75 to 100 m (246 to 328 ft). The surface wave phase velocities from the passive surface wave measurements are generally in good agreement with those from the MASW data in the region of overlapping wavelength. The character of the dispersion curves provide no strong evidence of the possible S-wave velocity inversion modeled to the north on Lines A and B.

A comparison of the 4 surface wave velocity models along Line C, both on a single plot and in cross section format, are presented as Figures 8 and 9. The models exhibit similar velocity structure with V_S30 and V_S60 ranging from 355 to 372 m/s (1,164 to 1,222 ft/s) and 452 to 480 m/s (1,482 to 1,574 ft/s), respectively. There are no detectible low velocity zones that could be associated with continuous sequences of silt with thickness greater than 15 to 20% of depth in the velocity models along this line.

6 CONCLUSIONS

Active and passive surface wave were made at 15 locations on the Nu-Way Reclaimed Aggregate Mine located in Irwindale, California between January 2 and February 8, 2008. The locations of the surface wave soundings are shown on Figure 1. The purpose of this investigation was to characterize S-wave velocity structure to a minimum depth of 60 meters (200 ft) to both map lateral variability of sediment velocity structure and identify, thick accumulations of low-velocity, unconsolidated silt units, if present.

A surface wave sounding was conducted near Borehole B-4 (Array SW-B4) during the planning stages of the geophysical investigation. Becker hammer blow counts abruptly decreased below a depth of 50 m (164 ft) in this borehole, indicating that S-wave velocity may be lower. The shape of the dispersion curve for Array SW-B4 (Figure 3) indicated that a thin S-wave velocity inversion may be present near this depth, and the dispersion data was modeled accordingly with a 10 m (33 ft) thick 350 m/s (1,148 ft/s) layer between layers with S-wave velocity of about 550 m/s (1,805 ft/s). The actual velocity structure of this layer is poorly constrained because the layer is not very thick relative to its depth. A thinner layer with lower velocity or thicker layer with higher velocity could equally well fit the dispersion curve. Similar velocity structure was modeled on Lines A and B in the vicinity of Borehole B-4. There was not evidence of a velocity inversion south of Stations 699 and 463 ft on Lines A and B, respectively or on Line C. Surface wave models for Lines A to C are presented in Appendix B. Comparisons of the S-wave velocity models along Lines A, B and C are presented as Figures 4 and 5, 6 and 7, and 8 and 9, respectively.

The surface wave models for Lines A to C can be characterized into three regions of similar S-wave velocity structure: the southern portion of the site encompassing Station 226 and 463 ft on Line A, Station 226 ft on Line B and Line C; the central portion of the site south of Station 1171 on Lines A and B, and the northernmost portion of the site consisting of Station 1340 ft on Lines A and B. In the southern portion of the site there is no evidence of the velocity inversion modeled for Array SW-B4. If present in this portion of the site the low velocity layer is too thin to image. Velocity structure is very similar in the S-wave velocity models and V_S30 and V_S60 range from 346 to 372 m/s (1,135 to 1,222 ft/s) and 438 to 480 m/s (1,437 to 1,574 ft/s), respectively.

In the central portion of the site, the velocity models are similar to those in the south, with the exception of a possible low velocity layer at depths below 43 m (141 ft), thickness of about 5 to 10 m (16 to 33 ft), and approximate velocity of 325 to 375 m/s (1,066 to 1,230 ft/s). This possible low velocity unit is too thin to accurately model and, therefore, the velocity, depth and thickness are not well constrained. The combined thickness/velocity is better constrained and a lower velocity for this unit would require a smaller thickness. V_S30 and V_S60 in the central portion of the site range from 359 to 379 m/s (1,176 to 1,242 ft/s) and 411 to 465 m/s (1,347 to 1,525 ft/s), respectively.

In the northernmost portion of the site (Station 1340 ft on Lines A and B), the S-wave velocity at a depth of 13 to 26 m (41 to 84 ft), appears to be lower than in other portions of the site. Although modeled to remain consistent with the models to the south, there is no conclusive evidence for the low velocity layer modeled at a depth of 43 m (141 ft). V_S30 and V_S60 in this portion of the site range from 335 to 337 m/s (1,100 to 1,107 ft/s) and 404 to 418 m/s (1,326 to 1,372 ft/s), respectively.

Several projects conducted on native soils within one mile of the site yielded V_S30 and V_S60 in the 362 to 567 m/s (1,188 to 1,860 ft/s) and 424 to 664 m/s (1,391 to 2,178 ft/s) range, respectively. The values of V_S30 and V_S60 in the fill soils at this site are at the low end of that observed for native soils in the site vicinity.

In summary, S-wave velocities are relatively high at this site with velocity typically exceeding 350 m/s (1,148 ft/s) below a depth of 13 m (43 ft) in the southern and central portions of the site and 25.5 m (84 ft) in the northernmost portion of the site. Based on the average S-wave velocity of the upper 30 m, the site would be classified as D/C, stiff soil/very dense soil and soft rock for seismic design.

7 REFERENCES

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8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Antony J Martin 02/25/08

Antony J. Martin Date
California Professional Geophysicist GP989
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* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

TABLES

Table 1 V_S Model for Surface Wave Array SW-B4 (Near Borehole B-4)

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
124.0	406.8	0.0	0.0	2.0	6.6	260	853.0	520	1706.0
122.0	400.3	2.0	6.6	3.0	9.8	300	984.3	600	1968.5
119.0	390.4	5.0	16.4	8.0	26.2	350	1148.3	700	2296.6
111.0	364.2	13.0	42.7	15.0	49.2	425	1394.4	850	2788.4
96.0	315.0	28.0	91.9	20.0	65.6	550	1804.5	1100	3608.6
76.0	249.3	48.0	157.5	10.0	32.8	350	1148.3	700	2296.6
66.0	216.5	58.0	190.3	15.0	49.2	550	1804.5	1100	3608.6
51.0	167.3	73.0	239.5	15.0	49.2	675	2214.6	1350	4428.8
36.0	118.1	88.0	288.7	>12.0	>39.4	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 377 m/s (1,238 ft/s), Vs60 = 423 m/s (1,387 ft/s)

Table 2 V_S Model for Surface Wave Array on Line A, Station 226 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
124.0	406.8	0.0	0.0	2.0	6.6	185	607.0	370	1213.9
122.0	400.2	2.0	6.6	3.0	9.8	265	869.4	530	1738.8
119.0	390.4	5.0	16.4	8.0	26.2	335	1099.1	670	2198.2
111.0	364.1	13.0	42.7	15.0	49.2	425	1394.4	850	2788.4
96.0	314.9	28.0	91.9	10.0	32.8	525	1722.4	1050	3444.6
86.0	282.1	38.0	124.7	15.0	49.2	550	1804.5	1100	3608.6
71.0	232.9	53.0	173.9	15.0	49.2	575	1886.5	1150	3772.6
56.0	183.7	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
41.0	134.5	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 352 m/s (1,156 ft/s), Vs60 = 457 m/s (1,499 ft/s)

Table 3 V_S Model for Surface Wave Array on Line A, Station 463 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
124.1	407.0	0.0	0.0	2.0	6.6	215	705.4	430	1410.8
122.1	400.4	2.0	6.6	3.0	9.8	250	820.2	500	1640.4
119.1	390.6	5.0	16.4	8.0	26.2	320	1049.9	640	2099.7
111.1	364.3	13.0	42.7	15.0	49.2	410	1345.1	820	2690.0
96.1	315.1	28.0	91.9	10.0	32.8	525	1722.4	1050	3444.6
86.1	282.3	38.0	124.7	15.0	49.2	550	1804.5	1100	3608.6
71.1	233.1	53.0	173.9	15.0	49.2	575	1886.5	1150	3772.6
56.1	183.9	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
41.1	134.7	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 346 m/s (1,135 ft/s), Vs60 = 451 m/s (1,481 ft/s)

Table 4 V_S Model for Surface Wave Array on Line A, Station 699 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
124.0	406.7	0.0	0.0	2.0	6.6	265	869.4	530	1738.8
122.0	400.1	2.0	6.6	3.0	9.8	300	984.3	600	1968.5
119.0	390.3	5.0	16.4	8.0	26.2	340	1115.5	680	2231.0
111.0	364.0	13.0	42.7	15.0	49.2	425	1394.4	850	2788.4
96.0	314.8	28.0	91.9	20.0	65.6	525	1722.4	1050	3444.6
76.0	249.2	48.0	157.5	7.5	24.6	350	1148.3	700	2296.6
68.5	224.6	55.5	182.1	15.0	49.2	575	1886.5	1150	3772.6
53.5	175.4	70.5	231.3	15.0	49.2	675	2214.6	1350	4428.8
38.5	126.1	85.5	280.5	>14.5	>47.6	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 374 m/s (1,228 ft/s), Vs60 = 434 m/s (1,424 ft/s)

Table 5 V_S Model for Surface Wave Array on Line A, Station 935 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
123.8	406.1	0.0	0.0	2.0	6.6	245	803.8	490	1607.6
121.8	399.5	2.0	6.6	3.0	9.8	290	951.4	580	1902.9
118.8	389.7	5.0	16.4	8.0	26.2	325	1066.3	650	2132.5
110.8	363.5	13.0	42.7	15.0	49.2	425	1394.4	850	2788.4
95.8	314.2	28.0	91.9	20.0	65.6	525	1722.4	1050	3444.6
75.8	248.6	48.0	157.5	10.0	32.8	350	1148.3	700	2296.3
65.8	215.8	58.0	190.3	10.0	32.8	575	1886.5	1150	3772.6
55.8	183.0	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
40.8	133.8	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 365 m/s (1,197 ft/s), Vs60 = 411 m/s (1,347 ft/s)

Table 6 V_S Model 1 for Surface Wave Array on Line A, Station 1171/1340 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
123.6	405.5	0.0	0.0	2.5	8.2	285	935.0	570	1870.1
121.1	397.3	2.5	8.2	3.0	9.8	300	984.3	600	1968.5
118.1	387.5	5.5	18.0	7.0	23.0	320	1049.9	640	2099.7
111.1	364.5	12.5	41.0	13.0	42.7	335	1099.1	670	2198.2
98.1	321.9	25.5	83.7	17.5	57.4	475	1558.4	950	3116.5
80.6	264.4	43.0	141.1	10.0	32.8	350	1148.3	700	2296.6
70.6	231.6	53.0	173.9	15.0	49.2	600	1968.5	1200	3936.7
55.6	182.4	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
40.6	133.2	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 337 m/s (1,107 ft/s), Vs60 = 404 m/s (1,326 ft/s)

Table 7 V_S Model 2 for Surface Wave Array on Line A, Station 1171/1340 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
123.6	405.5	0.0	0.0	2.5	8.2	285	935.0	570	1870.1
121.1	397.3	2.5	8.2	3.0	9.8	300	984.3	600	1968.5
118.1	387.5	5.5	18.0	7.0	23.0	320	1049.9	640	2099.7
111.1	364.5	12.5	41.0	13.0	42.7	425	1394.4	850	2788.4
98.1	321.9	25.5	83.7	17.5	57.4	490	1607.6	980	3214.9
80.6	264.4	43.0	141.1	10.0	32.8	330	1082.7	660	2165.4
70.6	231.6	53.0	173.9	15.0	49.2	600	1968.5	1200	3936.7
55.6	182.4	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
40.6	133.2	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 373 m/s (1,224 ft/s), Vs60 = 426 m/s (1,398 ft/s)

Table 8 V_S Model for Surface Wave Array on Line B, Station 226 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
121.7	399.3	0.0	0.0	2.0	6.6	205	672.6	410	1345.1
119.7	392.8	2.0	6.6	3.0	9.8	255	836.6	510	1673.2
116.7	382.9	5.0	16.4	8.0	26.2	330	1082.7	660	2165.4
108.7	356.7	13.0	42.7	15.0	49.2	425	1394.4	850	2788.4
93.7	307.5	28.0	91.9	10.0	32.8	450	1476.4	900	2952.4
83.7	274.7	38.0	124.7	15.0	49.2	500	1640.4	1000	3280.5
68.7	225.5	53.0	173.9	15.0	49.2	575	1886.5	1150	3772.6
53.7	176.2	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
38.7	127.0	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 351 m/s (1,151 ft/s), Vs60 = 438 m/s (1,437 ft/s)

Table 9 V_S Model for Surface Wave Array on Line B, Station 463 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
121.6	398.9	0.0	0.0	2.0	6.6	240	787.4	480	1574.8
119.6	392.4	2.0	6.6	3.0	9.8	255	836.6	510	1673.2
116.6	382.5	5.0	16.4	6.0	19.7	325	1066.3	650	2132.5
110.6	362.8	11.0	36.1	15.0	49.2	400	1312.3	800	2624.3
95.6	313.6	26.0	85.3	17.5	57.4	525	1722.4	1050	3444.6
78.1	256.2	43.5	142.7	5.0	16.4	375	1230.3	750	2460.3
73.1	239.8	48.5	159.1	15.0	49.2	550	1804.5	1100	3608.6
58.1	190.6	63.5	208.3	15.0	49.2	675	2214.6	1350	4428.8
43.1	141.4	78.5	257.5	20.0	65.6	775	2542.7	1550	5085.0
23.1	75.8	98.5	323.2	>1.5	>4.9	875	2870.7	1750	5741.1

Note: Model valid to 75 - 100 m, Vs30 = 359 m/s (1,176 ft/s), Vs60 = 465 m/s (1,525 ft/s)

Table 10 V_S Model for Surface Wave Array on Line B, Station 699 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
122.3	401.3	0.0	0.0	2.0	6.6	270	885.8	540	1771.7
120.3	394.8	2.0	6.6	3.0	9.8	295	967.8	590	1935.7
117.3	384.9	5.0	16.4	6.0	19.7	300	984.3	600	1968.5
111.3	365.3	11.0	36.1	15.0	49.2	425	1394.4	850	2788.4
96.3	316.0	26.0	85.3	17.5	57.4	525	1722.4	1050	3444.6
78.8	258.6	43.5	142.7	9.0	29.5	375	1230.3	750	2460.3
69.8	229.1	52.5	172.2	15.0	49.2	550	1804.5	1100	3608.6
54.8	179.9	67.5	221.5	15.0	49.2	675	2214.6	1350	4428.8
39.8	130.7	82.5	270.7	>17.5	>57.4	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 373 m/s (1,223 ft/s), Vs60 = 447 m/s (1,465 ft/s)

Table 11 V_S Model for Surface Wave Array on Line B, Station 935 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
123.2	404.3	0.0	0.0	2.0	6.6	245	803.8	490	1607.6
121.2	397.8	2.0	6.6	3.0	9.8	300	984.3	600	1968.5
118.2	387.9	5.0	16.4	7.0	23.0	335	1099.1	670	2198.2
111.2	365.0	12.0	39.4	15.0	49.2	435	1427.2	870	2854.0
96.2	315.7	27.0	88.6	20.0	65.6	525	1722.4	1050	3444.6
76.2	250.1	47.0	154.2	10.0	32.8	350	1148.3	700	2296.6
66.2	217.3	57.0	187.0	15.0	49.2	550	1804.5	1100	3608.6
51.2	168.1	72.0	236.2	15.0	49.2	675	2214.6	1350	4428.8
36.2	118.9	87.0	285.4	>13.0	>42.7	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 379 m/s (1,242 ft/s), Vs60 = 423 m/s (1,388 ft/s)

Table 12 V_S Model 1 for Surface Wave Array on Line B, Station 1171/1340 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
123.5	405.2	0.0	0.0	2.5	8.2	245	803.8	490	1607.6
121.0	397.0	2.5	8.2	3.0	9.8	325	1066.3	650	2132.5
118.0	387.1	5.5	18.0	7.0	23.0	370	1213.9	740	2427.5
111.0	364.2	12.5	41.0	13.0	42.7	300	984.3	600	1968.5
98.0	321.5	25.5	83.7	17.5	57.4	575	1886.5	1150	3772.6
80.5	264.1	43.0	141.1	10.0	32.8	360	1181.1	720	2361.9
70.5	231.3	53.0	173.9	15.0	49.2	600	1968.5	1200	3936.7
55.5	182.1	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
40.5	132.9	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 335 m/s (1,100 ft/s), Vs60 = 418 m/s (1,372 ft/s)

Table 13 V_S Model 2 for Surface Wave Array on Line B, Station 1171/1340 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
123.5	405.2	0.0	0.0	2.5	8.2	245	803.8	490	1607.6
121.0	397.0	2.5	8.2	3.0	9.8	325	1066.3	650	2132.5
118.0	387.1	5.5	18.0	7.0	23.0	355	1164.7	710	2329.4
111.0	364.2	12.5	41.0	13.0	42.7	400	1312.3	800	2624.3
98.0	321.5	25.5	83.7	17.5	57.4	550	1804.5	1100	3608.6
80.5	264.1	43.0	141.1	10.0	32.8	325	1066.3	650	2132.5
70.5	231.3	53.0	173.9	15.0	49.2	600	1968.5	1200	3936.7
55.5	182.1	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
40.5	132.9	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 376 m/s (1,233 ft/s), Vs60 = 435 m/s (1,429 ft/s)

Table 14 V_S Model for Surface Wave Array on Line C, Station 226 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
123.7	406.0	0.0	0.0	2.0	6.6	250	820.2	500	1640.4
121.7	399.4	2.0	6.6	3.0	9.8	275	902.2	550	1804.5
118.7	389.6	5.0	16.4	8.0	26.2	325	1066.3	650	2132.5
110.7	363.3	13.0	42.7	15.0	49.2	420	1378.0	840	2755.6
95.7	314.1	28.0	91.9	10.0	32.8	575	1886.5	1150	3772.6
85.7	281.3	38.0	124.7	15.0	49.2	600	1968.5	1200	3936.7
70.7	232.1	53.0	173.9	15.0	49.2	625	2050.5	1250	4100.7
55.7	182.9	68.0	223.1	15.0	49.2	700	2296.6	1400	4592.8
40.7	133.7	83.0	272.3	>17.0	>55.8	800	2624.7	1600	5249.0

Note: Model valid to 75 - 100 m, Vs30 = 363 m/s (1,190 ft/s), Vs60 = 480 m/s (1,574 ft/s)

Table 15 V_S Model for Surface Wave Array on Line C, Station 463 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
122.7	402.6	0.0	0.0	2.0	6.6	250	820.2	500	1640.4
120.7	396.0	2.0	6.6	3.0	9.8	300	984.3	600	1968.5
117.7	386.2	5.0	16.4	8.0	26.2	315	1033.5	630	2066.9
109.7	359.9	13.0	42.7	15.0	49.2	450	1476.4	900	2952.4
94.7	310.7	28.0	91.9	10.0	32.8	525	1722.4	1050	3444.6
84.7	277.9	38.0	124.7	15.0	49.2	550	1804.5	1100	3608.6
69.7	228.7	53.0	173.9	15.0	49.2	575	1886.5	1150	3772.6
54.7	179.5	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
39.7	130.2	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 372 m/s (1,222 ft/s), Vs60 = 473 m/s (1,553 ft/s)

Table 16 V_S Model for Surface Wave Array on Line C, Station 699 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
122.5	401.8	0.0	0.0	2.0	6.6	225	738.2	450	1476.4
120.5	395.3	2.0	6.6	3.0	9.8	290	951.4	580	1902.9
117.5	385.4	5.0	16.4	8.0	26.2	325	1066.3	650	2132.5
109.5	359.2	13.0	42.7	15.0	49.2	425	1394.4	850	2788.4
94.5	310.0	28.0	91.9	10.0	32.8	525	1722.4	1050	3444.6
84.5	277.2	38.0	124.7	15.0	49.2	550	1804.5	1100	3608.6
69.5	228.0	53.0	173.9	15.0	49.2	600	1968.5	1200	3936.7
54.5	178.7	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
39.5	129.5	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

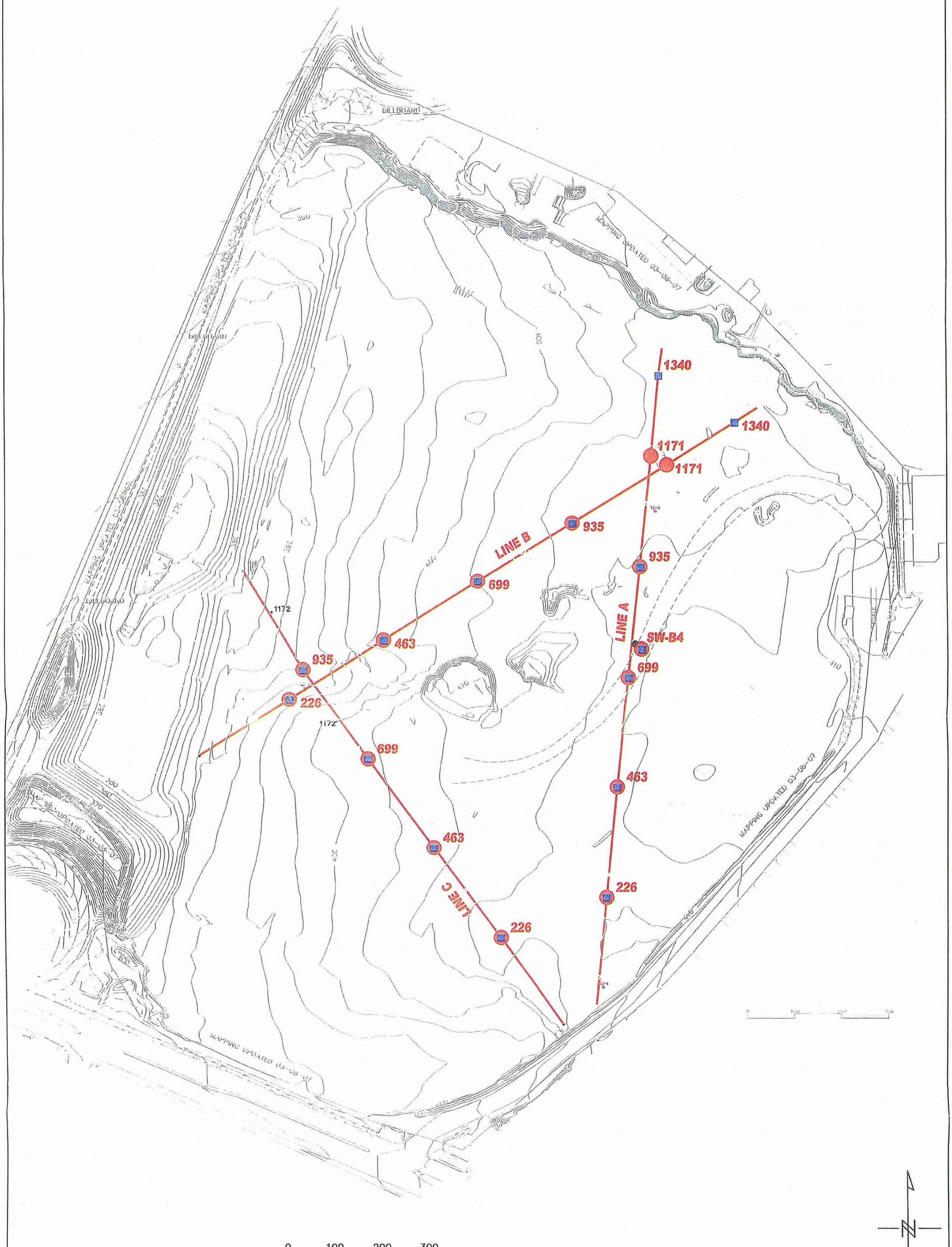
Note: Model valid to 75 - 100 m, Vs30 = 362 m/s (1,187 ft/s), Vs60 = 465 m/s (1,526 ft/s)

Table 17 V_S Model for Surface Wave Array on Line C, Station 935 ft

Elevation of Top of Layer		Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m	ft	m/s	ft/s	m/s	ft/s
120.7	396.0	0.0	0.0	2.0	6.6	205	672.6	410	1345.1
118.7	389.4	2.0	6.6	3.0	9.8	275	902.2	550	1804.5
115.7	379.6	5.0	16.4	8.0	26.2	325	1066.3	650	2132.5
107.7	353.3	13.0	42.7	15.0	49.2	425	1394.4	850	2788.4
92.7	304.1	28.0	91.9	10.0	32.8	500	1640.4	1000	3280.5
82.7	271.3	38.0	124.7	15.0	49.2	525	1722.4	1050	3444.6
67.7	222.1	53.0	173.9	15.0	49.2	575	1886.5	1150	3772.6
52.7	172.9	68.0	223.1	15.0	49.2	675	2214.6	1350	4428.8
37.7	123.7	83.0	272.3	>17.0	>55.8	775	2542.7	1550	5085.0

Note: Model valid to 75 - 100 m, Vs30 = 355 m/s (1,164 ft/s), Vs60 = 452 m/s (1,482 ft/s)

FIGURES



LEGEND

LINE C — APPROXIMATE LOCATION OF GEOPHYSICAL TRAVERSE

226 — CENTER OF 1-D PASSIVE SURFACE WAVE SOUNDING / STATION NUMBER

1340 — CENTER OF 1-D ACTIVE SURFACE WAVE SOUNDING / STATION NUMBER

NOTE: BASE MAP PROVIDED BY IRVINE GEOTECHNICAL

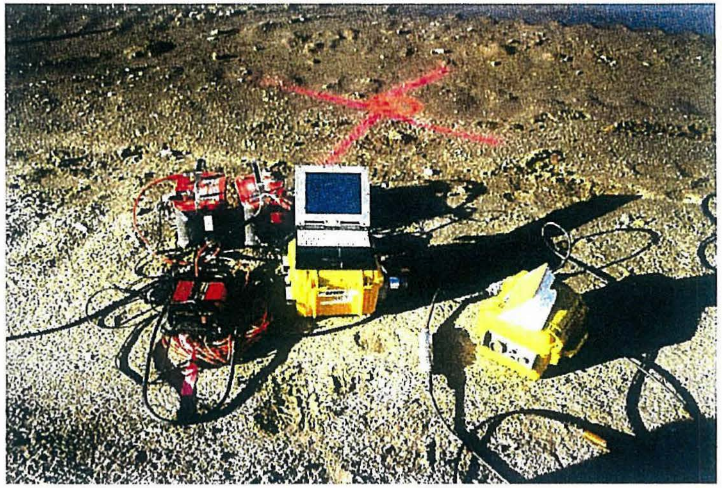
GEOVision geophysical services

Project #	7629
Date	Feb 25,2008
Developed by	L ANNIS
Drawn by	T RODRIGUEZ
Approved by	
File	G:\7600-7699\7629\7629-1.dwg

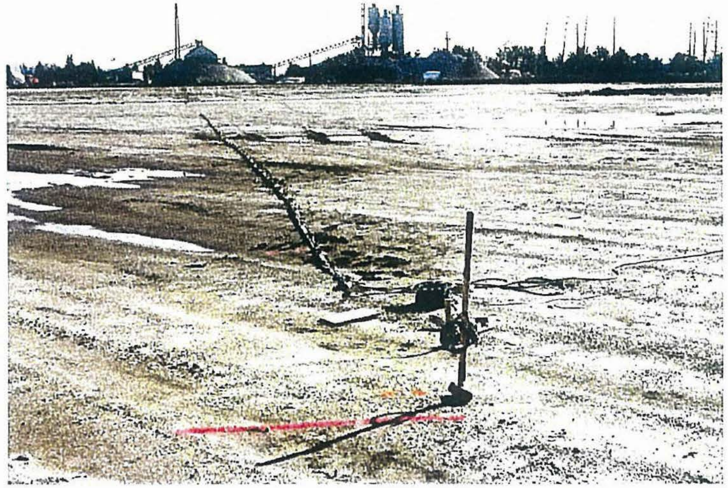
FIGURE 1
SITE MAP

NU-WAY RECLAIMED AGGREGATE MINE
IRWINDALE, CALIFORNIA

PREPARED FOR
IRVINE GEOTECHNICAL

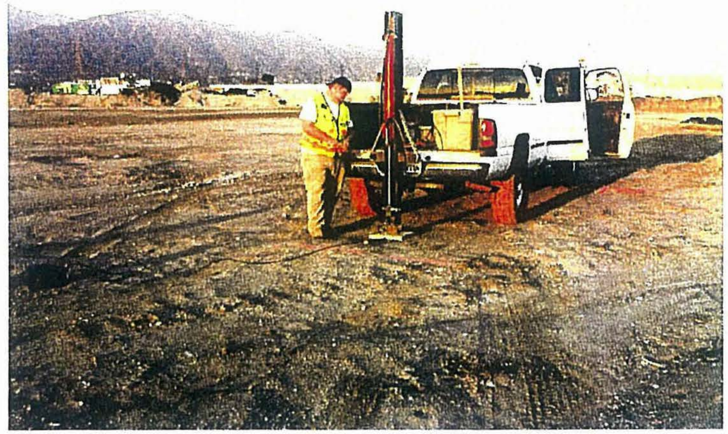


SEISMIC DATA ACQUISITION SYSTEM



LANDSTREAMER USED FOR MASW SURVEY

PASSIVE SURFACE WAVE (NOISE) MEASUREMENTS
ALONG LINEAR ARRAY



ACCELERATED WEIGHT DROP ENERGY SOURCE

GEOVision geophysical services	FIGURE 2 SURFACE WAVE SURVEY PHOTOGRAPHS
Project # 7629 Date: FEB 18, 2008	NU-WAY RECLAIMED AGGREGATE MINE IRWINDALE, CALIFORNIA
Drawn By: A. MARTIN Approved By: [signature] File C:\gvprojects\7629irvgeo\report\f2.cdr	PREPARED FOR IRVINE GEOTECHNICAL

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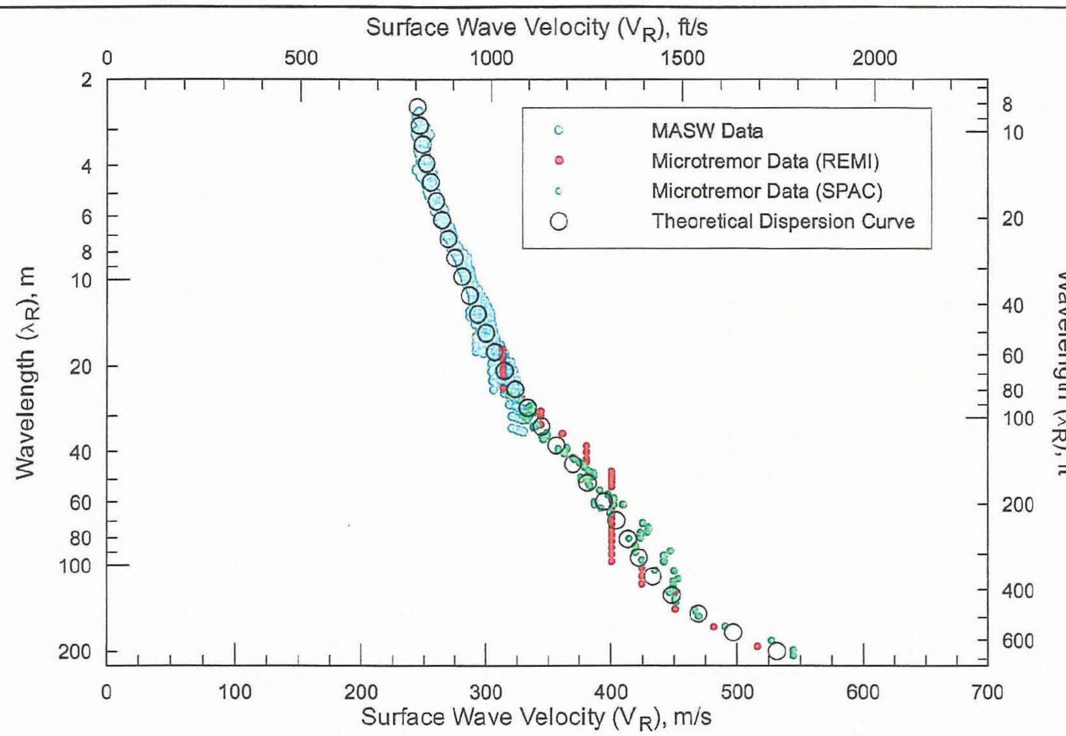
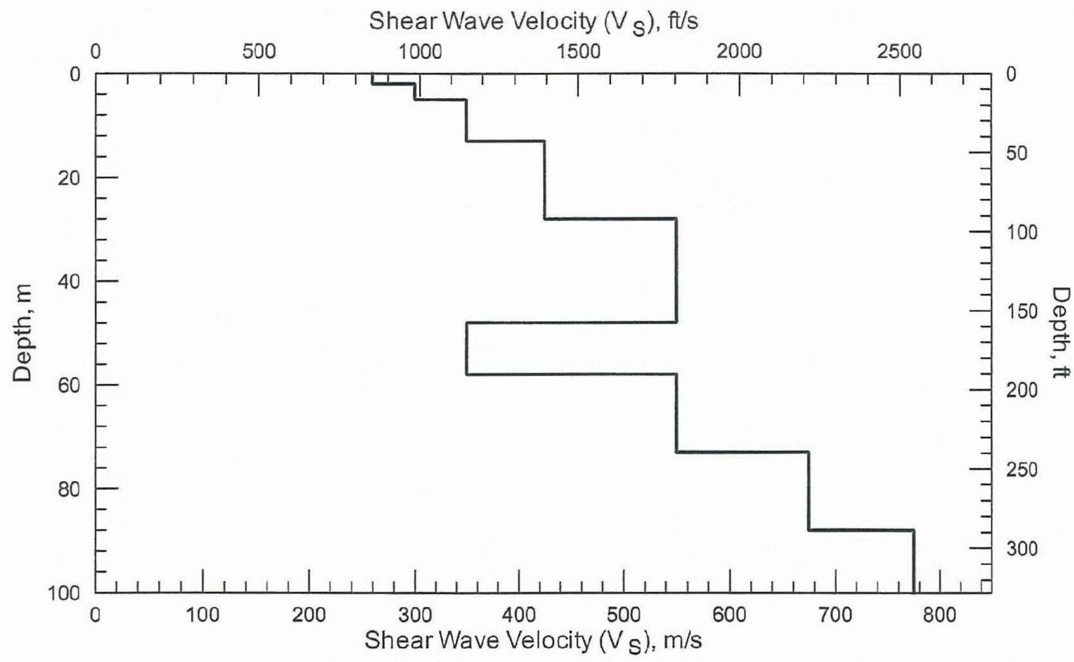
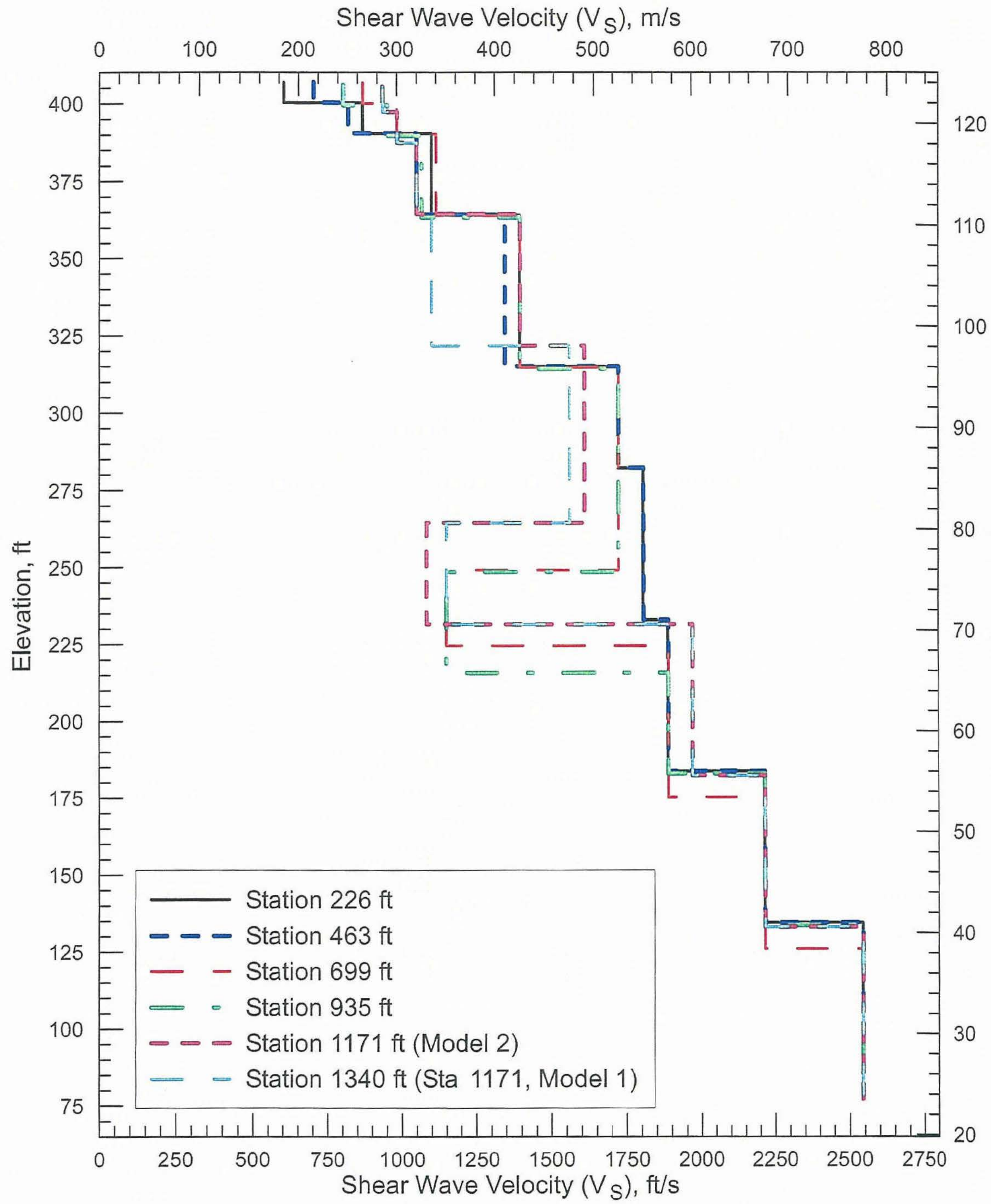


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array SW-B4, near Borehole B-4



V_S Profile from MASW and Microtremor Array SW-B4, near Borehole B-4

GEOVision geophysical services	FIGURE 3 SURFACE WAVE MODEL FOR ARRAY SW-B4
Project # 7629	NU-WAY RECLAIMED AGGREGATE MINE IRWINDALE, CALIFORNIA
Date: FEB 18, 2008	
Drawn By: A. MARTIN	PREPARED FOR IRVINE GEOTECHNICAL
Approved By:	
File C:\gvprojects\7629irvgeo\report\f3.cdr	



GEOVision geophysical services	FIGURE 4 SURFACE WAVE MODELS FOR LINE A
Project # 7629	NU-WAY RECLAIMED AGGREGATE MINE IRWINDALE, CALIFORNIA
Date: FEB 18, 2008	
Drawn By: A. MARTIN	PREPARED FOR IRVINE GEOTECHNICAL
Approved By:	
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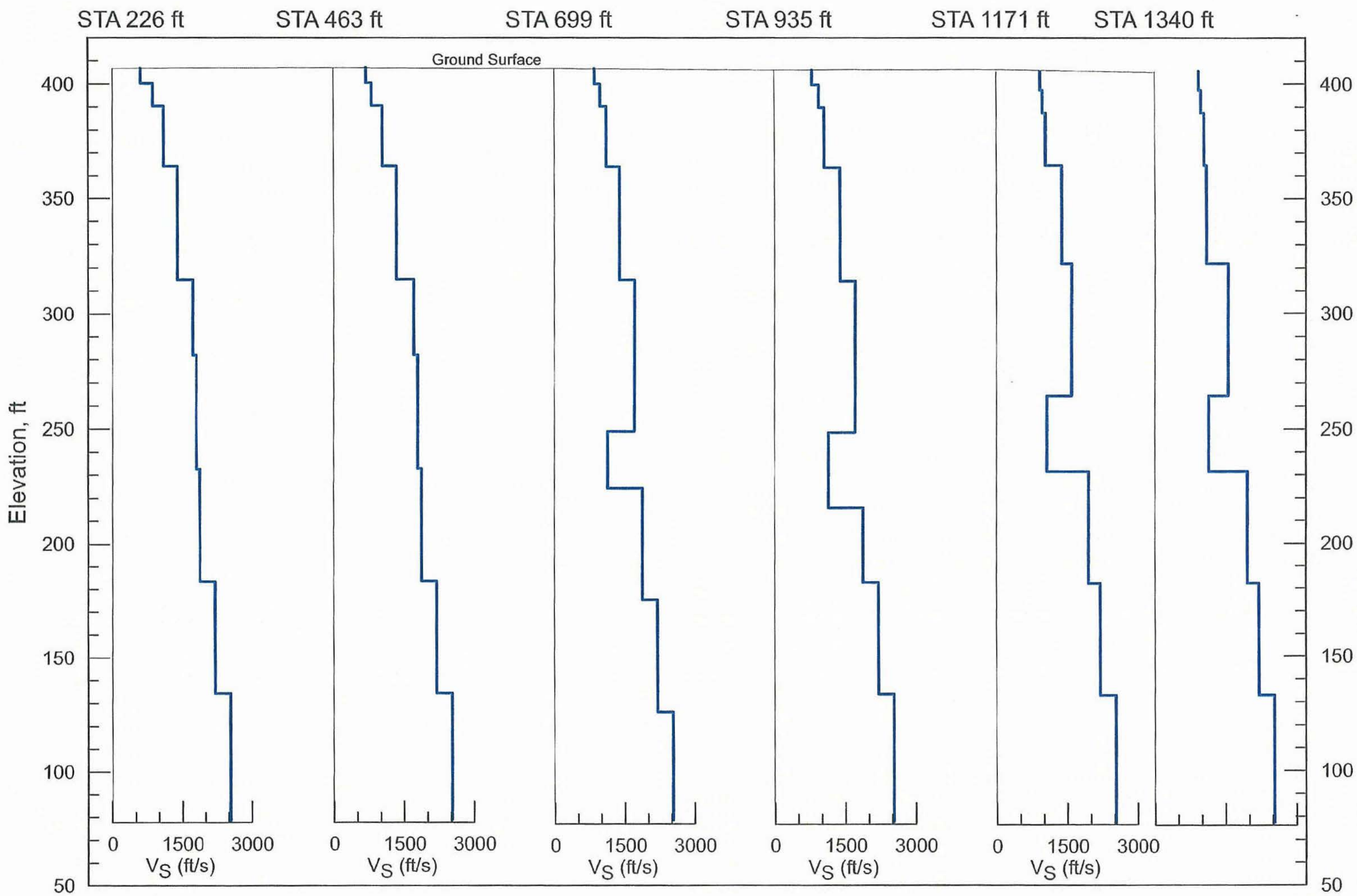

GEOVision geophysical services	FIGURE 5 S-WAVE VELOCITY SECTION - LINE A
Project # 7629 Date: FEB 18, 2008	NU-WAY RECLAIMED AGGREGATE MINE IRWINDALE, CALIFORNIA
Drawn By: A. MARTIN Approved By: File C:\gvprojects\7629irvgeo\reportf5.cdr	PREPARED FOR IRVINE GEOTECHNICAL

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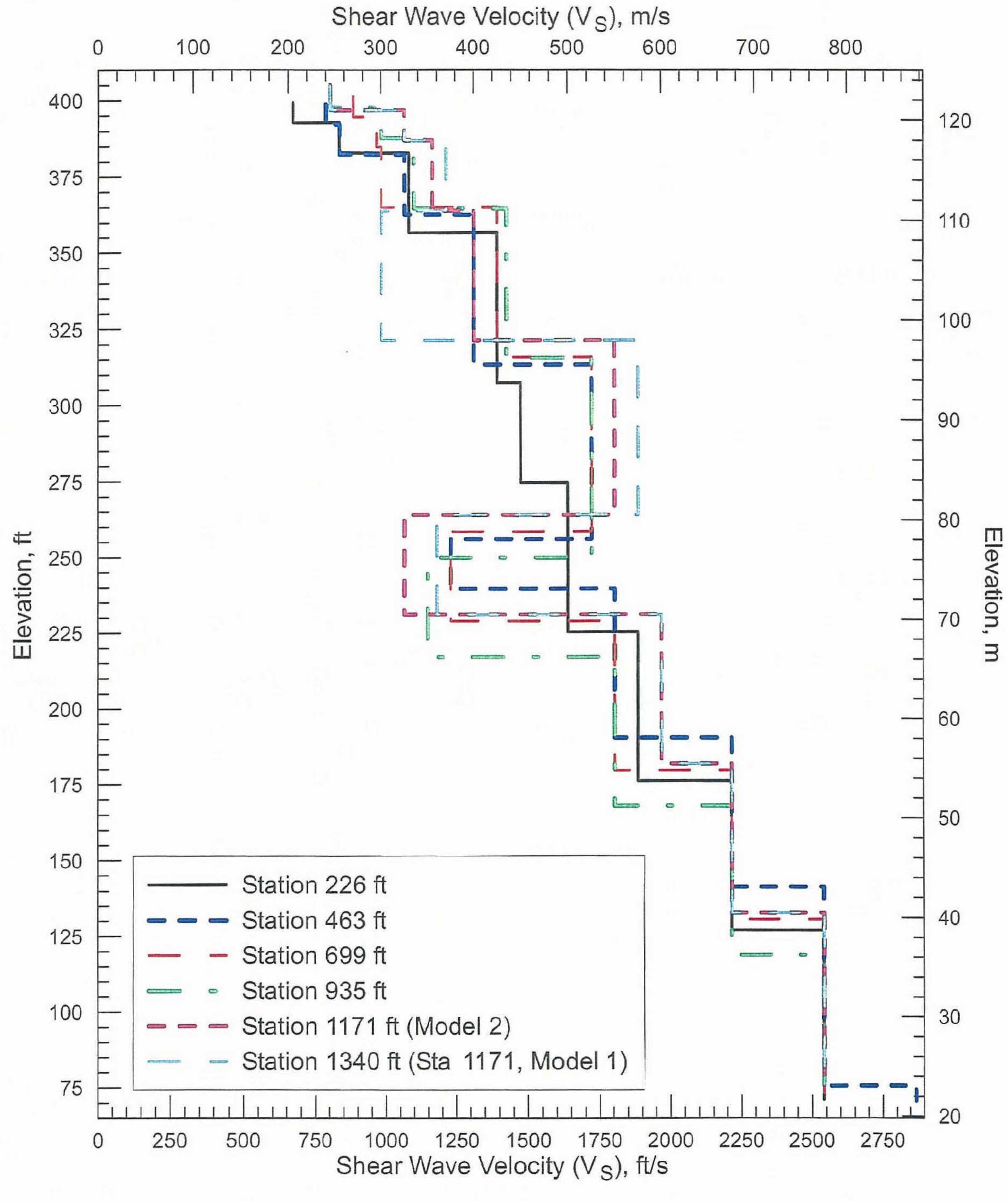

GEOVision geophysical services	FIGURE 6 SURFACE WAVE MODELS FOR LINE B
Project # 7629	NU-WAY RECLAIMED AGGREGATE MINE IRWINDALE, CALIFORNIA
Date: FEB 18, 2008	
Drawn By: A. MARTIN	PREPARED FOR IRVINE GEOTECHNICAL
Approved By: [signature]	
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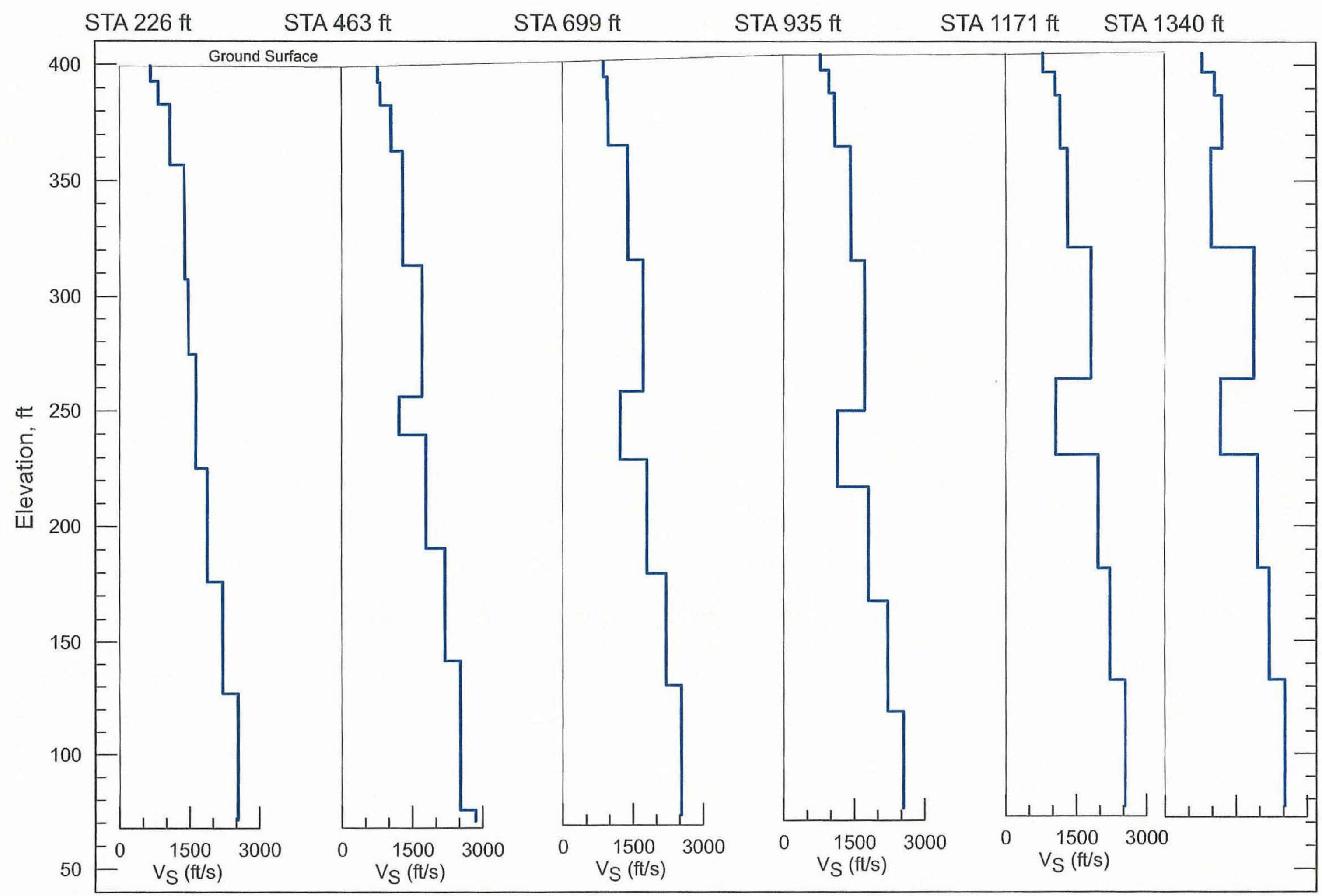

GEOVision geophysical services	FIGURE 7 S-WAVE VELOCITY SECTION - LINE B
Project # 7629 Date: FEB 18, 2008	NU-WAY RECLAIMED AGGREGATE MINE IRWINDALE, CALIFORNIA
Drawn By: A. MARTIN Approved By: Antony J Martin File C:\gvprojects\7629irvgeo\report\f7.cdr	PREPARED FOR IRVINE GEOTECHNICAL

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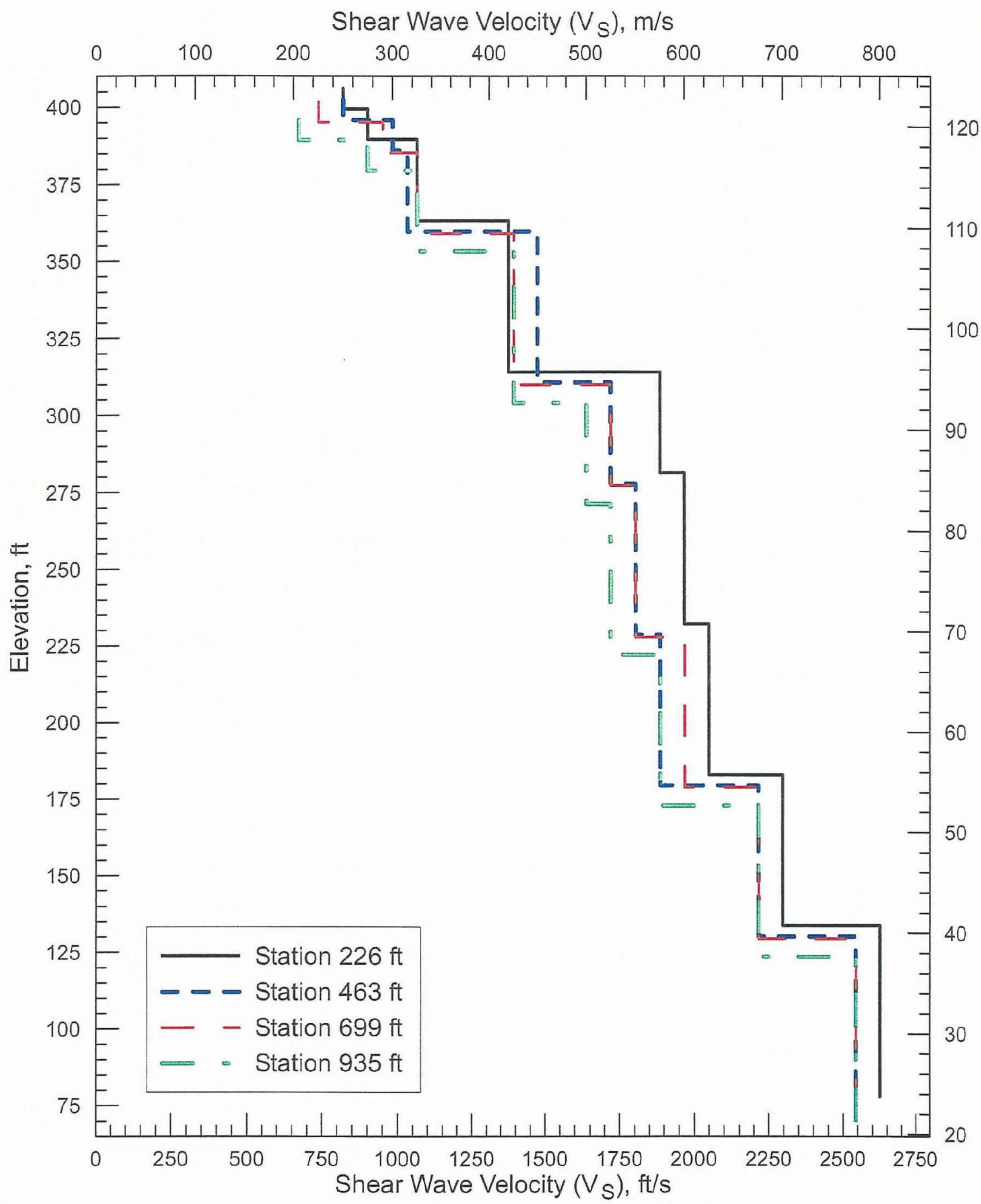

GEOVision geophysical services	FIGURE 8 SURFACE WAVE MODELS FOR LINE C
Project # 7629	NU-WAY RECLAIMED AGGREGATE MINE IRWINDALE, CALIFORNIA
Date: FEB 18, 2008	
Drawn By: A. MARTIN	PREPARED FOR IRVINE GEOTECHNICAL
Approved By:	
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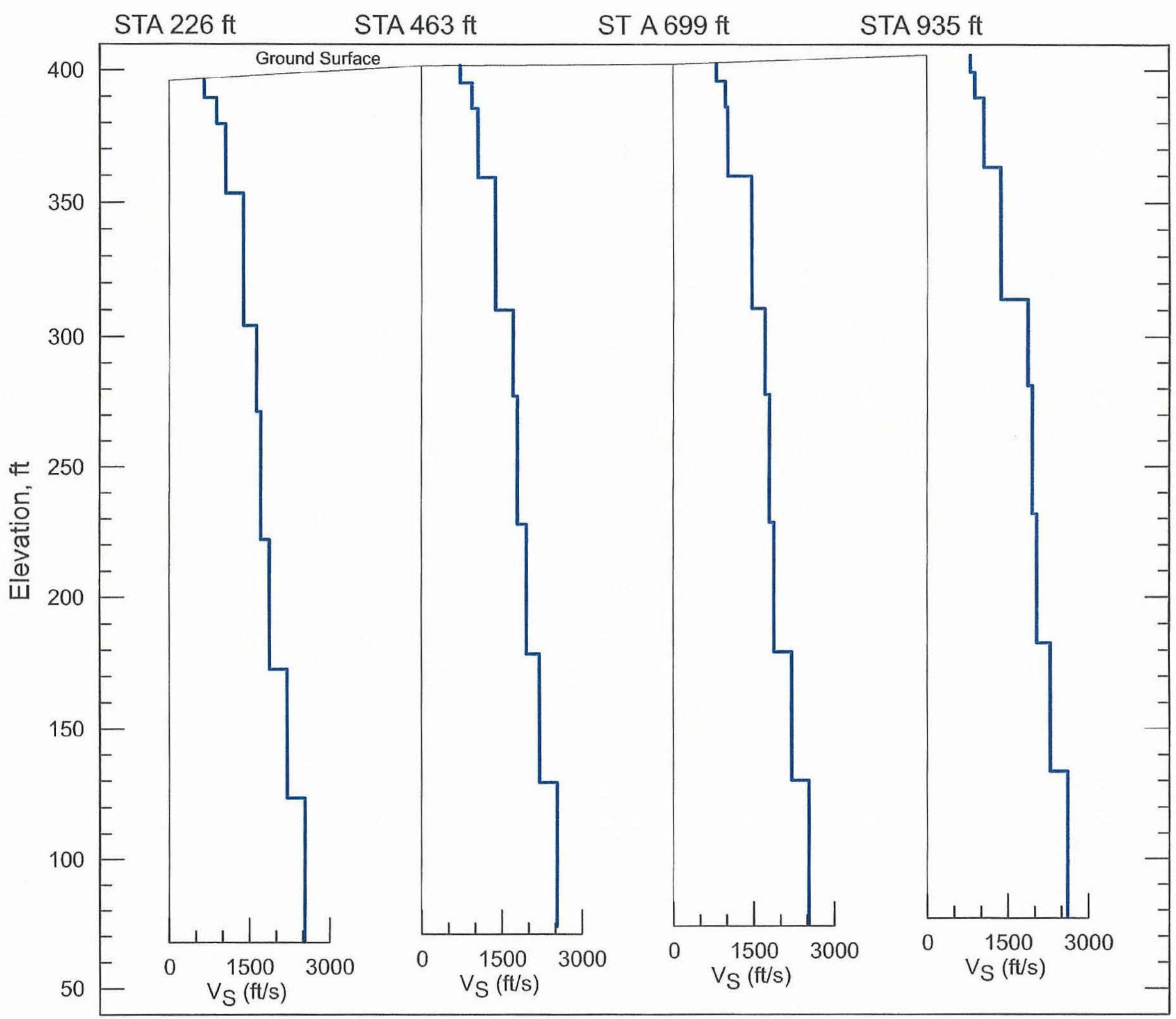

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Project # 7629

Date: FEB 18, 2008

Drawn By: A. MARTIN

Approved By:

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FIGURE 9
S-WAVE VELOCITY SECTION - LINE C

NU-WAY RECLAIMED AGGREGATE MINE
IRWINDALE, CALIFORNIA

PREPARED FOR
IRVINE GEOTECHNICAL

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APPENDIX A

TECHNICAL NOTE

ACTIVE AND PASSIVE SURFACE WAVE TECHNIQUES

ACTIVE AND PASSIVE SURFACE WAVE TECHNIQUES

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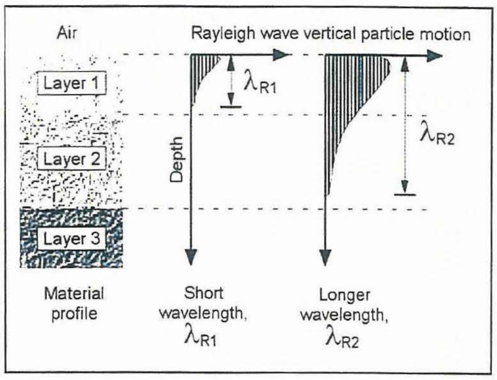
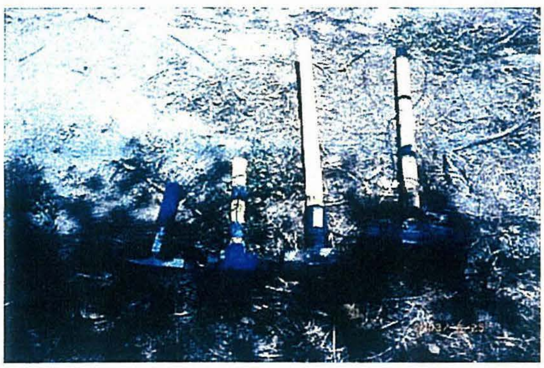
Overview

Active and passive surface wave techniques are relatively new in-situ seismic methods for determining shear wave velocity (V_S) profiles. Testing is performed on the ground surface, allowing for less costly measurements than with traditional borehole methods. The basis of surface wave techniques is the dispersive characteristic of Rayleigh waves when traveling through a layered medium. Rayleigh wave velocity is determined by the material properties (primarily shear wave velocity, but also to a lesser degree compression wave velocity and material density) of the subsurface to a depth of approximately 1 to 2 wavelengths. As shown in the adjacent diagram, longer wavelengths penetrate deeper and their velocity is affected by the material properties at greater depth. Surface wave testing consists of measuring the surface wave dispersion curve at a site and modeling it to obtain the corresponding shear wave velocity profile.



Active Surface Wave Techniques

Active surface wave techniques measure surface waves generated by dynamic sources such as hammers, weight drops, electromechanical shakers, vibroseis and bulldozers. These techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods.



Hammer Energy Sources

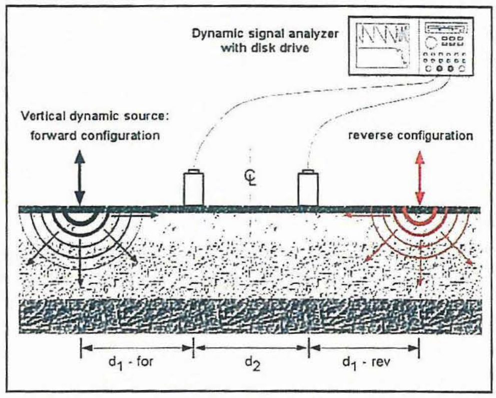
Accelerated Weight Drop

Electromechanical Shaker

Bulldozer Energy Source

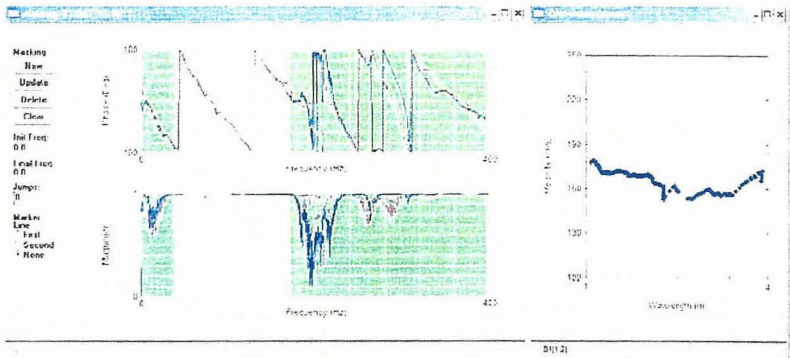
1151 Pomona Road, Corona, California 92882 ph. 951-549-1234 fx 951-549-1236 www.geovision.com

The SASW method is optimized for conducting V_S depth soundings. A dynamic source is used to generate surface waves of different wavelengths (or frequencies) which are monitored by two or more receivers at known offsets. An expanding receiver spread and optimized source-receiver geometry are used to minimize near field effects, body wave signal and attenuation. A dynamic signal analyzer is typically used to calculate the phase and coherence of the cross spectrum of the time history data collected at a pair of receivers. During data analysis, an interactive masking process is used to discard low quality data and to unwrap the phase spectrum, as shown in the figure below. The dispersion curve (Rayleigh wave phase velocity versus frequency or alternatively wavelength) is calculated from the unwrapped phase spectrum.



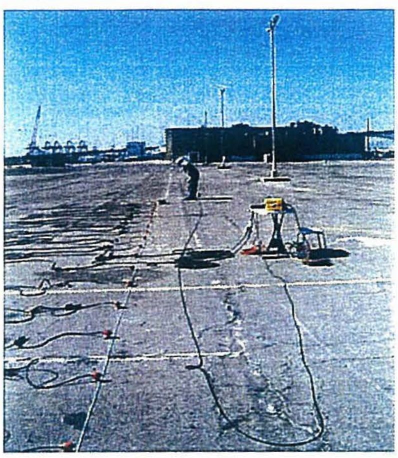
SASW Setup

HP Dynamic Signal Analyzer

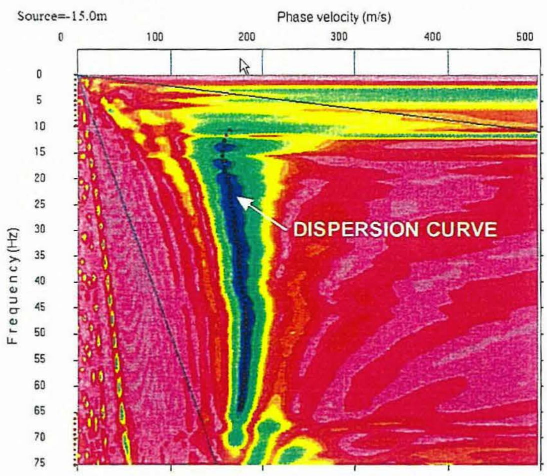


Masking of Wrapped Phase Spectrum and Resulting Dispersion Curve

The MASW field layout is similar to that of the seismic refraction technique. Twenty four, or more, geophones are laid out in a linear array with 1 to 2m spacing and connected to a multi-channel seismograph as shown below. This technique is ideally suited to 2D V_S imaging, with data collected in a roll-along manner similar to that of the seismic reflection technique. The source is offset at a predetermined distance from the near geophone usually determined by field testing. The Rayleigh wave dispersion curve is obtained by a wavefield transformation of the seismic record such as the f-k or τ-p transforms. These transforms are very effective at isolating surface wave energy from that of body waves. The dispersion curve is picked as the peak of the surface wave energy in slowness (or velocity) – frequency space as shown. One advantage of the MASW technique is that the wavefield transformation may not only identify the fundamental mode but also higher modes of surface waves. At some sites, particularly those with large velocity inversions, higher surface wave modes may contain more energy than the fundamental mode.



MASW Field Setup

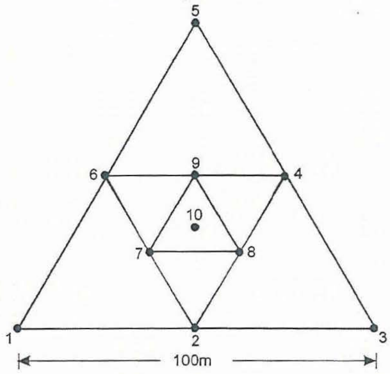


Wavefield Transform of MASW data

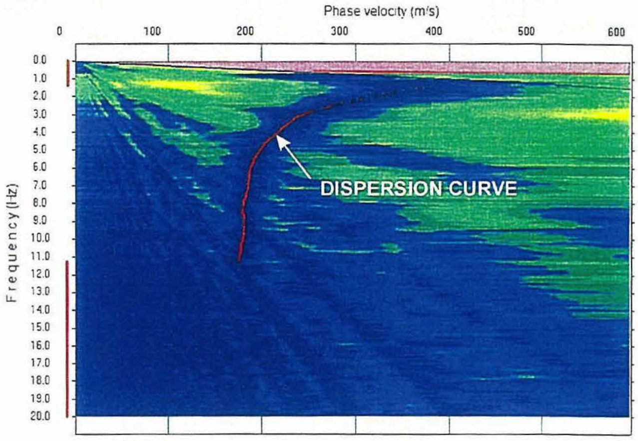
Passive Surface Wave Techniques

Passive surface wave techniques measure noise; surface waves from ocean wave activity, traffic, factories, etc. These techniques include the array microtremor and refraction microtremor (REMI) techniques.

The array microtremor technique typically uses 7 or more 4.5- or 1-Hz geophones arranged in a two-dimensional array. The most common arrays are the triangle, circle, semi-circle and "L" arrays. The triangle array, which consists of several embedded equilateral triangles, is often used as it provides good results with a relatively small number of geophones. With this array the outer side of the triangle should be at least as long as the desired depth of investigation. Typically, fifteen to twenty 30-second noise records are acquired for analysis. The spatial autocorrelation (SPAC) technique is one of several methods that can be used to estimate the Rayleigh wave dispersion curve. A first order Bessel function is fit to the SPAC function to determine the phase velocity for particular frequency. The image shown below shows the degree of fitness of the Bessel function to the SPAC function for a wide range of phase velocity and frequency. The dispersion curve, is the peak (best fit), as shown in the figure below.

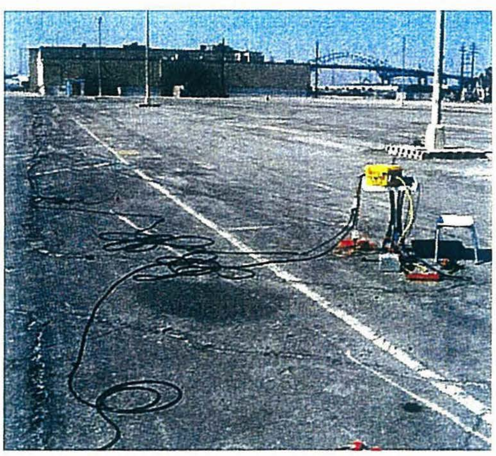


Triangle Array Geometry

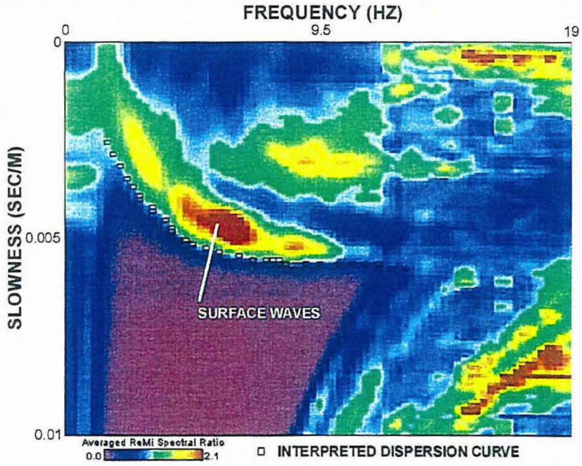


Dispersion Curve from Array Microtremor Measurements

The refraction microtremor (REMI) technique uses a field layout similar to the seismic refraction method (hence its name). Twenty-four, 4.5 Hz geophones are laid out in a linear array with a spacing of 6 to 8m and fifteen to twenty 30-second noise records are acquired. A slowness-frequency (p-f) wavefield transform is used to separate Rayleigh wave energy from that of other waves. Because the noise field can originate from any direction, the wavefield transform is conducted for multiple vectors through the geophone array, all of which are summed. The dispersion curve is defined as the lower envelope of the Rayleigh wave energy in p-f space. Because the lower envelope is picked rather than the energy peak (energy traveling along the profile is slower than that approaching from an angle), this technique may be somewhat more subjective than the others, particularly at low frequencies. The SPAC technique can also be used to extract the surface wave dispersion curve from linear array microtremor data providing there are omni-directional noise sources.



Refraction Microtremor Array Layout



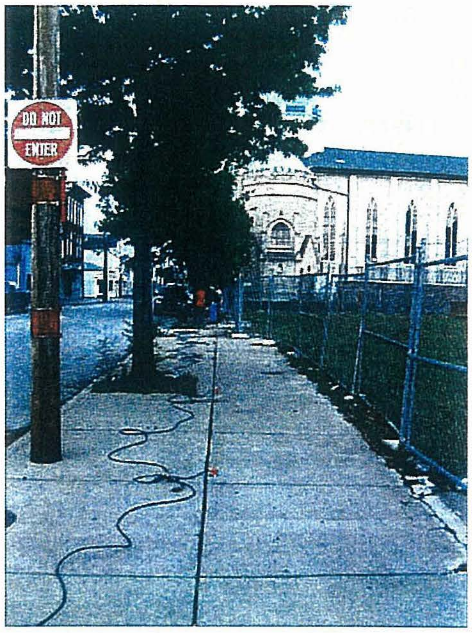
Wavefield Transform of REMI Data

Depth of Investigation

Active surface wave investigations typically use various sized sledge hammers to image the shear wave velocity structure to depths of up to 15m. Weight drops and electromechanical shakers can often be used to image to depths of 30m. Bulldozers and vibroseis trucks can be used to image to depths as great as 100m. Passive surface wave techniques can often image shear wave velocity structure to depths of over 100m, given sufficient noise sources and space for the receiver array. Large passive arrays, utilizing long-period seismometers with GPS clocks have been used to image shear wave velocity structure to depths of several kilometers.

Combined Active and Passive Surface Wave Testing

The combined use of active and passive techniques may offer significant advantages on many investigations. It can be very costly to mobilize large energy sources for 30m/100ft active surface wave soundings. In urban environments, the combined use of active and passive surface wave techniques can image to these depths without the need for large energy sources. We have found that dispersion curves from active and passive surface wave techniques are generally in good agreement, making the combined use of the two techniques viable. It is not recommended that passive surface wave techniques be applied alone for UBC/IBC site classification investigations. Microtremor techniques do not generally characterize near surface velocity, which may have a significant impact of the average shear wave velocity of the upper 30m or 100ft and so should always be used in conjunction with SASW or MASW. An SASW sounding to a depth of 30m requires at least a 60m linear array. If sufficient space is not available for this, it may be possible to use a 45m triangle array on the site or place a 100-200m long REMI array along an adjacent sidewalk or an "L" array at an adjacent street intersection.



Microtremor Measurements along Sidewalk

Modeling

There are several options for interpreting surface wave dispersion curves, depending on the accuracy required in the shear wave velocity profile. A simple empirical analysis can be done to estimate the average shear wave velocity profile. For greater accuracy, forward modeling of fundamental-mode Rayleigh wave dispersion as well as full stress wave propagation can be performed using several software packages. A formal inversion scheme may also be used. With many of the analytical approaches, background information on the site can be incorporated into the model and the resolution of the final profile may be quantified.

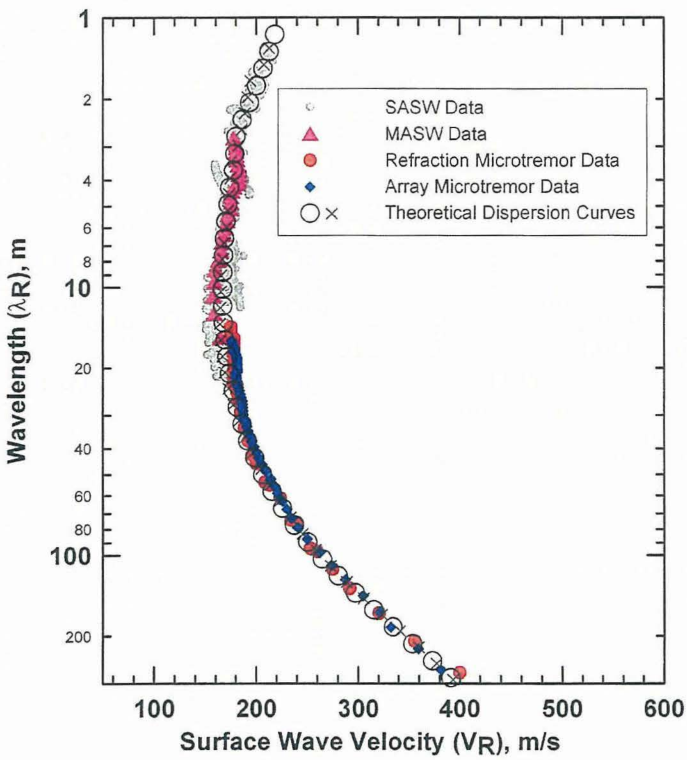
Applications

Active and passive surface wave testing can be used to obtain V_S profiles for:

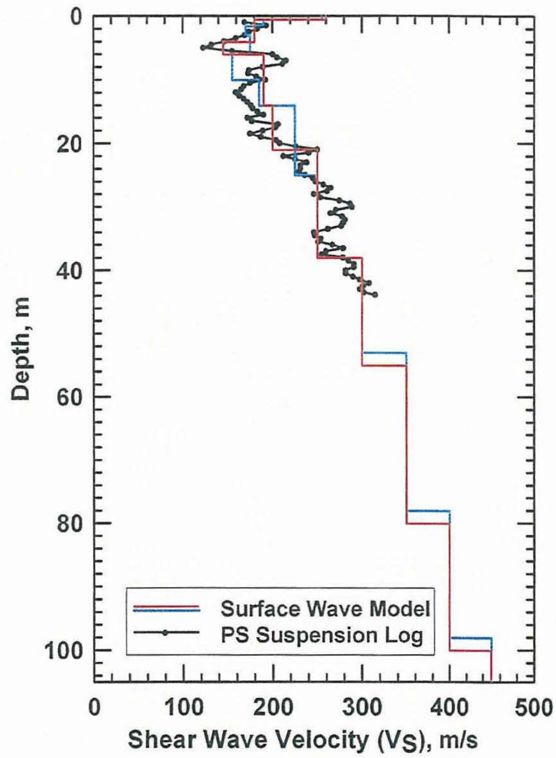
- UBC/IBC site classification for seismic design
- Earthquake site response
- Seismic microzonation
- Liquefaction analysis
- Soil compaction control
- Mapping subsurface stratigraphy
- Locating potentially weak zones in earthen embankments and levees

Case History

The figures below show the surface wave dispersion curves and alternative shear wave velocity models for a site in Los Angeles, California. All of the previous figures illustrating SASW, MASW, array and refraction microtremor techniques were from this site. The dispersion curves from all four methods are shown on the left along with the theoretical dispersion curves for alternative S-wave velocity versus depth models on the right. Conditions at this site were very poor for active surface wave techniques because of the presence of very low velocity hydraulic fill. In fact, with active surface wave techniques it was only possible to image to a depth of about 12.5m with energy sources typically capable of imaging to 30m. There is excellent agreement in the dispersion curves generated from all of the methods over the overlapping wavelength ranges. The minor differences probably result from variable velocity of the hydraulic fill within the sampling volume of the specific methods. Two Vs versus depth models were generated to illustrate the difficulty modeling the highly variable, near surface velocity structure evident in the PS log. The two surface wave models yielded similar values for the average shear-wave velocity of the upper 30m (V_S30), 201 and 202 m/s, illustrating that Vs30 is much more tightly constrained than the actual layer thicknesses and velocities in the models. V_S30 estimated from the PS log (194 m/s) is within 4% of that estimated from the two surface wave models (201 and 202 m/s). The small differences in V_S30 between the two methods may easily result from the different sampling regimes (borehole versus large area) rather than errors in either of the methods.



Field Data and Theoretical Dispersion Curve

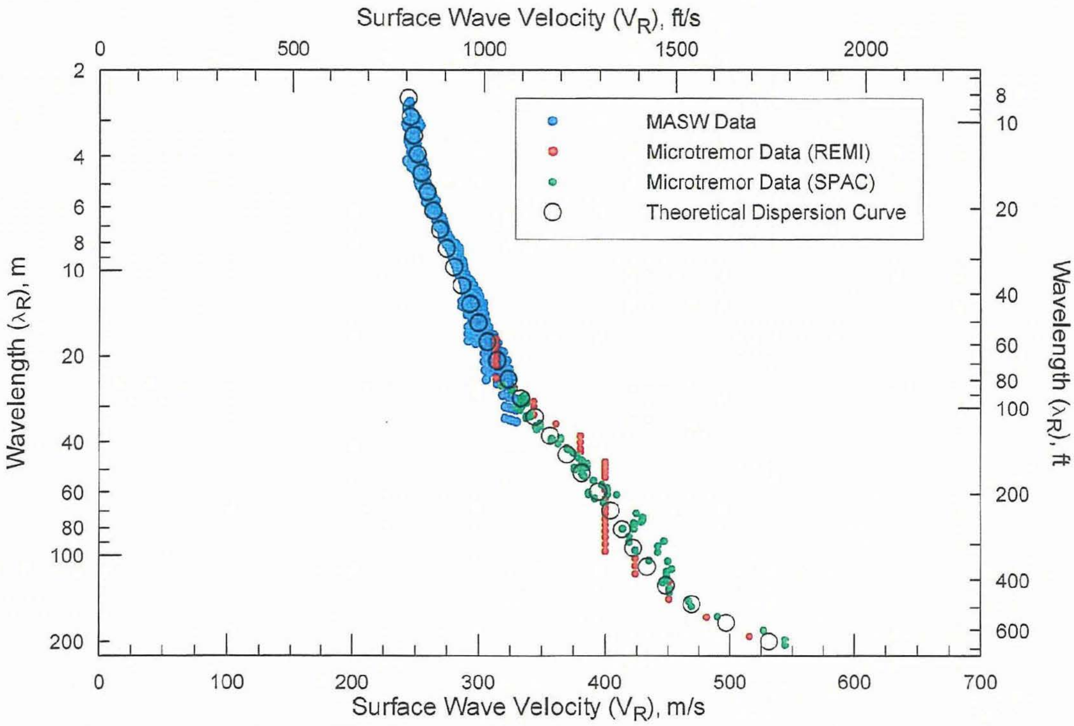


V_S Model

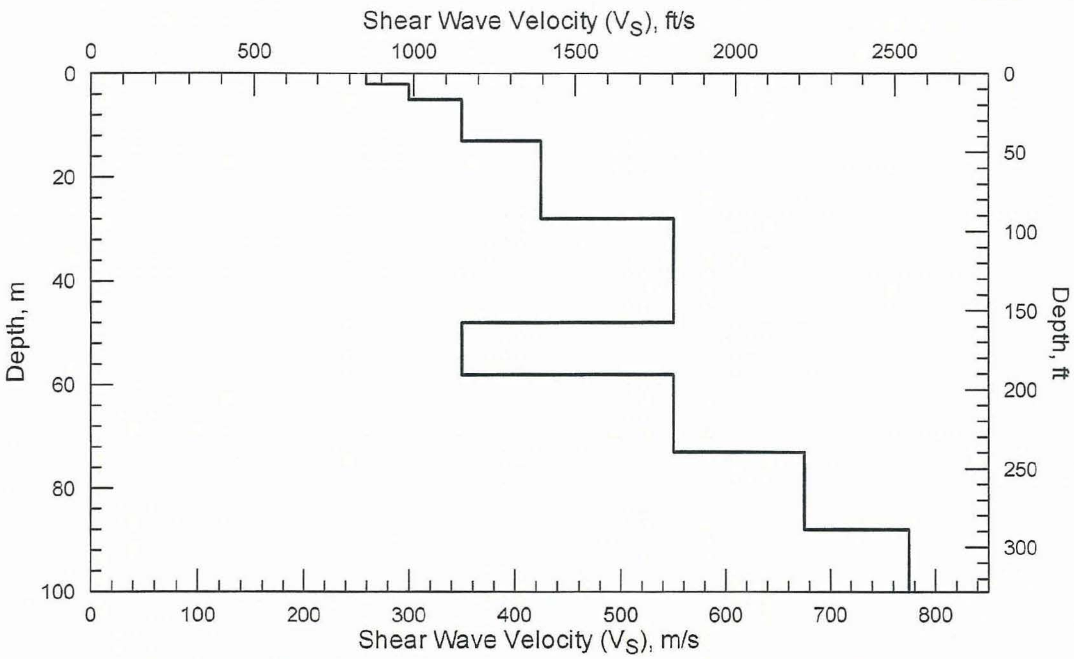
In contrast to borehole measurements which are point estimates, surface wave testing is a global measurement, that is, a much larger volume of the subsurface is sampled. The resulting profile is representative of the subsurface properties averaged over distances of up to several hundred feet. Although surface wave techniques do not have the layer sensitivity or accuracy (velocity and layer thickness) of borehole techniques; the average velocity over a large depth interval (i.e. the average shear wave velocity of the upper 30m or 100ft) is very well constrained. Because surface wave methods are non-invasive and non-destructive, it is relatively easy to obtain the necessary permits for testing. At sites that are favorable for surface wave propagation, active and passive surface wave techniques allow appreciable cost and time savings.

APPENDIX B

SURFACE WAVE MODELS

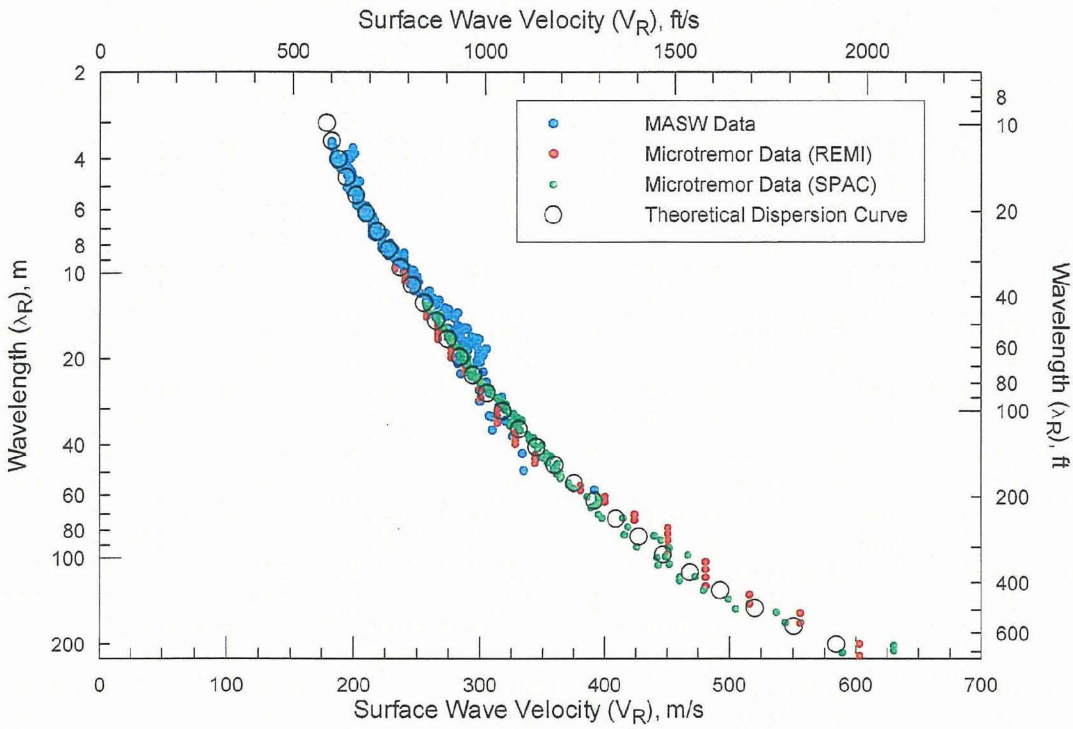


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array SW-B4, near Borehole B-4

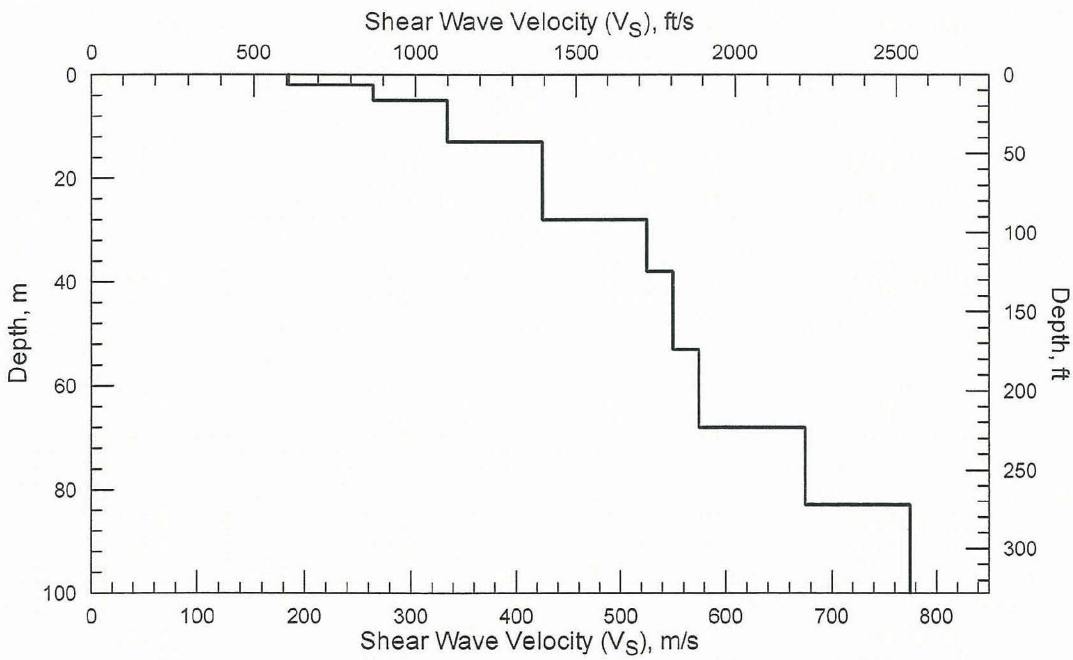


V_S Profile from MASW and Microtremor Array SW-B4, near Borehole B-4

FIGURE B-1

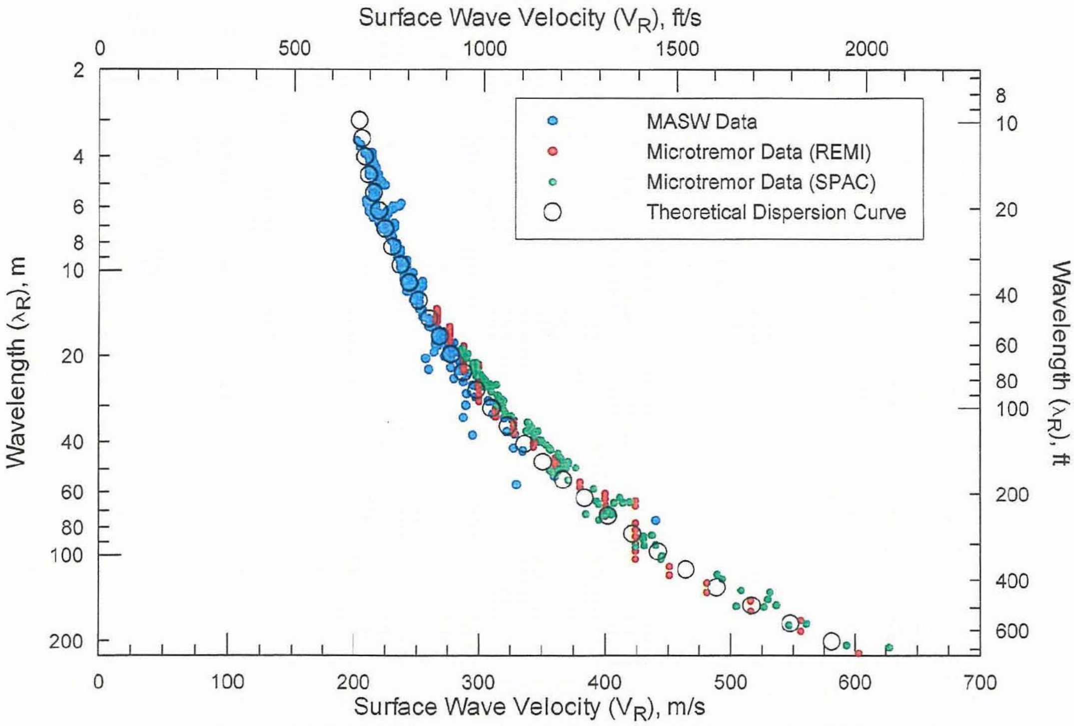


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line A - Station 226 ft

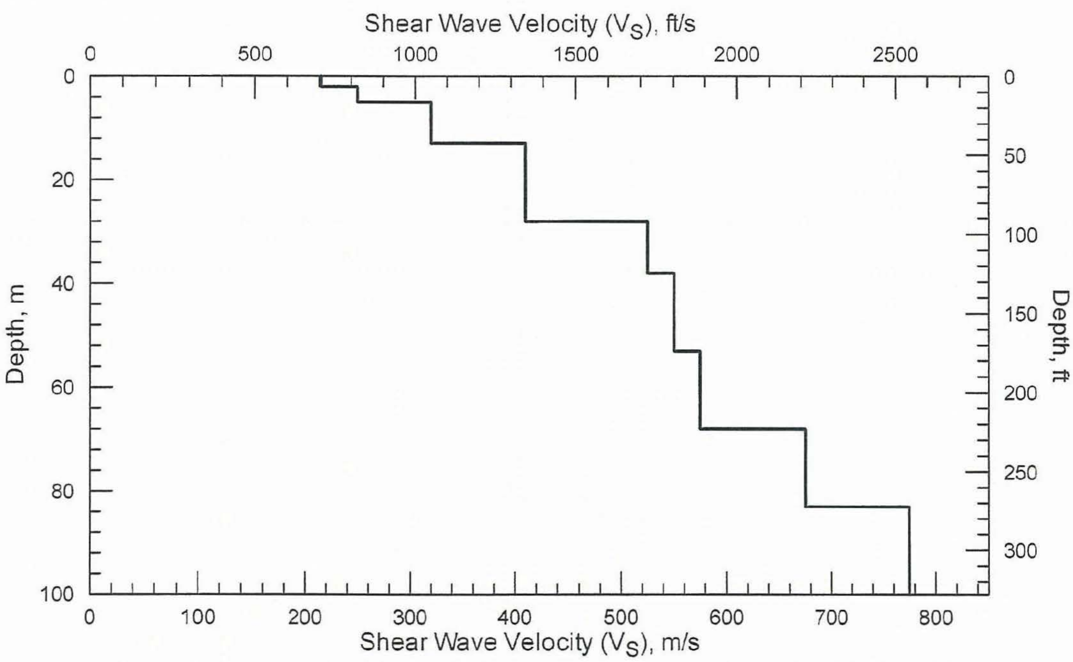


V_S Profile from MASW and Microtremor Array, Line A - Station 226 ft

FIGURE B-2

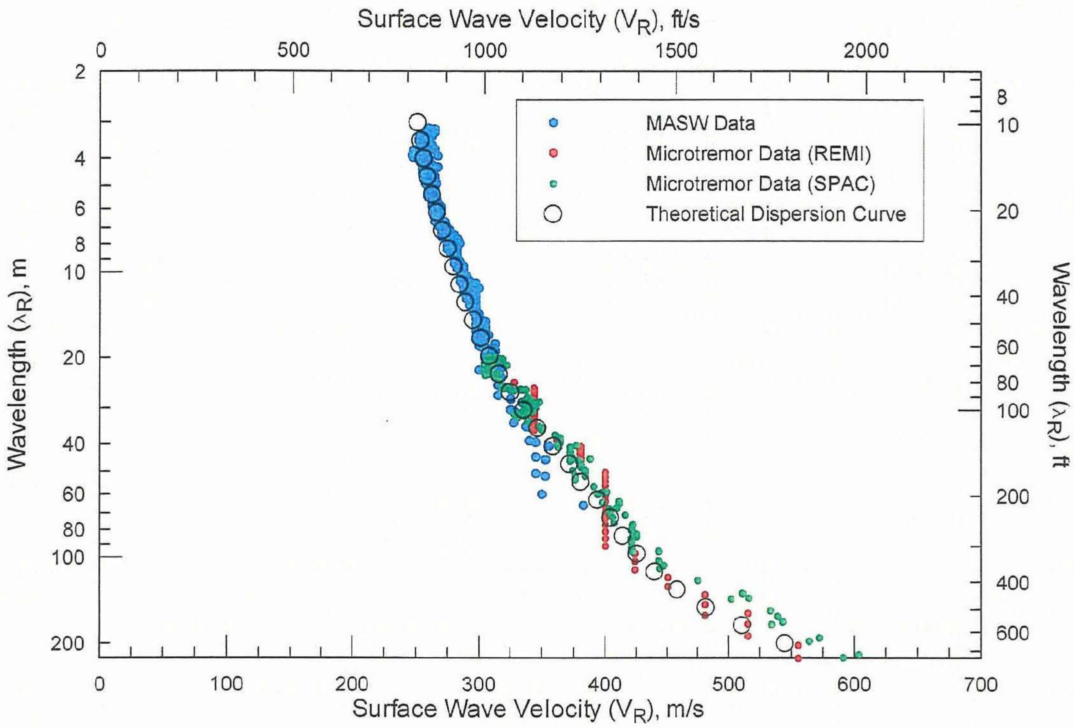


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line A - Station 463 ft

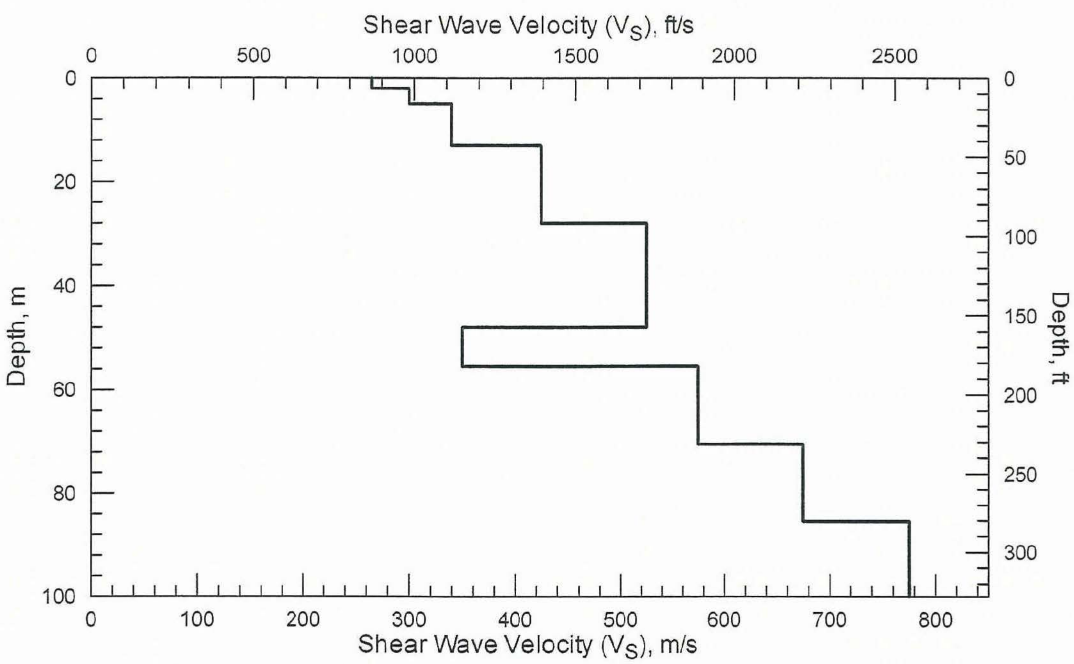


V_S Profile from MASW and Microtremor Array, Line A - Station 463 ft

FIGURE B-3

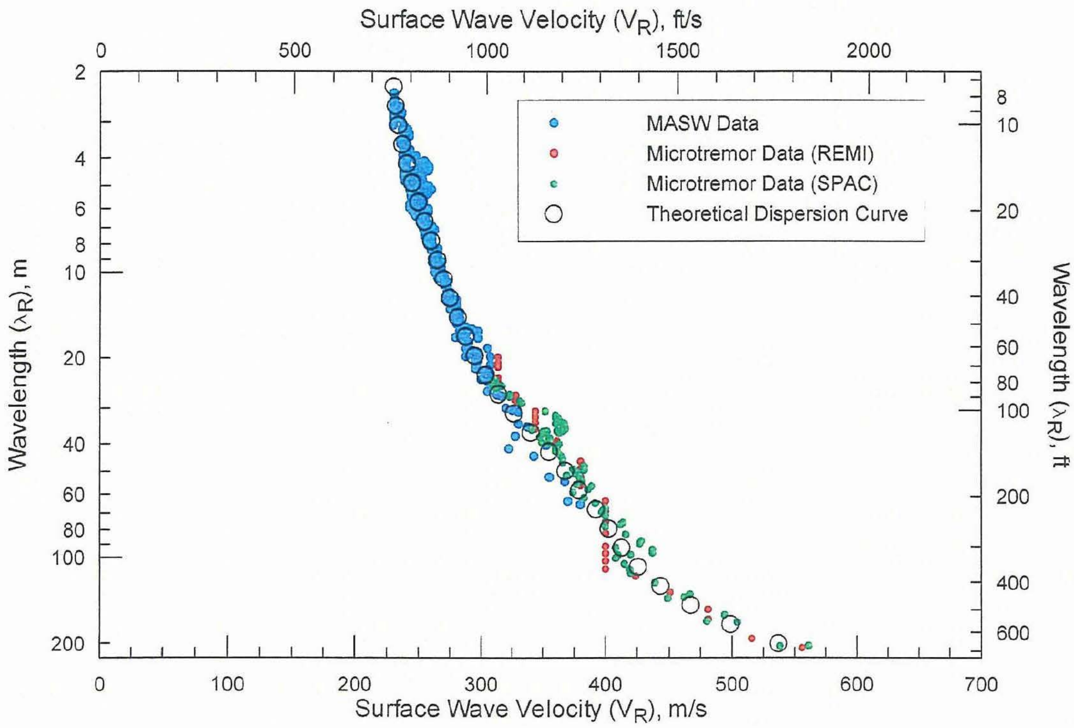


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line A - Station 699 ft

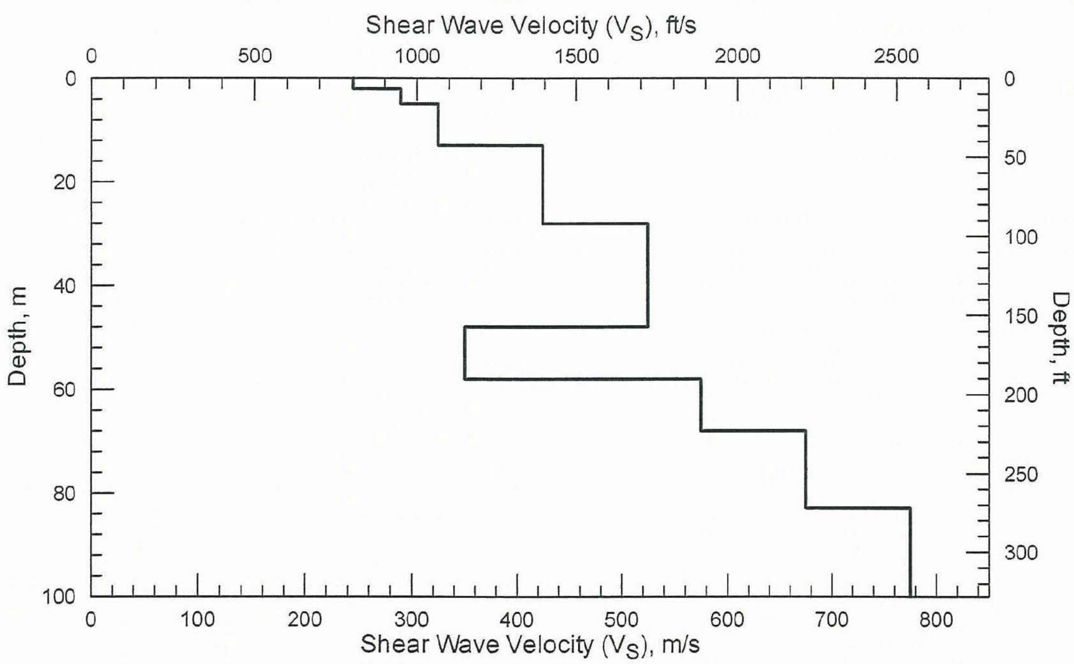


V_S Profile from MASW and Microtremor Array, Line A - Station 699 ft

FIGURE B-4

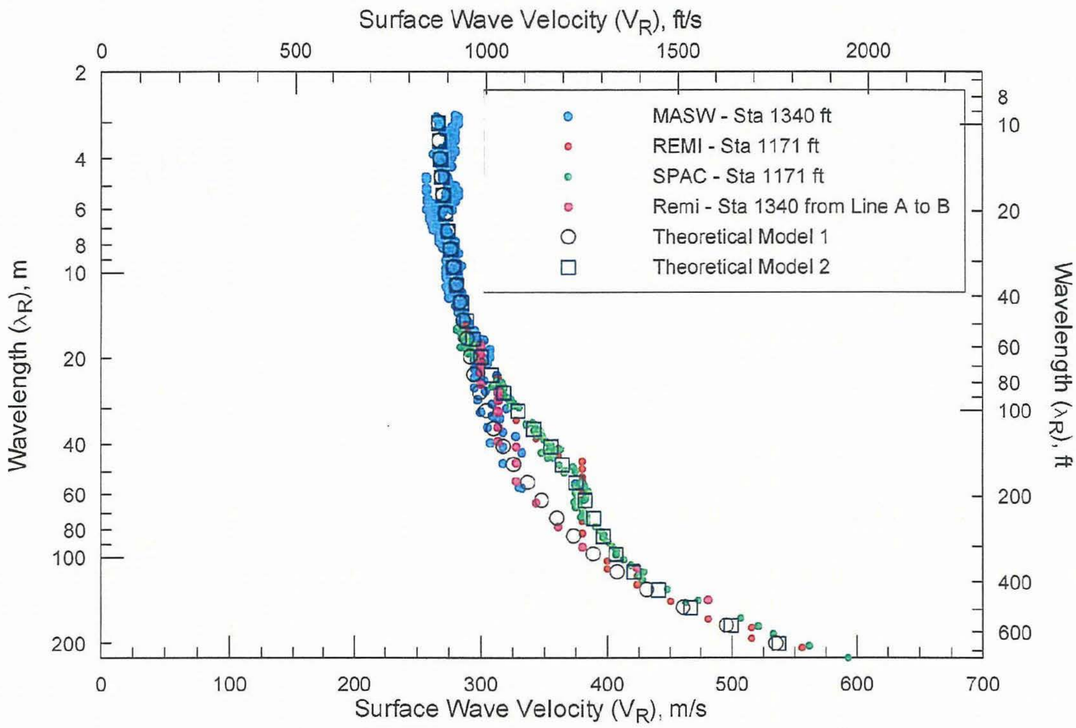


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line A - Station 935 ft

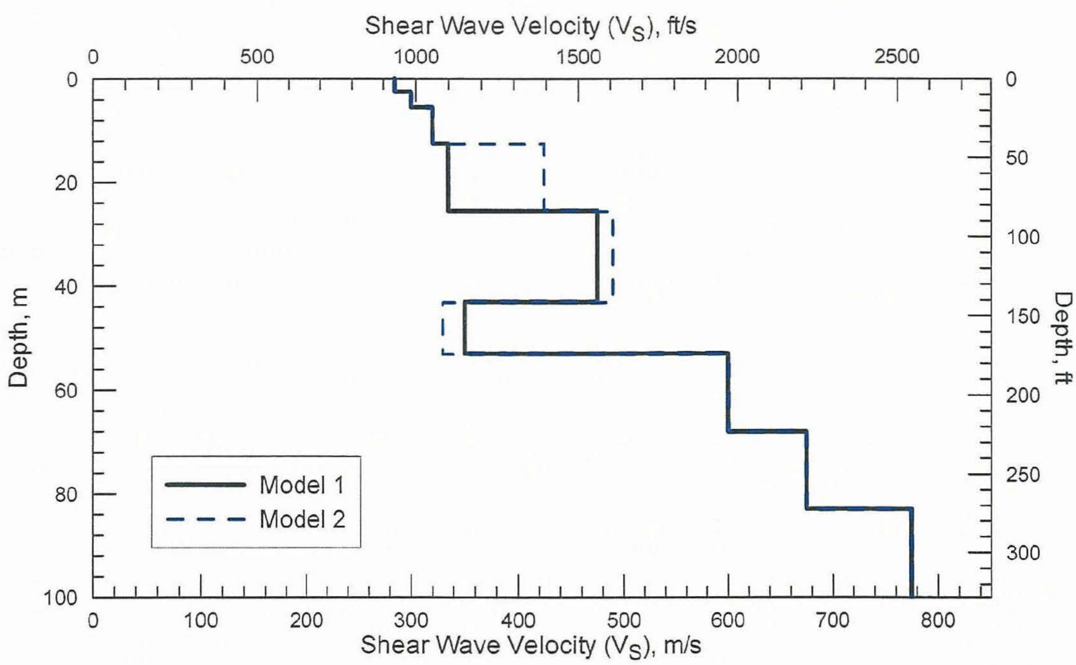


V_S Profile from MASW and Microtremor Array, Line A - Station 935 ft

FIGURE B-5

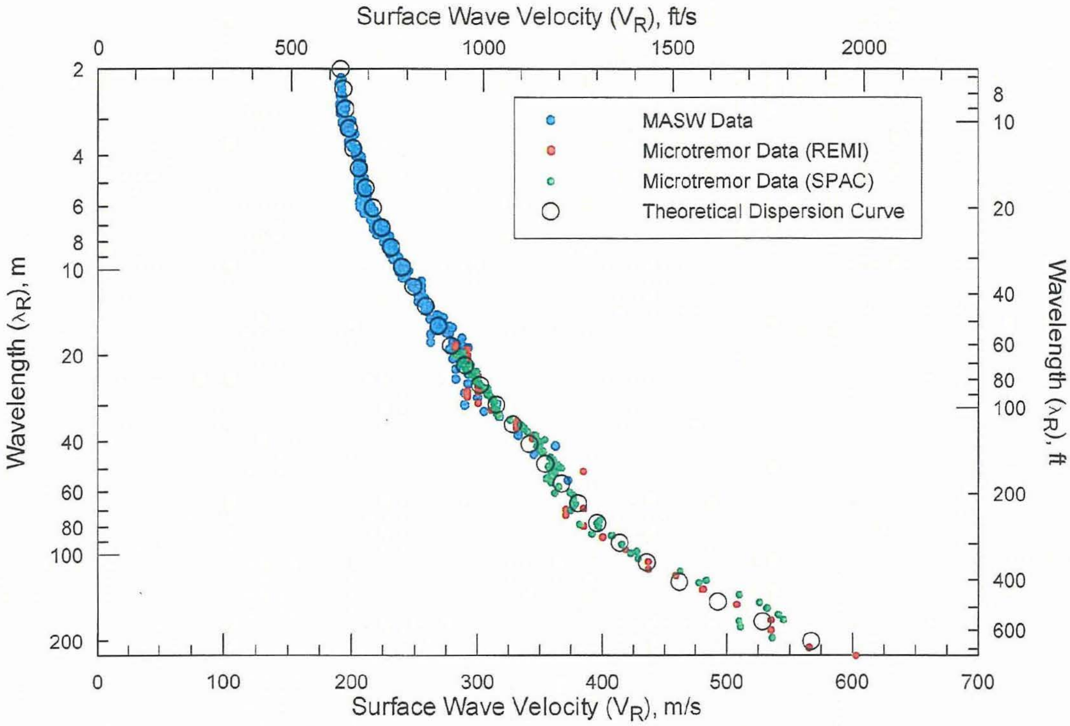


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line A - Station 1171/1340 ft

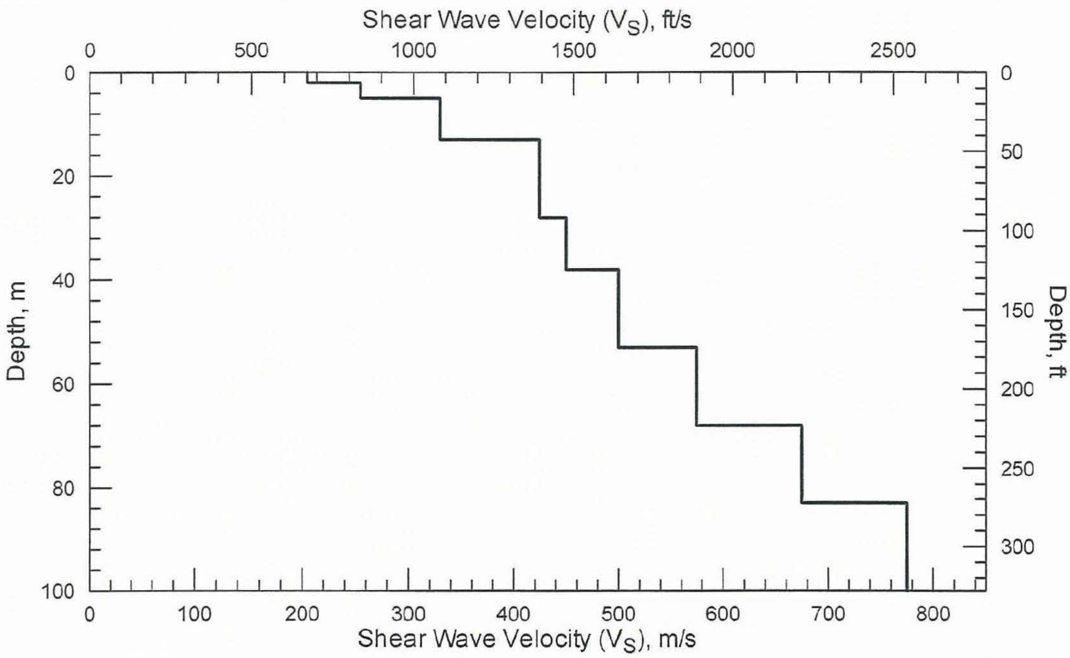


V_S Profile from MASW and Microtremor Array, Line A - Station 1171/1340 ft

FIGURE B-6

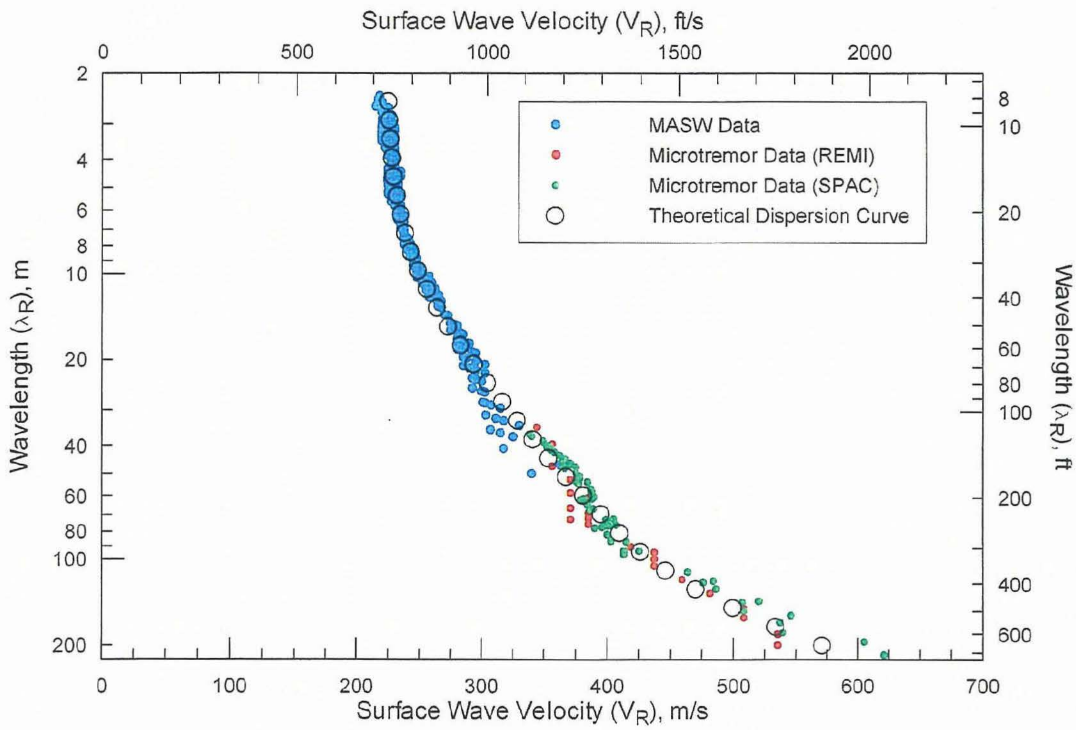


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line B - Station 226 ft

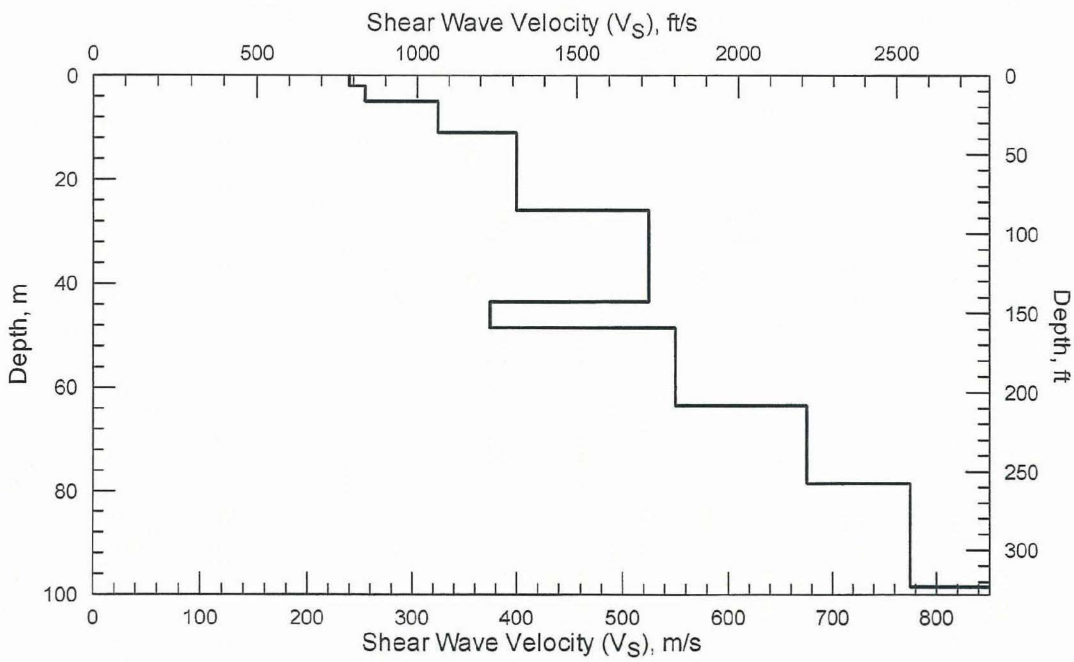


V_S Profile from MASW and Microtremor Array, Line B - Station 226 ft

FIGURE B-7

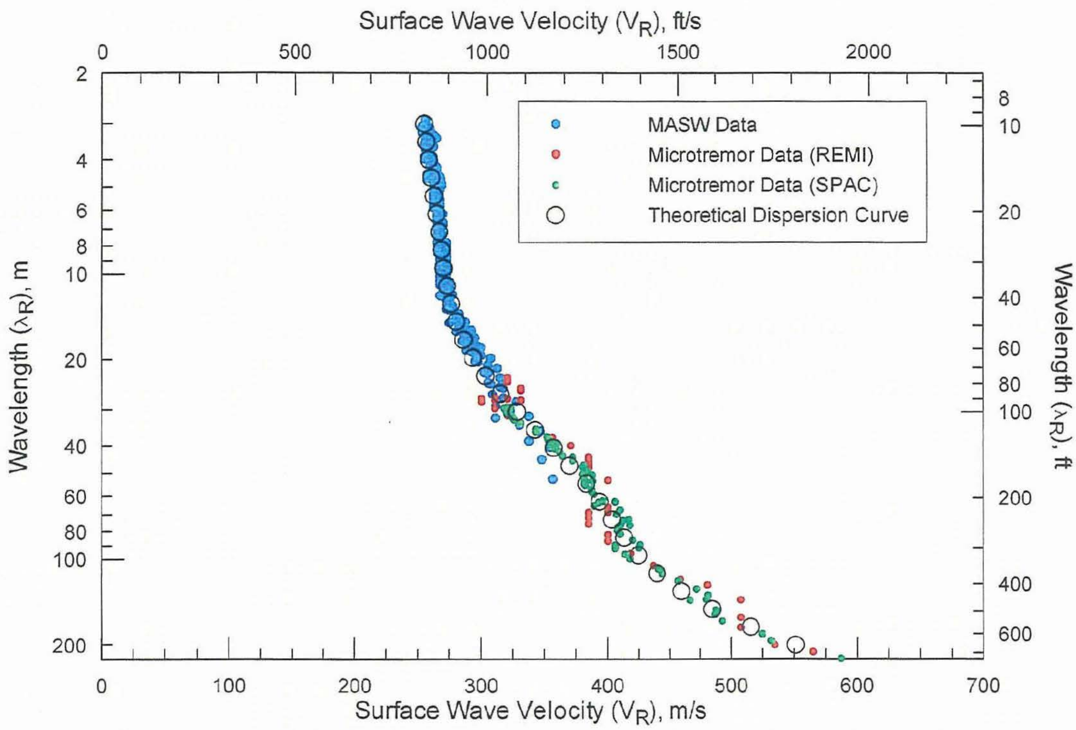


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line B - Station 463 ft

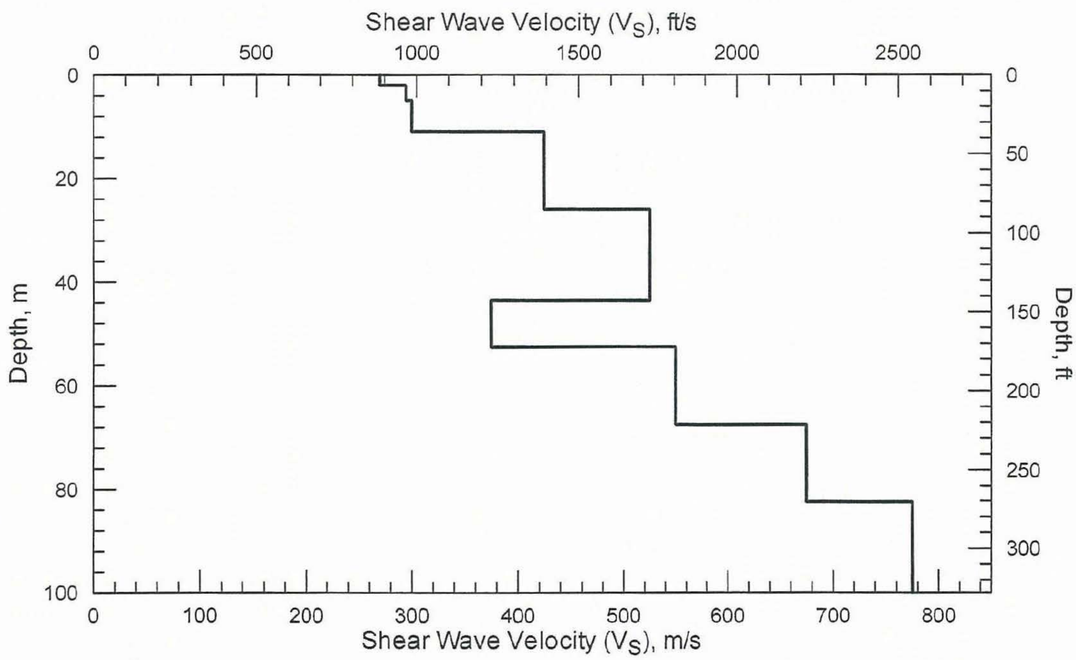


V_S Profile from MASW and Microtremor Array, Line B - Station 463 ft

FIGURE B-8

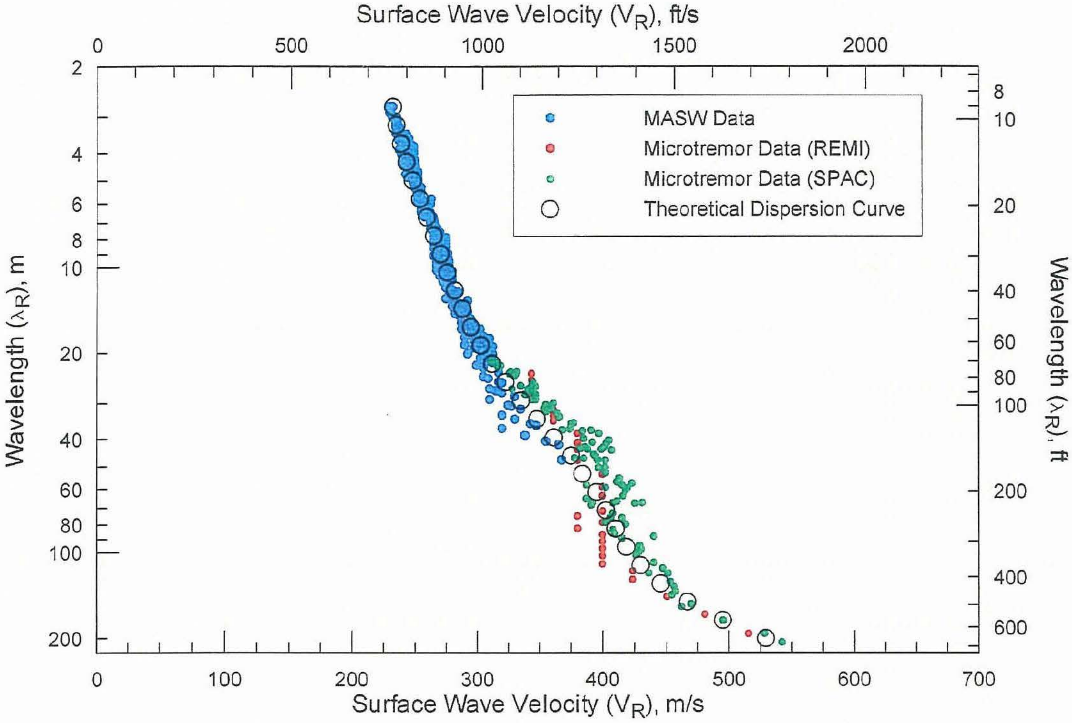


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line B - Station 699 ft

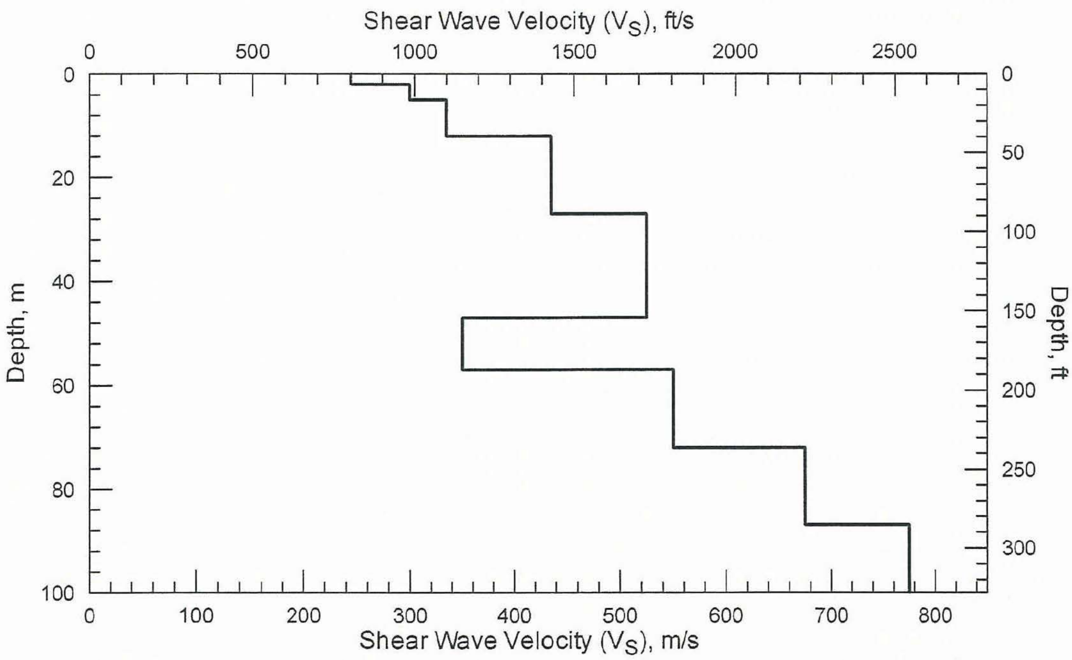


V_S Profile from MASW and Microtremor Array, Line B - Station 699 ft

FIGURE B-9

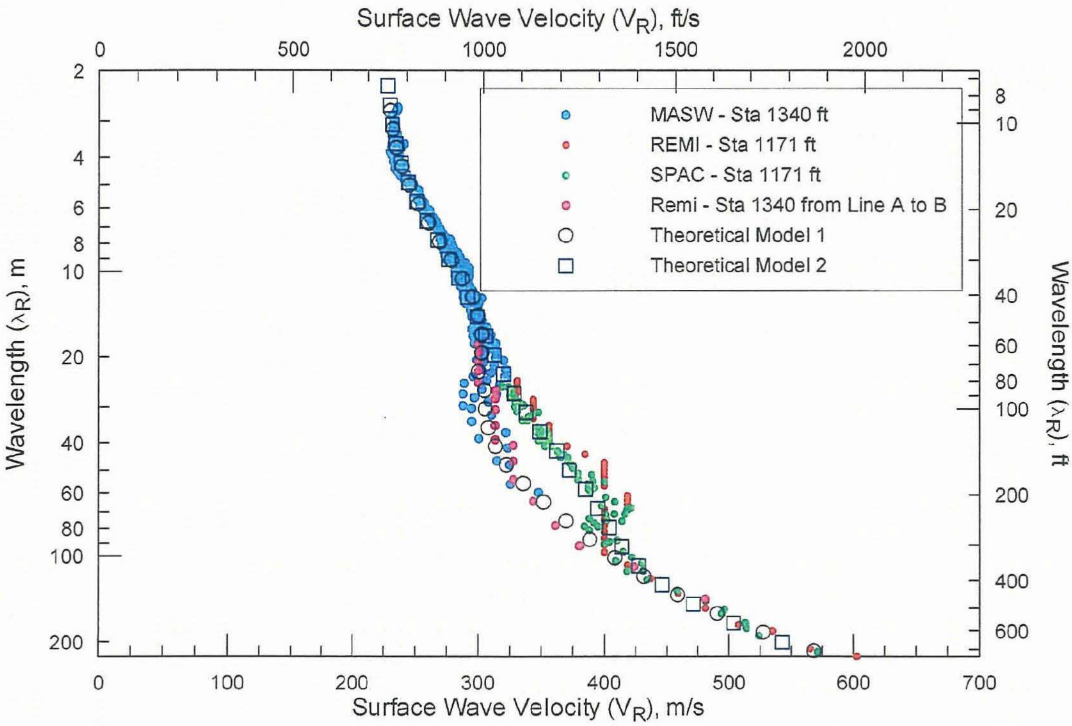


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line B - Station 935 ft

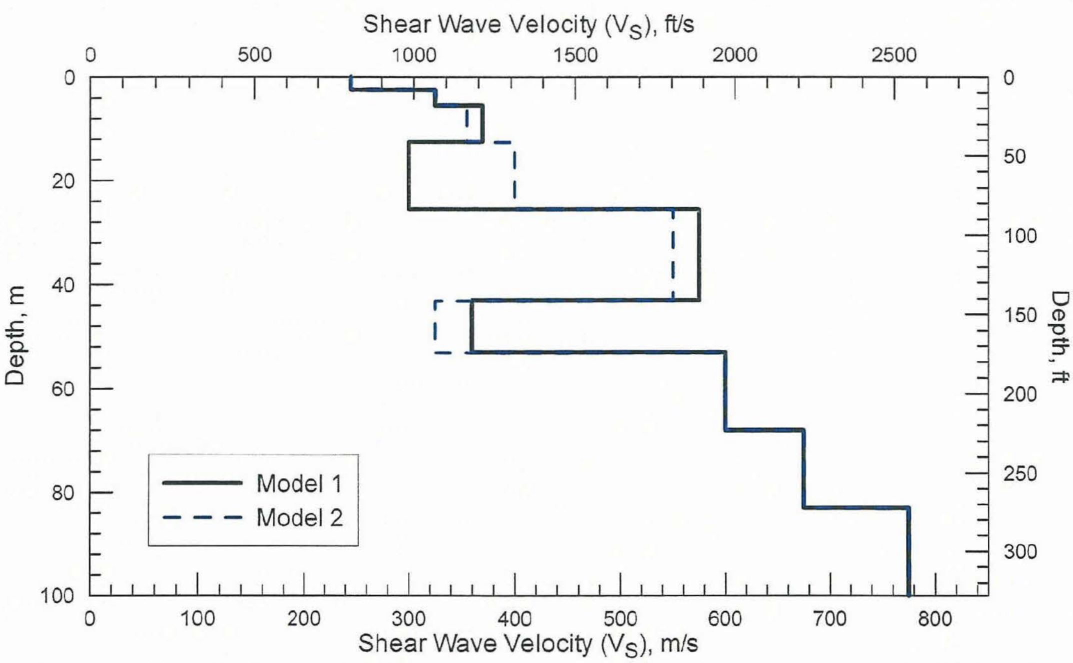


V_S Profile from MASW and Microtremor Array, Line B - Station 935 ft

FIGURE B-10

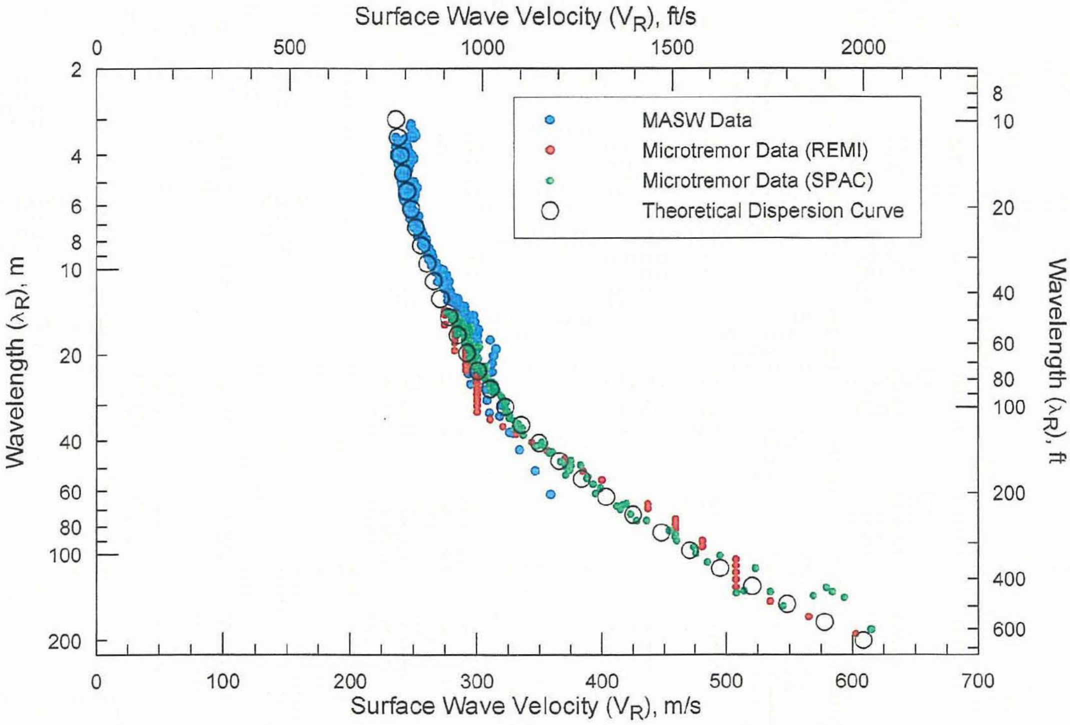


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line B - Station 1171/1340 ft

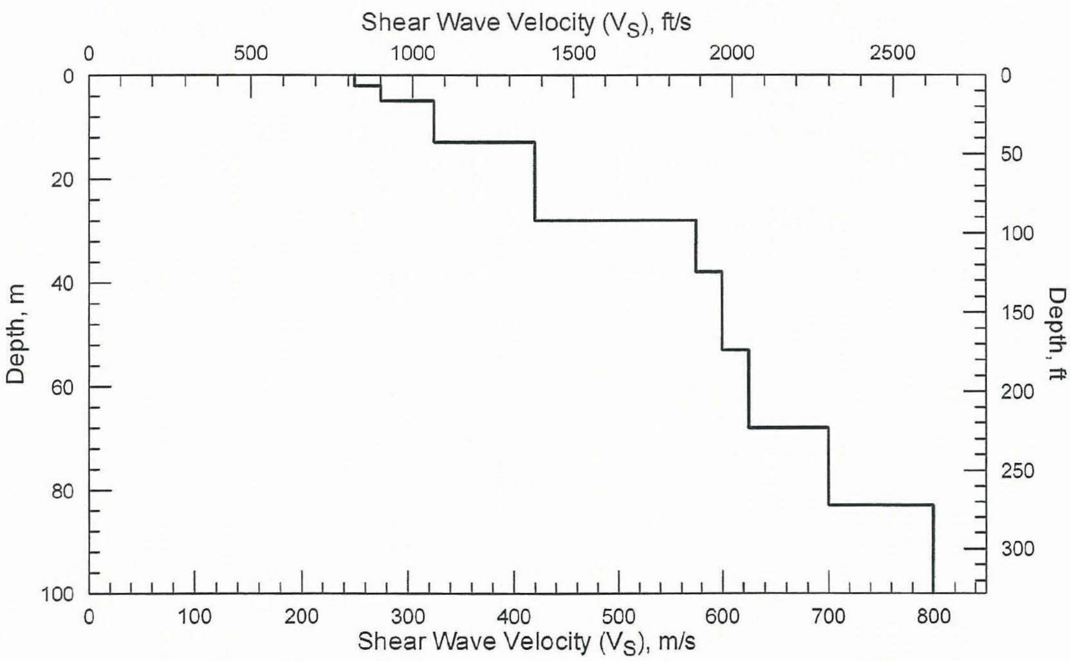


V_S Profile from MASW and Microtremor Array, Line B - Station 1171/1340 ft

FIGURE B-11

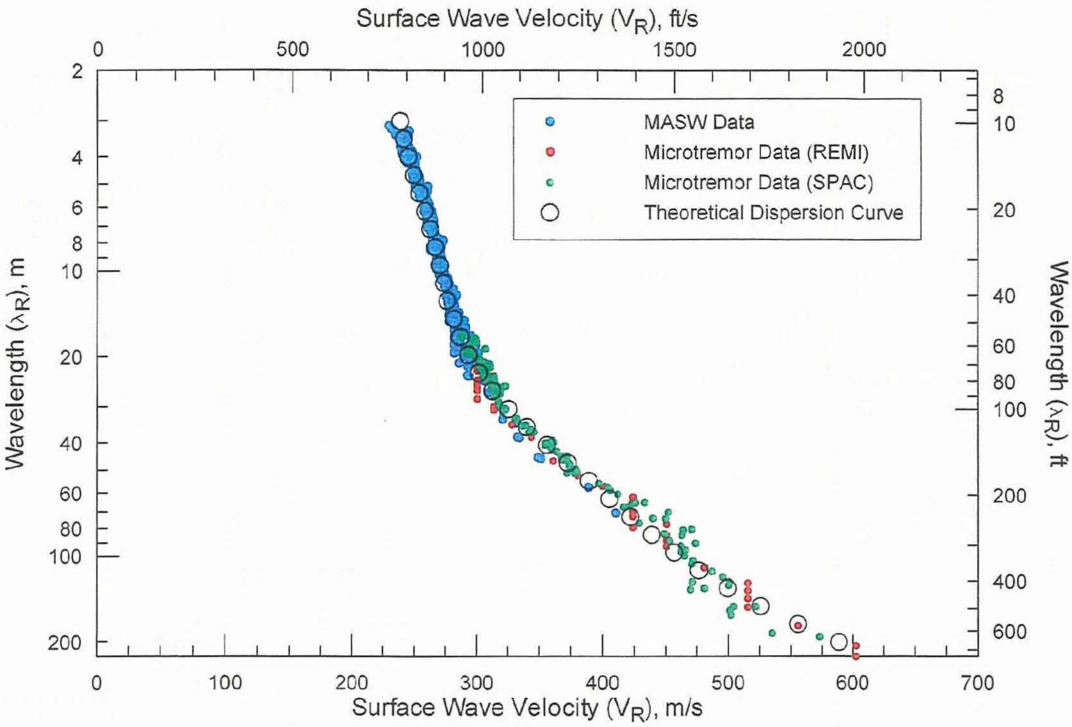


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line C - Station 226 ft

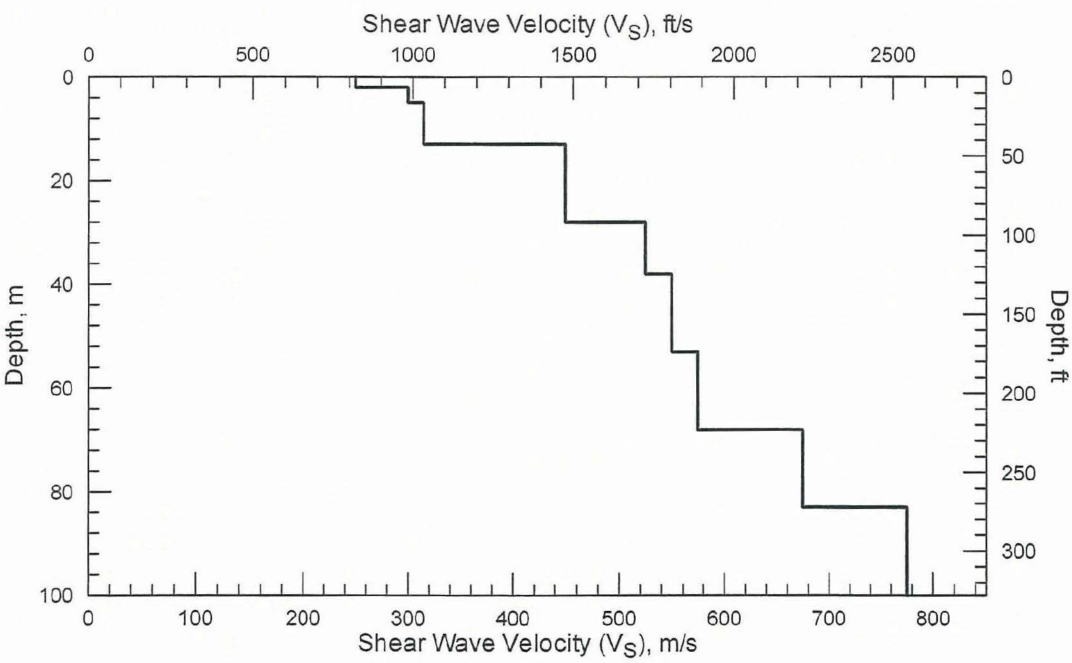


V_S Profile from MASW and Microtremor Array, Line C - Station 226 ft

FIGURE B-12

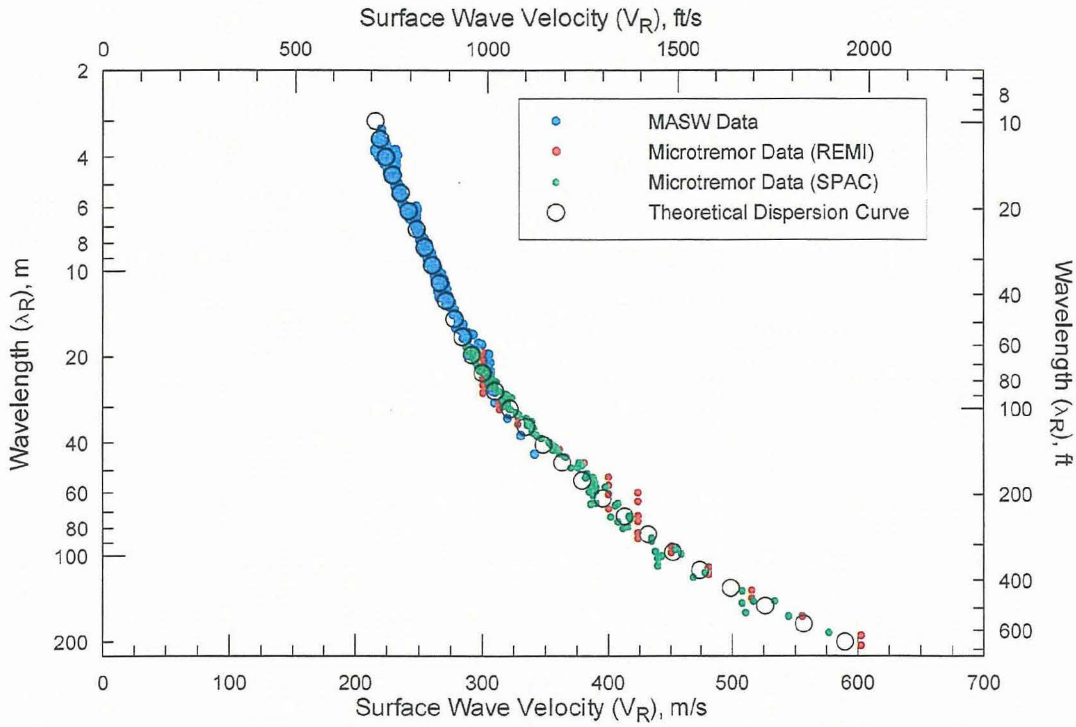


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line C - Station 463 ft

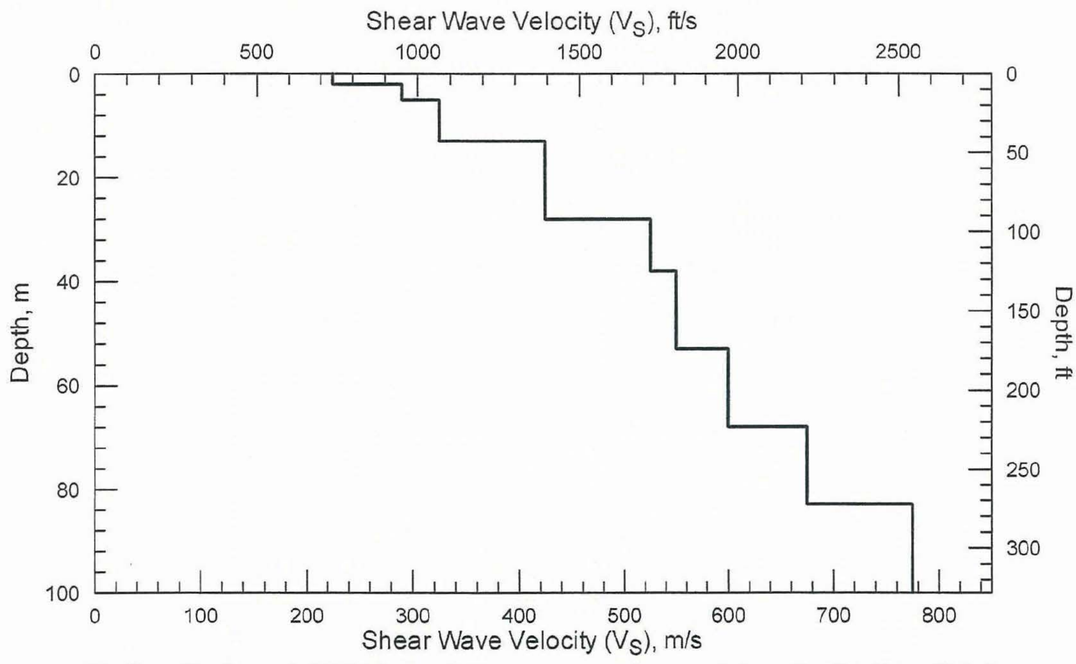


V_S Profile from MASW and Microtremor Array, Line C - Station 463 ft

FIGURE B-13

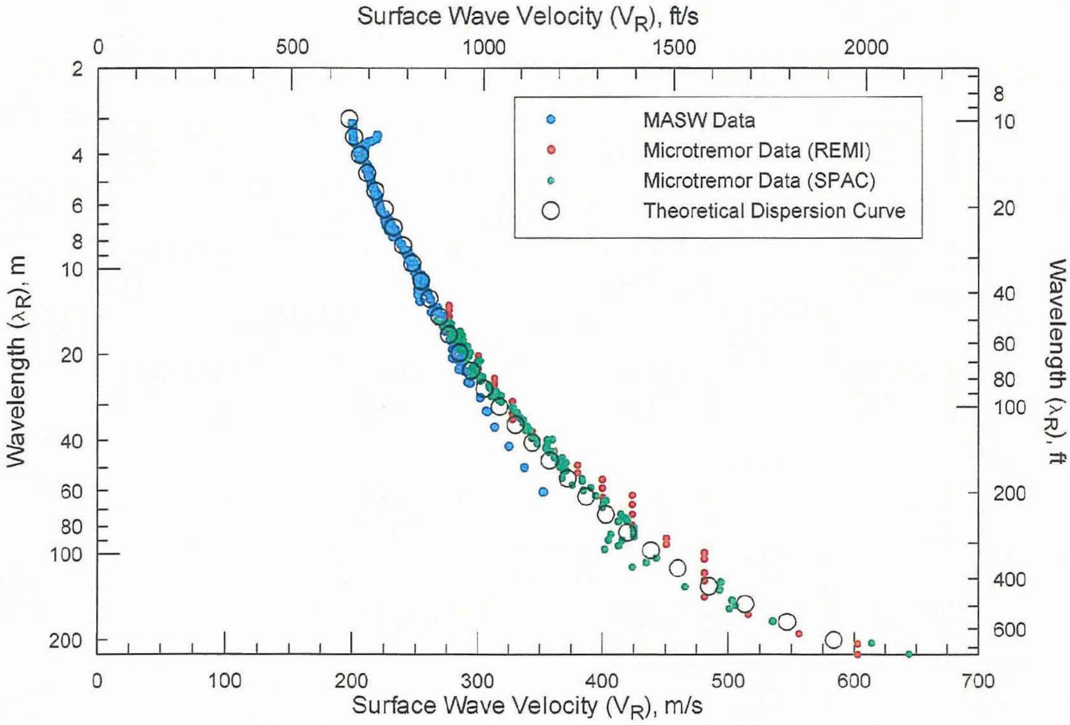


Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line C - Station 699 ft

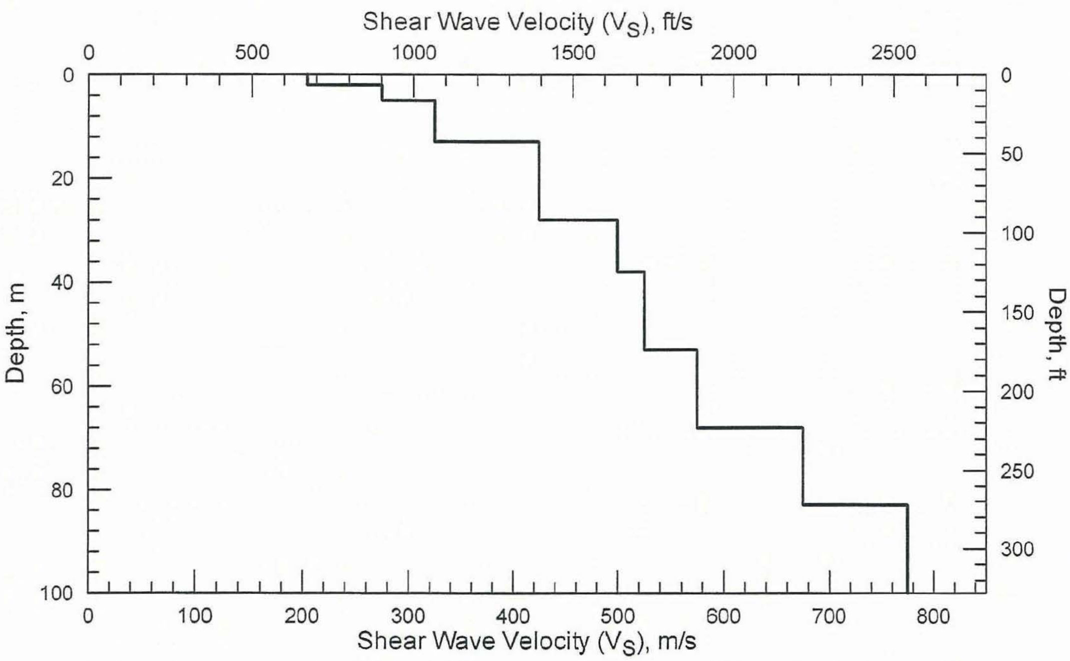


V_S Profile from MASW and Microtremor Array, Line C - Station 699 ft

FIGURE B-14



Comparison of Field Experimental Data and Theoretical Dispersion Curve from MASW and Microtremor Array, Line C - Station 935 ft



V_S Profile from MASW and Microtremor Array, Line C - Station 935 ft

FIGURE B-15

APPENDIX IV
PHASE I, II AND III REPORTS BY IRVINE GEOTECHNICAL

IRVINE GEOTECHNICAL Inc

NU-WAY LIVE OAK LANDFILL

EAST OF 605 FREEWAY, WEST AND SOUTH LIVE OAK LANE

AND NORTH OF LIVE OAK AVENUE

IRWINDALE, CALIFORNIA

FOR MNOIAN MANAGEMENT, INC.

IRVINE GEOTECHNICAL, INC. PROJECT NUMBER IC 07034-I

MARCH 31, 2008

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INTRODUCTION

This report has been prepared per our agreement and summarizes the preliminary findings of Irvine Geotechnical's Phase I geotechnical engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution and engineering properties of the earth materials underlying the site with respect to obtaining approval of the Mine Reclamation Backfill and developing the site with a commercial/retail development.

PHASE 1 - INVESTIGATION

The Phase I investigation plan was identified in our July 26, 2007 proposal, and was based on our initial site visit, review of records of the site, consultation with drilling and geophysical consultants, review of other exploration performed at a nearby landfill and consultation with the client.

Research

As part of our investigation, records and documents on file at the City of Irwindale and provided by the client were reviewed. The documents were scanned into an Adobe Acrobat (PDF) format and indexed for quick reference. It is our understanding that a copy of the scanned documents has been provided to the City of Irwindale on a Compact Disk. Historical topographic maps and aerial photographs before, during and after mining and filling were reviewed. Most of the historical photographs were provided by Kent McMillan, who is currently reviewing the mining and filling histories at the Nu-Way and nearby United Rock Products site for the City of Irwindale. Some of the photographs were scanned and scaled to match topographic maps and property boundaries to facilitate interpretation. Kent McMillan also provided a hydrograph that includes yearly groundwater elevations at the site extending back to 1932. The hydrograph was used to estimate the elevation of groundwater "lakes" visible in the air photos.

Subsurface and Geophysical Exploration

The site was explored between October, 2007 and February 2008 and included performing two seismic reflection line surveys, advancing six Becker Hammer borings, performing one downhole seismic shear wave survey and performing an active/passive surface wave survey.

The Becker Hammer borings were advanced by Great Western Drilling of Fontana using a new AP 1000 Becker Hammer rig. Between October 15 and 24, 2007 borings were advanced to depths of 120 to 190 feet using the "closed bit" method. Blow counts per foot and diesel combustion chamber pressure were recorded by staff of Irvine Geotechnical. The Becker Hammer borings are graphically logged on the enclosed Log of Borings. Because of the nature of the soils encountered in the borings, in-situ samples of the fill and alluvium were not obtained in this phase.

The locations of the borings are shown on the Geologic Map. Borings 2 and 3 were terminated just below the 1991, "pre landfill backfilling surface" and settlement monuments were installed. Boring 6 was advanced to near the base of the fill and a settlement monument was installed at a depth of 149 feet. A multi-stage gas vapor well was also installed in Boring 6 under the direction of Environmental Applications for future monitoring. A solid 3" diameter PVC casing was installed to the total depth of Boring 4, with the annular space filled with clean medium sand. The boring was used for the downhole shear wave survey.

The settlement monuments consist of 1 inch diameter, steel pipes that are connected by threaded pipe couplings. The base of the monument is secured in five feet of cement. C & M Duraflex, PVC centralizers were used to keep the pipe within the center of the boring. The centralizers were spaced about 12 to 15 feet apart from top to bottom. The lower 50 feet of the annular space was filled with clean, medium sand. The remainder of the annular space was filled with bentonite pellets up to the ground surface. The monuments were constructed from within the drill stem and backfilled as the stem was removed to ensure that caving did not occur. The tops of the monuments are protected by 12 inch thick, a cast-in-place concrete pads. Four inch diameter PVC sleeves extend through the pads to ensure that the pads can move (settle) independently of the monument pipes.

The seismic reflection lines and downhole shear wave survey were performed by Terra Physics, with interpretation assistance from Wilson Geosciences. The locations of the seismic reflection profiles are shown on the Geologic Map. The PVC casing installed in Boring 4 was used for the downhole shear wave survey. The procedures and results of the seismic reflection and downhole survey are contained in the Terra Physics report, "*Seismic Reflection and Borehole Seismic Velocity Surveys to Delineate Subsurface Backfill Material and Underlying Native Soil Boundaries Nu-way Reclaimed Aggregate Mine Landfill - Irwindale, California,*" which is appended to this report. Additional interpretation of the geophysical study and correlation between borings and geologic stratigraphy was performed by Wilson Geosciences (*Technical Report: Seismic Reflection and Borehole Seismic Investigation: Nu-way Live Oak Landfill Reclamation, Northeast of the Interstate 605 Freeway and Live Oak Avenue Intersection, Irwindale, California*).

GeoVision performed surface wave soundings and created shear wave velocity profiles through three areas of the landfill. The results of the surface wave study are contained in the GeoVision report, "*Geotechnical Investigation, Nu-Way Reclaimed Aggregate Mine, Irwindale, California,*" dated February 25, 2008. The surface wave study included collecting 1-D surface wave soundings at 230 foot intervals along three profiles totaling about 3,700 linear feet. The purpose of the surface wave soundings is to provide 2-D shear wave velocity models of the upper 200 to 300 feet along each profile. Both active (spectral analysis of surface waves [SASW] or multi-channel analysis of surface waves [MASW]) and passive (array or ReMi) were used. The active techniques were able to image the S-wave velocity of the upper 100 to 130 feet, while passive techniques extended the depth of investigation past 300 feet. The 2-D shear wave profiles are in the shape of a large triangle as shown on the Geologic Map. Seven of the velocity profiles are plotted on Sections A and C.

The surface wave survey was added to the Phase I exploration program to offset two of our planned Phase I techniques that could not be implemented. Downhole wire line logging, which was to include video and compensated density, could not be implemented due to caving. All of the borings caved within minutes to days of pulling the drill stem. It was considered too risky to lose a radioactive source and/or an expensive camera system in the ground. A subsequent exploration phase may attempt downhole logging as the drill stem is removed.

SITE DESCRIPTION - HISTORY

The study area is located in the western portion of the City of Irwindale, California (117.976W; 34.110N) and consists of approximately 65 acres of a mostly level, former gravel pit that has been filled and is known as the "Nu-Way Live Oak Landfill." The Nu-Way site is located just south of Arrow Highway, southeast of the Santa Fe Flood Control Basin, west of the San Gabriel River channel and north of Live Oak Avenue. Nu-Way is bounded by the 605 Freeway on the west and Live Oak Lane and industrial properties on the east and north, respectively. Elevations range from about 408 to 410 feet along the eastern portions of the property to 375 feet in a basin along the western edge of the pad and within the Southern California Edison easement.

Geomorphically, this area of Irwindale is characterized as a gently, south-southwest sloping alluvial fan that emanates from San Gabriel Canyon. The alluvial fan is comprised of sand and gravel deposits that have been historically mined for construction aggregate.

Mining within the study area started in the late 1950's by the Owl Rock Company. Owl Rock did not own the entire Nu-Way site and the boundary between the Owl Rock (east) and Blue Diamond (west) properties trended north-south and nearly bisected the study area. The majority of the Blue Diamond property extended westerly, beyond what was to become the 605 Freeway, to near the intersection of Arrow Highway and Live Oak Avenue. Mining was performed solely on the Owl Rock property (eastern portion of Nu-Way) until 1962. In 1962, mining commenced in the western portion of the Nu-Way pit, with material moved by conveyor belts westerly toward the Blue Diamond processing area. The Owl Rock and Blue Diamond properties were mined independently until the mid-1960's, when the pits merged.

From 1957 to the mid-1960's, waste material (silt) from the Owl Rock operation appears to have been disposed of offsite and north of the limits of the Nu-Way pit. Waste material generated from mining on the Nu-Way site by Blue Diamond appears to have been disposed of in silt ponds near the intersection of Arrow Highway and Live Oak Avenue. After the mid-1960's, all waste materials from mining appear to have been disposed of within and north of the Nu-Way site. The 605 Freeway had been graded in 1970, formally separating the Blue Diamond and Owl Rock properties. However, access beneath the freeway between the two pits was preserved. In the mid-1970's, permission was granted to dispose of liquid waste material (silt) within the Nu-Way pit. Mining within the Nu-Way site was mostly complete by the mid to late-1980's. A Liquid Waste Permit was obtained from the Water Quality Control Board in 1985 to allow the placement of imported silt in the pit. The "waste" silt was reportedly dredged from active pits on the west side of the 605 Freeway and transported as a slurry beneath the freeway and placed into the pit, predominantly in the southern portion of the study area. Reportedly, 800,000 cubic yards of silt slurry was accepted at the site, mostly within the southwestern half.

The waste permit was amended in 1990 to include inert materials such as: concrete, bricks, rocks, asphalt, ceramics, sand and non-contaminated soils. Drywall was originally accepted within the landfill and later rejected. Although, dry wall recycling and processing was apparently conducted onsite until a much later date. A geotechnical study was performed by Zeiser Geotechnical (later became Zeiser-Kling Consultants, Inc. and referred to herein as "Zeiser") to provide recommendations for placing and compacting fill into the pit. Zeiser reported 5 to 40 feet of existing fill throughout the base of the pit, which was around elevation 280 to 285 feet (120 to 130 feet below existing grade). Borings were not drilled in the southern portion of the "silt pond," which was present in the southwestern corner of the pit. The locations of the Zeiser 1991 borings are shown on the Geologic Map and copies of their boring logs are appended to this report. Zeiser provided specific recommendations for the placement of engineered fill and measures to ensure quality control.

Along with active filling, additional mining was performed in the early 1990's. Primarily, the mining consisted of "pushing" the slopes toward Live Oak Lane, Live Oak Avenue and the 605 Freeway. Starting at the pit boundaries, slopes were trimmed down at a 1:1 (horizontal:vertical) or steeper gradient. The lower 10 to 20 feet of the trim was made vertically.

The City of Irwindale granted a Conditional Use Permit on December 15, 1994 to operate the landfill. Between 1992 and 2005 Zeiser performed periodic geotechnical observations and testing. The results of the compaction testing and a description of the grading observed at the time of the site inspections are contained in numerous Zeiser field notices and file documents, which are contained on the CD.

The records indicate that Hushmand and Associates replaced Zeiser-Kling as the geotechnical engineer of record in 2005 for the placement of the "clean" compacted fill cap. Fill placed from 2005 to present was performed under the geotechnical supervision of Hushmand. The results of compaction testing by Hushmand are contained in their report, *Construction Quality Assurance Services Nu-Way Live-Oak Landfill, Irwindale, Los Angeles County, California*, dated March, 2007.

Groundwater lakes are visible on the historical photographs. It is our understanding that active mining operations would extend to and stop at the groundwater table. Relative low groundwater years would allow for deeper mining. The limits of the groundwater lakes, combined with the hydrograph data, was used to estimate the approximate maximum depths of mining. During the mining period, relative groundwater elevation lows occurred in 1964-1965 (210 feet) and 1978 (205 feet). Groundwater was encountered in this exploration in Boring 4 at an elevation of about 242 feet (165 feet below ground surface).

LIMITS AND THICKNESS OF FILL

One of the major tasks of this Phase I investigation was to determine the thickness and distribution of earth materials. This was accomplished using: the seismic reflection surveys; Becker-Hammer borings; Zeiser-Kling's 1991 borings; historical topographic maps; historical photographs and home videos; and the surface wave profiling. The interpreted geologic profile across three areas of the Nu-Way pit are shown on Sections A, B, and C. The maximum depth of fill appears to be around 180 to 185 feet below ground surface as shown on the cross sections.

The March 1984 topographic survey by Hekimian - Van Dorpe Associates, which was used by Zeiser-Kling as the basis for their 1991 Geologic Map, was assumed to roughly represent the pre-controlled fill conditions in the pit. Fill deposits (silt waste, soil and rubble) and with elevations lower than the contours shown on the Zeiser-Kling Geologic Map, are certainly "uncontrolled fill" or "unpermitted fill." It should be noted that this topographic map, predates the 800,000 cubic yards of silt that were accepted at the site in 1987/1988. Also, based upon aerial photographs, some additional mining (on the south) and "dumping" of fill (on the north) appears to have been on going up until 1990. The Zeiser borings contain elevations of the top of the boring. It is presumed that the elevations for the top of borings were checked against a known datum elevation.

Home videos between 1990 and 1993 show significant slope trims along the eastern, southern, and western margins of the pit. Processing and exporting of aggregate were occurring in early stages of controlled filling. Slope trims shown on Section B and C were estimated based upon the video evidence.

The aerial photos that showed "groundwater lakes" were used in determining limits and minimum depths of mining. The approximate elevations of the lakes were estimated from the hydrograph. The limits of the lake with a corresponding elevation were then plotted onto the base topographic survey, with the composite used to define the minimum depths and extent of mining.

The seismic reflection lines mostly coincide with Sections A and C. Reflectors that represent the base of the fill and the 1991 surface were found in the data record. The reflector that represents the top of the native alluvium correlates well the data collected from the borings, photo interpretation and the surface wave profiles. For the most part, the seismic reflectors are the basis for showing undulating and relatively deeper sections of the fill. Sloping reflectors were also interpreted within the fill. At first glance, these reflectors were believed to represent sloping fill surfaces or "lifts." However, this interpretation was discounted as they appear to be over 100 feet high in places. Because of assumptions made for the reflection survey, and the much higher than anticipated velocities in the fill, the depth of imaging extended more than 2,000 feet below the ground surface. As a result, the data record that represents the depth of the landfill is only a fraction of the collected data. Refinements to the seismic reflection data collection methods in subsequent studies, if any, may yield more or better detail.

Surface wave soundings and resulting shear wave velocity models found a velocity inversion beneath the central to north-central portion of the pit. The thickness and distribution of the lower velocity layer appears to correlate with the lower, "uncontrolled fill" found by Zeiser. The lower velocity layer also correlates with lower Becker-Hammer blow counts near the base of the fill. Surface wave profile Line A roughly coincides with cross Section C, while Profile B crosses Section A. The two-dimensional velocity profiles along Line A are superimposed onto the cross section. The base of the lower velocity inversion zone correlates well with reflectors interpreted from the Seismic Reflection study and is believed to represent the base of the fill. The lower velocity zone terminates or pinches out toward the south, north of Line C. Elevations that represent the base of the lower velocity zone from Lines A, B and C were contoured and used to interpret the base of fill in the central portion of Section B.

Because the surface wave study generates an "averaging" effect near boundaries, the contacts appear to be linear and diffused. The undulating contact between the base of the older fill and the alluvial deposits shown in some of the cross sections was interpreted from the reflection lines.

BECKER-HAMMER BORINGS

Becker-Hammer borings 1 through 5 were advanced to the planned depths using the "closed bit" method. Boring 6 was advanced to about 100 feet with a closed bit, where refusal was met in what appeared to be drywall debris. The drill stem was tripped out and the boring advanced to the planned depth using an open bit. The Becker-Hammer blow counts were corrected for bounce pressure chamber pressure using the methodology of L. F. Harder and H. B. Seed, 1986 (*Determination of Penetration Resistance for Coarse-Grained Soils Using The Becker Hammer Drill*). For the most part, due to the high resistance of the fill material, the bounce chamber pressures were higher than 26 psi and only minor corrections were required. The corrected blow counts and bounce chamber pressures are shown graphically on the Log of Borings.

The blow counts are variable with depth as would be expected with a rubble fill. However, there is a definite increase with depth the correlates with apparent fill quality. Becker-Hammer blow counts were also converted to equivalent SPT N_{60} blow counts using methodology of Harder and Seed. The estimated compactness of the materials encountered in the borings is shown graphically on the NAVFAC Density graphs (From Figure 1 for sands and gravels, page 7.1-14 of the NAVFAC Design Manual). The SPT N_{60} blow counts plotted against Effective Stress Vertical show the materials to be "normally consolidated" with compactness generally between medium dense and dense.

The SPT N_{60} blow counts were also estimated using the shear wave velocity profiles and correlations between shear wave velocity and SPT N_{60} blow count values published by the Department of Defense, (*Soil Dynamics and Special Design Aspects*, MIL-HDBK-1007/3, Figure 2, November 15, 1997). The SPT N_{60} blow counts from the borings are compared to those from the shear wave velocity profiles on the Blow Count Comparison charts. The shear wave velocity intervals assigned to Borings 1 through 6 were based upon the nearest shear wave velocity profile station from the surface wave study. There is very good correlation between SPT N_{60} blow counts determined from the Becker-Hammer borings and inferred from the surface wave testing.

GENERAL SEISMIC CONSIDERATIONS

Southern California is located in an active seismic region (CBC Seismic Zone 4) and numerous known and undiscovered earthquake faults are present in the region. Hazards associated with fault rupture and earthquakes include direct affects such as strong ground shaking and ground rupture, as well as secondary affects such as liquefaction, landsliding and lurching. The United States Geological Survey (USGS), California Geologic Survey (CGS), Southern California Earthquake Center (SCEC), private consultants and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and early warning of strong ground shaking. Research and practice have shown that earthquake prediction is not practical or sufficiently accurate to benefit the general public. Also, several recent and damaging earthquakes have occurred on faults that were unknown prior to rupture. Current standards and the California Building Code call for earthquake resistant design of structures as opposed to prediction.

Seismic Hazard Zones

The California State Legislature enacted the Seismic Hazards Mapping Act of 1990, which was prompted by damaging earthquakes in California, and was intended to protect public safety from the effects of strong ground shaking, liquefaction, landslides, and other earthquake-related hazards. The Seismic Hazards Mapping Act requires that the State Geologist delineate various "seismic hazards zones." The maps depicting the zones are released by the California Geological Survey (the CGS).

The Seismic Hazards Mapping Act requires a site investigation by a certified engineering geologist and/or civil engineer with expertise in geotechnical engineering, for projects sited within a hazard zone. The investigation is to include recommendations for a "minimum level of mitigation" that should reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy. The Seismic Hazards Mapping Act does not require mitigation to a level of no ground failure and/or no structural damage.

Seismic Hazard Zone delineations are based on correlation of a combination of factors, including: surface distribution of soil deposits; physical relief; depth to historic high groundwater; shear strength of the soils; and occurrence of past seismic deformation. The subject property is located within the United States Geologic Survey, Azusa Quadrangle. Seismic hazards within the Azusa Quadrangle were evaluated by the CGS in their report, *"Seismic Hazard Zone Report for the Azusa 7.5-minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report* 021." According to the Seismic Hazard Zones Map, the study area is not within an area that has been subject to, or may be subject to liquefaction or earthquake induced ground deformation. The historic high groundwater level is more than 50 feet below the ground surface, which effectively precludes liquefaction.

Ground Motion

Figure 3.3 of SHZR-021 contains ground motion values assigned by the CGS for this area of Irwindale. The Design Basis Earthquake (10% Exceedance in 50 years) for the study area is a peak ground acceleration (PGA) of 0.57g (Figure 3.3, Probabilistic PGA). The de-aggregated predominant earthquake magnitude (M_w) is 7.0 (Figure 3.4, Predominate Earthquake). The major seismic sources relative to the subject property are the Raymond and Sierra Madre fault systems, which are present toward the north along the base of the San Gabriel Mountains, and the Whittier fault, which is present toward the south. These probabilistic ground motions could be expected at the site during the design lifespan of any structures. These ground motions were also adopted to estimate dynamic settlement.

Dynamic Settlement

The potential for dynamic settlement of the fill deposits was estimated using a method developed by Tokimatsu and Seed, 1987 and modified for computer use by Pradel, 1998. In this method, the volumetric strain in the soil is estimated from the cyclic shear strain and the number of shear cycles. In addition to the ground motion input, the two most important parameters for this analysis include the maximum shear modulus of the soil and normalized SPT blow count [$(N_1)_{60}$]. The maximum shear modulus of representative soil layers was estimated using the shear wave velocity profiles determined in the surface wave study and by assuming bulk densities. A bulk density of 135 pcf was assumed for the fill. The $(N_1)_{60}$ values were calculated from the corrected Becker-Hammer blow counts using standard procedures. The calculated settlements are shown in the Dynamic Settlement graph. The amount of dynamic settlement ranges from 2.8 inches near Boring 4 to 3.8 inches near Boring 6.

PRELIMINARY FINDINGS

The preliminary findings of this Phase I exploration are based upon six borings, geophysical studies and research of available records. The study area is underlain by 160 to 185 feet of rubble, earth and silt fill. The lower 30 to 50 feet of the fill was placed prior to the Zeiser-Kling

1991 geotechnical report and is considered "unpermitted fill" with respect to the City of Irwindale, *"Bulletin, Process of Obtaining Approval of Mine Reclamation Backfills."* The older fill appears to consist mostly of the waste byproducts of mining and is described as clays, silts and sand with cobbles. Based upon laboratory testing and analysis, Zeiser Kling concluded that pre-1991 fill was compressible, but upon burial with approved compacted fill and cessation of consolidation, the site would be suitable for development with structures. The City concurred and issued a Conditional Use Permit to fill the pit.

Grading recommendations were presented in the Zeiser Kling report for placing and compacting engineered fill. The report contained specific guidelines for minimum compaction (90 percent of the maximum dry density), moisture content, maximum lift thickness, disposal of oversize materials and frequency of tests. From 1991 to 2005, Zeiser performed inspections and compaction tests of the fill. In addition to compaction tests, test pits were routinely excavated into the fill to estimate the percentage of voids.

The frequency of testing the first few years was high, decreasing through time. Conversely, the rate of filling was low at first, increasing to a maximum rate after 2000. It appears that at the Nu-Way Live Oak landfill, the frequency of testing is inversely proportional to the rate of filling. It is our understanding that the City of Irwindale limited the volume of import earth and debris to 6,000 tons/day. In or around 2003, the volume restriction was increased to 12,000 tons/day.

The thickness of individual fill lifts has been a constant issue. In 1991 through 1993, the thickness of fill lifts appeared to be 2 to 3 feet, maximum. Thicker lifts were failed and reprocessed. By 2000, the fill lifts were thicker than 4 to 5 feet thick. After 2003, the lifts

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appear to be 6 to 8 feet and up 10 feet thick. There is no evidence of failing tests and reworking of thick lifts after 2000.

In the early to mid-1990's there is evidence that oversize material was disposed of in a controlled manner, primarily within the Southern California Edison easement. However, the mapping of elevations and locations of windrows and "rock blankets" are limited. Zeiser indicates that the oversize disposal areas and methods were ultimately acceptable and reasonable. It was understood that the fill within the SCE easement was to be non-structural.

In light of thickness of undocumented fill and silt, and less than optimal professional inspection and jurisdictional oversight, the engineering properties of the landfill are surprisingly good. Downhole shear wave velocity testing and surface wave testing indicate that the average shear wave velocity of the upper 30 meters (V_S30 = 100 feet) of fill ranges from 1,107 to 1,242 fps. From a 2007 California Building Code standpoint, the fill deposit would classify as "Site Class D - Stiff Soil Profile to C - Very dense soil and soft rock." Surface wave testing indicates that the average shear wave velocity of the upper 60 meters (V_S60) of fill ranges from 1,326 to 1,574 fps. The central portion of the site has higher average velocities and no discernable velocity inversion as compared to the north.

According to surface wave testing of natural alluvial deposits by Geovision elsewhere in Irwindale, the fill at the Nu-Way site has shear wave velocities in the lower range of what is found in normally consolidated alluvial fan deposits below the mouth of San Gabriel Canyon. Becker-Hammer test results and calculated SPT blow count correlations also indicate a normally consolidated deposit that is characterized as medium dense to dense.

The shear wave velocity of a material is a function of it's stiffness (shear modulus) and the bulk density. Bulk density is a function of the volume of solids present relative to the total volume. The higher the bulk density the lower the percent voids. One of the goals of this investigation that was not met was to directly measure the bulk density through compensated density logging and to estimate the percent voids visually. Because of the high shear wave velocities, it is apparent that the average bulk density is high. A subsequent phase of testing, which is to include deep test pits and large scale density testing, will provide bulk density information for the upper 50 feet of fill.

The results of Becker-Hammer testing correlate well with the geophysical testing. Unlike other nearby gravel pits that have been filled with inert construction debris, the blow counts reveal a relatively uniform fill that is similar to normally consolidated sand and gravel alluvium. The blow counts and surface wave testing also reveal a less cobbly and lower shear wave velocity zone below the "1991 surface," which is believed to be the older, undocumented fill. The lower velocity zone is discontinuous and pinches out toward the center of the pit (north of Line C and is thin to not present near the north edge of the pit. Although the velocities are lower than the shallower fill, shear wave velocities in the undocumented fill range from 1,082 fps to 1,230 fps, which falls in the high end of CBC "Site Class D - Stiff Soil Profile."

The method of processing and placing the rubble fill, at least through the 1990's, appears to have resulted in a relatively high quality, engineered fill. The most important factors for the result are believed to be the onsite mobile crusher and the early lift thickness control. The crusher was able to reduce oversize concrete and rubble to six inches in diameter and smaller. Also, the crusher was able to separate the rebar from concrete. The resulting fill material is similar to crushed miscellaneous base (CMB) or crushed aggregate base (CAB), which at times, was apparently produced at the site and exported commercially.

The records and physical evidence indicate that the quality of the fill became lower after 2000. The fill lifts appear to have become thicker, there was less professional oversight and testing, more rebar and non-approved materials were reported, and the fill was placed too rapidly. As a result the engineering properties of the fill below 45 to 50 feet appear better than the shallower fill.

Geotechnical Findings

The following geotechnical findings are based upon the exploration performed to date and should be considered preliminary. Additional exploration and peer review will be required to determine the ultimate suitability and/or requirements for developing the site with new buildings. Ultimately, the goal is to verify that the total and differential settlements of any future structures will bw within what is allowed by the Building Code and State and County guidelines for static and dynamic loading.

Our main finding is that the upper 40 to 50 feet of the fill is considered inadequate and should be removed and recompacted. Any new fill should be placed in conformance with the City of Irwindale Building Code, including Appendix J, and the Guidelines for Above-Water Backfilling of Open-Pit Mines. This will reduce the total dynamic settlement potential to a reasonable amount and within State guidelines and provide more uniform support for foundations and slabs. It is possible that the remedial grading could be limited to the building footprints plus the depth of removal outside the footprints. Mat foundations or other types of structural elements could be employed to reduce the depths of remedial grading and/or to mitigate any remaining excessive differential settlement potential.

December 8, 2008
IC 07034-I

IRVINE GEOTECHNICAL Inc

Mnoian Management, Inc.
P.O. Box 661238
Arcadia, California 91066-1238

Attention: Jim Mnoian

Subject

Phase 2 - Geotechnical Engineering Exploration
Evaluation of Nu-Way Live Oak Landfill
Remainder Parcel of Parcel Map As Per Book 186 P 79-82
Approximately 65 Acres
East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue
Irwindale, California

References: Report and Proposal by Irvine Geotechnical, Inc.:

Phase I - Geotechnical Engineering Exploration, Nu-way Live Oak Landfill, East of 605 Freeway, West and South Live Oak Lane, And North of Live Oak Avenue, Irwindale, California, dated March 31, 2008 and

Proposal to Provide Phase II of the Phased Geotechnical Engineering Exploration, Evaluation of NU-WAY Landfill for Proposed Development, dated June 9, 2008

Introduction

This report has been prepared per our agreement and summarizes the findings of Irvine Geotechnical's Phase II geotechnical engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution and engineering properties of the earth materials underlying the upper 50 feet beneath two portions of the site. This study is being performed in conjunction with a phased geotechnical study of the Nuway-Live Oak landfill for obtaining approval of the Mine Reclamation Backfill, developing the site with a commercial/retail development and to determine compliance with the Conditional Use Permit.

PHASE 2 - INVESTIGATION

The Phase 2 investigation plan was identified in our June 9, 2008 proposal, and was based on the results of our Phase 1 exploration and testing. The goal was to excavate two, large test pits, from which density testing, gradations and detailed logging were to be performed. The northerly pit, (Test Pit 1) was chosen to coincide with Boring 5 and the intersection of Shear Wave Velocity Profiles A and B. The westerly pit (Test Pit 2) was chosen to coincide with Boring 3 and the intersection of Shear Wave Velocity Profiles B and C. Test Pits 1 and 2 are shown on the Geologic Map. The corners of the pits were located by Geo-Logic Associates using a Trimble GeoXH 2005 series GPS receiver, which is a mapping grade GPS device that is generally accurate to within 12 inches.

Both pits were planned to extend to 50 feet below the ground surface, and were then ultimately deepened to 70 to 75 feet. The initial footprint of the pits was based upon a 50 foot high 1:1 slope on 3 sides of the pit and an entry ramp to four benches. Four level benches (benches 1 - 4) were created in both pits at approximately 12 foot vertical increments. The approximate corners of the benches and elevations were determined by Geo-Logic using the GPS mapping device and plotted onto the Geologic Map. The benches and elevations are shown on the Test Pits Map appended to this report.

Excavation of the trenches began on August 4, 2008 using excavators, dozers and loaders. Fill soils removed from the pits were segregated by depth and stockpiled outside of the pits. The stockpiles were later used as source material to create Maximum Achievable Density (MAD) test pads. Excavation and testing of the test pits and 8 benches had been completed by September 3, 2008.

The deepest portions of the pits were deepened an additional 25 ± feet between October 21 through 27, 2008 with an excavator. The deeper holes were not considered safe to enter to perform physical testing. The deeper portions of the test pits were logged, photographed and video taped by the engineering geologist and soils engineer. Profiles through the test pits are shown on Section A and the shear wave Velocity Profile plates.

Excavation of the trenches and stockpiling were performed under the observation of the project engineer and geologist, who also photo-documented and videotaped the process. Periodic observations of the excavation process were also performed by personnel of Hushmand and Associates and the City of Irwindale. Hushmand is also in the process of performing independent bulk density and gradation testing of the eight benches.

Bulk Density Tests

One Bulk Density (BD) test was performed on each of the benches, with the approximate locations shown on the Test Pits plate and the Geologic Map. By design, no testing was performed of clean, compacted fill cap that overlies the rubble fill. The tests were performed in conformance with ASTM 5030-04 and the ITAC guidelines. A large, ¾-inch thick steel plate with a 6-foot diameter hole in the center, was used as a template. It was decided to level the template on the bench in lieu of providing a raised lip because of cost and timing constraints. The bulk density test holes were excavated to depths of 4 to 5 feet using a small backhoe and hand labor. Material excavated from the BD tests was transferred to a roll-off bin via a loader.

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Care was taken to minimize the loss of soil via spillage by using tarps and being careful. Re-bar and other non-soil and concrete debris protruding into the BD test holes were cut flush with the sides of the hole. The holes were excavated as close to vertical as possible and cleaned by hand. The bottom of the excavation was also cleaned to a smooth surface by hand. Upon completion, the BD test hole was photographed, measured and logged by the project engineering geologist.

Earth materials from the BD density test holes were stored in the roll-off bins and covered with tarps to prevent moisture and fines loss due to evaporation and wind. The moisture content of the in-situ soils removed from the BD test was also measured and recorded. The bins were carried by truck to the Nu-Way recycling center in Monrovia for weighing and then transferred back to the site for Bulk Gradation testing.

The volumes of the BD tests were determined using "water-replacement" by accurately measuring the volume of water required to exactly fill the BD test pit to the bottom edge of the template. An inline water meter (Blue-White Industries RT-200MI-GPM3) connected to a water truck via a fire hose was used to fill the BD test pits. The meter is accurate with flow rates between 10 and 100 gpm and was calibrated on August 1, 2008. PVC pipe transitions were placed on either side if the in-line valve to ensure laminar flow past the venturi.

Prior to filling the pits with water, two layers of visqueen were placed to form a water-tight container. Two layers were considered necessary due to the sharp concrete, glass and re-bar debris exposed in the sides and bottoms of the BD pits. From within the hole, the engineer verified that the plastic liner was in firm contact with the underlying soils and not stretched across voids.

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Upon filling the hole and metering of the volume, the liners were perforated and removed. The time required for the holes to drain were recorded by the staff engineer and project geologist. The measured volume was also compared to the mathematical volume based upon the actual dimensions of the BD test pits.

The moist bulk density is the ratio of the weight removed from the pit to the volume of the pit. The dry bulk density is the corrected weight after subtracting the water content from the fraction of the mass finer than ¾ inch. Material coarser than ¾ inch consists of steel, brick and concrete fragments, which generally do not contain appreciable moisture. The moisture content relative to the dry density was plotted on the Moisture-Density Relationship chart. A clustering of dry densities near 132.5 pcf with a moisture content of 11 percent for the MAD pads is believed to represent the maximum dry density.

Bulk Gradations

All of the material from the BD tests were sieved to determine the distribution of material sizes. A rack containing screen sizes 12x12, 8x8, 6x3, 3x3, and 1x1 inch grids was manufactured by the client and placed near the entrance to the Nu-Way pit. Materials collected on the screens were sorted in bins and weighed. The sorting and weighing were performed in a paved portion of the site, which facilitated weighing of bins and large samples and in controlling spillage. A representative sample of the materials passing the 1 inch sieve was transferred to the soils laboratory to determine the additional fractions through the sand-size range and the percentage of fines (percent passing the #200 sieve). The gradations and weighing were performed by the project geologist and staff engineer of Irvine Geotechnical. The results of the gradation testing are shown on the Grain Size Distribution graphs.

The following table summarizes the results of the bulk density testing. Refer to the Geologic Map and Test Pit Map plate for the locations of the benches, depths and individual tests.

SUMMARY OF BULK DENSITY TESTING						
SAMPLE	TYPE	WET DENSITY (PCF)	DRY DENSITY (PCF)	VOID RATIO (%)	MAX. DENSITY (PCF)	RELATIVE COMPACTION (%)
TP1 - Bench 1	Bulk Density - In situ	79.7	57.0	1.100	133.0	42.9
TP1 - Bench 2	Bulk Density - In situ	133.2	125.6	0.280	133.0	94.4
TP1 - Bench 2	Bulk Density - Test Pad	142.0	131.5	0.179	133.0	98.9
TP1 - Bench 3	Bulk Density - In situ	114.9	109.6	0.473	133.0	82.4
TP1 - Bench 3	Bulk Density - Test Pad	142.6	132.4	0.249	133.0	99.5
TP1 - Bench 4	Bulk Density - In situ	91.0	86.1	1.100	133.0	64.7
TP1 - Bench 4	Bulk Density - Test Pad	142.1	132.5	0.181	133.0	99.6
TP2 - Bench 1	Bulk Density - In situ	127.8	121.1	0.334	133.0	91.1
TP2 - Bench 2	Bulk Density - In situ	133.6	126.4	0.260	133.0	95.0
TP2 - Bench 2	Bulk Density - Test Pad	139.6	129.7	0.211	133.0	97.5
TP2 - Bench 3	Bulk Density - In situ	129.2	120.2	0.319	133.0	90.4
TP2 - Bench 3	Bulk Density - Test Pad	137.1	127.3	0.235	133.0	95.7
TP2 - Bench 4	Bulk Density - In situ	126.5	117.3	0.329	133.0	88.2
TP2 - Bench 4	Bulk Density - Test Pad	136.1	126.5	0.239	133.0	95.1
TP2 - Bench 4	Bulk Density - Test Pad2	135.6	127.3	0.250	133.0	95.7

Sand Cone Density Tests

A sand cone conforming to ASTM 1556 was used to determine the in-situ moisture and density of the soil exposed at the surface elevation of each of the benches. Bulk samples of the soils at the locations of the soil samples were also obtained and transferred to the soils laboratory for maximum density testing (ASTM 1557). The sand cone tests were performed by the soils technician.

The results of the sand cone testing are shown on the following table. Refer to the Test Pits plate for the locations of the individual tests.

SUMMARY OF SAND CONE DENSITY TESTING						
SAMPLE	TYPE	WET DENSITY (PCF)	DRY DENSITY (PCF)	VOID RATIO (%)	MAX. DENSITY (PCF)	RELATIVE COMPACTION (%)
TP1 - Bench 1	Sand Cone - In situ	119.4	116.1	---	123.5	94.0
TP1 - Bench 2	Sand Cone - In situ	105.9	96.7	---	122.5	78.9
TP1 - Bench 3	Sand Cone - In situ	112.3	102.4	---	115.5	88.7
TP1 - Bench 4	Sand Cone - In situ	98.9	90.6	---	125.0	72.5
TP2 - Bench 1	Sand Cone - In situ	116.4	107.5	---	129.5	83.0
TP2 - Bench 2	Sand Cone - In situ	107.2	98.1	---	130.5	75.2
TP2 - Bench 3	Sand Cone - In situ	109.9	97.4	---	132.0	73.8
TP2 - Bench 4	Sand Cone - In situ	124.8	107.5	---	131.5	81.7

Visual Observations

Visual observations and mapping performed of the test pit walls, bulk density tests and stockpiles reveal that landfill deposit is highly variable. Significant (approximately 15 to 20 percent of the landfill deposit) oversize (larger than 12 inch) fragments are present within the fill. Of the oversize fraction, steel-reinforced concrete fragments, foundation elements, columns and other construction demolition debris are present with dimensions that range from 24 inches to more than 20 feet. One reinforced beam exposed in Test Pit 1 near bench 3 was 6 feet wide, 3 feet deep and more than 40 feet long (the ends were not exposed as the beam is longer than the width of the test pit). For many of the fragments, numerous and large reinforcing bars protrude from the concrete. In addition to concrete, drywall, roofing materials, wood, glass, steel beams, asphalt, water-filtration cake, a gas pump, a tire and other debris were observed and photo-documented. The amount, distribution and content of debris within the fill appears to be similar between the two pits.

Fill lifts are clearly visible in the walls of the test pits. In general, the tops of the rubble lifts are identified by a level to gently sloping soil caps. Lifts thicknesses are visible that vary from a few feet thick to more than 8 feet. The lifts also reveal little processing and spreading. Individual piles of debris that were apparently "end-dumped" in the landfill are surrounded by other end-dumped piles and in turn buried by additional lifts. Nesting and voids were observed and common. Nesting and bridging were primarily observed adjacent to very large oversize fragments and where oversize fragments were concentrated. Nesting and bridging also occurred where layers of drywall covered concrete fragments.

Discussion

It is clear from visual observations of Test Pits 1 and 2 that the fill exposed in the upper 50 or more feet was not processed and the deposits were essentially buried in piles dumped from trucks. Intuitively, the results of bulk density testing were expected to yield relative compaction percentages well below 90. However, only BD Test Pit 1, Bench 4 yielded a relative compaction result that was significantly less than 90 percent. Test Pit 2, Bench 4 yielded a relative compaction of 88 percent. All of the sand cone tests, except for Test Pit 1, bench 1, yielded relative compaction results less than 90 percent relative compaction. Observation of the BD test pits after the liners were removed showed water percolation removed the matrix soils to expose a clast-supported fill. Because the concrete percentage is so high and it is well graded, it is believed that only loose clean fill mixed into the voids is sufficient to raise the "relative compaction" to near 90 percent. The sand cone testing and water infiltration testing indicates the interstitial soils are poorly compacted. The general, clast-supported nature of the fill deposit is supported by the relatively high shear and compression wave velocities measured in our Phase 1 investigation.

Void ratio, which is represented by the ratio of Volume of Solids to Total Volume, may be a better method to quantify the quality of fill placement and compaction. For the in-situ tests, void ratios ranged from 0.260 to 1.10, with an average of 0.398. For the MAD tests, void ratios ranged from 0.179 to 0.250, with an average of 0.221. The difference between the in-situ void ratio and the minimum void ratio should also correlate to a "maximum static and dynamic consolidation potential."

Shear Wave Velocity Profiles

Since the test trenches coincide with the shear wave velocity sections, attempts were made to correlate shear wave velocity with observed materials. Shear wave Velocity Profiles Line A, Line B, and Line C were redrawn and re-contoured in equal vertical and horizontal scales. This was to make direct comparisons with geologic cross sections and test trenches possible. The following information was also plotted onto the Velocity Profiles: borings by both Irvine and Zeiser-Kling; the 1991 ground surface (base of Nu-Way rubble fill); contact between alluvium and fill; and Test Pits 1 and 2. The shear wave Velocity Profiles were also re-contoured after normalizing the measured shear wave velocities to overburden stress (Robertson, et. al. 1992, *"Seismic cone penetration test for evaluating liquefaction potential under seismic loading"* and Andrus and Stokoe, 2000, *"Liquefaction resistance of soils from shear wave velocity*).

The test pit locations were chosen to encounter what was believed to be the "worst" fill (lowest shear wave velocities and Becker-Hammer blow counts. For the fill exposed in Test Pit 1, the shear wave velocity (V_s) of the upper 50 feet is generally less than 1,200 fps. The portion of the pit below 50 feet encountered normalized shear wave velocities (V_{s1}) of 900 to 1,050 fps. The southerly, shallow side of Test Pit 1 was not deep enough to encounter a relatively low velocity zone V_s less than 855 fps. For Test Pit 2, the V_s were slightly higher with a range of 1,000 to 1,350 fps for the upper 50 feet. V_{s1} of Test Pit 2 below 50 feet ranges from 1,000 to 1,500 fps.

Normalization of the shear wave velocity greatly enhances the velocity contrasts between the older fill (pre-1991 and likely mining waste silt pond) with both the native alluvium and the rubble fill. For Velocity Profiles A and B, the waste silt has V_{s1} between 600 to 855 fps. The relatively low V_s values in the pre-1991 waste silt correlate with silt and clay reported by Zeiser-Kling and lack of concrete. Normalized V_{s1} in the alluvium ranges from 1,000 fps to more than 1,200 fps. Velocity Profile C indicates relatively uniform V_s and V_{s1} for the fill and alluvium.

CONCLUSIONS

The following conclusions and findings are based upon our Phase 2 exploration program and testing. Fill exposed in the Test Pits is of variable composition, density and quality. However, it is clear that the upper 50 to 75 feet of the fill was not processed and placed in an engineered and controlled manner. As such, the fill is not considered suitable for support of engineered structures. The test pit locations were chosen to find what was believed to be the "worst" fill. It is possible, that other locations of the Nu-Way site are underlain by processed and controlled fill that is shallower than 50 feet. However, based upon review of filling/grading related documents and testing performed to date, it is considered unlikely that the upper 65 to 75 feet of fill placed anywhere on the site was done in an engineered manner.

At a depth of approximately of 65 to 70 feet (elevations 335 to 340 feet) the test pits reveal that the fill lifts appear to have become thicker and more re-bar and non-approved materials are present. This could correlate with less professional oversight and testing and/or placement of fill too rapidly for the type of equipment and amount of processing.

Shear wave velocity data and Becker-Hammer boring data indicate better quality fill below 65 to 75 feet. Therefore, a mitigation plan that includes removing, processing and recompacting the upper 65 to 75 feet of fill materials is indicated. Additional subsurface exploration and shear wave profiling will be required to verify the depth to processed and engineered fill. Additional testing will also be required to quantify the engineering properties and limits of the low shear wave velocity silt pond materials (old fills). It is possible that in addition to, or in combination with remedial grading, deep dynamic compaction will be required to densify materials below the removal depths.

The fill observed in the test pits certainly does not conform to the 1994 CUP, which along with a October 15, 1990 Zeiser-Kling report, was intended to control how the Nu-Way Live Oak pit was backfilled. The fill quality and placement procedures also do not conform with published standards that predate fill placement. Procedures and quality control measures to place rubble (concrete and other inert debris mixed with soil) fills are based upon methods standardized by the US Military for "rock" fills (*Compaction Requirements and Procedures*, Foundations and Earth Structures, NAVFAC DM-7.2, May 1982 and (*Engineering Design of Test Quarries and Test Fills*, Corps of Engineers Engineer Manual, 1110-2-2301, September 30, 1994.

The following items are not in conformance: maximum size of fill materials; thickness of fill lifts; processing; frequency of compaction testing; type of compaction testing; degree of relative compaction; no use of approved windrows and inadequate documentation and reporting.

The maximum fill material size was restricted to 12 inches. Oversize (greater than 12 inches) materials could be placed in approved windrows. The test pits indicate that 15 to 20 percent of the fill consists of material larger than 12 inches that are not located in windrows.

The test pits indicate little to no processing of the fill. Fill appears to have been dumped from trucks and buried. All fill placement standards prior to and after the CUP call for separating oversize materials, mixing and spreading the fill materials and moisture conditioning. Lack of processing has resulted in "nesting."

Lifts are too thick. Zeiser-Kling recommended placing the fill in 6 to 12 inch lifts. The above references limit the maximum fill lift thickness to 1 to 2 times the maximum particle diameter, which for a 12 inch maximum, would be 24 inches. Lifts up 8 feet thick were observed in the Test Pits.

Since visual observations of the fill are as or more important than the results of testing, some exploration method will have to be implemented that can obtain videos or pictures of the deep fill. The feasibility of large diameter borings and video logging will be investigated. Also, methods to determine the in-situ bulk density and/or void ratio of the deep fill will be explored and presented in a subsequent Phase 3 exploration program.

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Irvine Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
Irvine Geotechnical, Inc.

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R:\ICprojects\2007Projects\IC07034 Mnoian NuWay\IC07034 MnoianPhase 2 Report Corrected 8-25-2010.wpd

Enc: Moisture-Density Relationship
Grain Size Distribution (14 plates)
Test Pit Map
Velocity Profiles (6)
In pocket: Geologic Map and Sections A, B and C

xc: (7) Addressee

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April 27, 2010
IC 07034-I

IRVINE GEOTECHNICAL Inc

Mnoian Management, Inc.
P.O. Box 661238
Arcadia, California 91066-1238

Attention: Jim Mnoian

Subject

Phase III - Geotechnical Engineering Exploration
Evaluation of Nu-Way Live Oak Landfill
Remainder Parcel of Parcel Map As Per Book 186 P 79-82
Approximately 65 Acres
East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue
Irwindale, California

References: Reports by Irvine Geotechnical, Inc.:

Phase I - Geotechnical Engineering Exploration, Nu-way Live Oak Landfill, East of 605 Freeway, West and South Live Oak Lane, And North of Live Oak Avenue, Irwindale, California, dated March 31, 2008 and

Phase II - Geotechnical Engineering Exploration, Evaluation of Nu-Way Live Oak Landfill, Remainder Parcel of Parcel Map As Per Book 186 P 79-82, Approximately 65 Acres, East of the 605 Freeway and Between Arrow Highway and Live Oak Avenue, Irwindale, California, dated December 8, 2008

Introduction

This report has been prepared per our agreement and summarizes the findings of Irvine Geotechnical's Phase III geotechnical engineering exploration performed on a portion of the site. The purpose of this study was to evaluate the deeper fill within Test Pit 2. This study is being performed in conjunction with a phased geotechnical study of the Nuway-Live Oak landfill for obtaining approval of the Mine Reclamation Backfill, developing the site with a commercial/retail development and to determine compliance with the Conditional Use Permit.

PHASE III - INVESTIGATION

Our Phase II investigation included excavating two test pits (Test Pit 1 and Test Pit 2) initially to depths of 50 feet and then ultimately to depths of 70 to 75 feet. The pits were excavated with conventional grading equipment to create pads, from which bulk density and gradation tests of the fill were taken. The sides of the test pits were laid back to a relatively stable 1:1 gradient for safety. The test pits could not be safely deepened without significant grading to make the footprint of the pits larger. The test pit and bench locations were surveyed to determine locations within the pit and elevations.

The purpose of the Phase III investigation was to drill a large diameter boring from within Test Pit 2 (westerly of the two pits). The boring is situated along the downhill side of Bench 3 and was drilled from an elevation of approximately 370 feet.

Between September 1 and 10, 2009 a three -foot diameter boring was drilled to a depth of 70 feet below the drill pad (elevation 300). Because of abundant rebar, large concrete fragments and caving, drilling was difficult. The geologist was onsite during the drilling to log the drilling spoils as they were removed from the boring. Depths were determined using a weighted tape measure. The completed boring was deemed too dangerous to for a man to downhole log. The boring was then video-logged on September 10, 2008.

A description of the earth materials encountered in the boring is contained on the Log of Boring. The location of the boring is shown on the Boring Map.

CONCLUSIONS

The following conclusions and findings are based upon our Phase III exploration performed from within Test Pit 2. Fill exposed in Test Pit 2 and Boring 1 is of variable composition, density and quality. However, it is clear that the fill placed to an elevation of approximately 313 to 310 feet (92 to 95 feet from the ground surface) was not processed and placed in an engineered and controlled manner. Oversize fragments, debris and rebar have resulted in nesting and significant voids. Drilling shallower than 52 feet (measured from the drill bench) was very difficult to drill due to caving, oversize concrete fragments and rebar. The fill below 52 feet was uniform, mostly devoid of oversize rubble fragments and easy to drill.

The fill shallower than elevation 310 to 313 feet is not suitably compacted and processed and is not considered suitable for support of engineered structures. Below elevation 310 to 313 feet, the fill has visible horizontal lifts, evidence of processing and mixing and is compact with little voids.

Elevation 310 coincides with the Nuway Live Oak Landfill having an operational year surface of approximately 1998/1999. Based upon field notices and compaction tests prepared by Zeiser-Kling, inspections and compaction tests were relatively frequent from 1991 to 1998/1999. After 1999 and up until 2005, geotechnical inspections and compaction testing were infrequent. This relationship is shown on the Nu-Way Live Oak fill placement and compaction test frequency charts.

The results of Phase III exploration indicates that large diameter borings can successfully be drilled through the rubble fill. Additional borings will be required to verify that the fill quality is consistently better for fill placed prior to 1998/1999 and below elevation 310 to 313 feet (depth of 92 to 95 feet below ground surface). Methods to determine the in-situ bulk density and/or void ratio of the deep fill will be explored and presented in a subsequent exploration program.

Irvine Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
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Enc: Boring Log (4 sheets)
Boring Map
Compaction Testing Frequency Charts (3)

xc: (7) Addressee

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IRVINE GEOTECHNICAL Inc

LOG OF BORING

PROJECT IC 07034 Mnoian
DRILL DATE 9/1/2009 - 9/10/2009
LOG DATE 9/1/2009 - 9/10/2009
LOGGED BY MH/JAI
DRILL TYPE Auger
DIAMETER 36 inch

SURFACE ELEVATION 370 feet
DRILLING CONTRACTOR Malcomb Drilling
SURFACE CONDITIONS On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
370	0	**RUBBLE FILL:** Rubble Debris and concrete fragments with a Silty Sand matrix
369	1	
368	2	
367	3	concrete slab fragments, 4 to 6 inches thick and 12 to 36 inches long, reabar protruding into hole, Silty Sand Matrix
366	4	
365	5	abundant rebar, voids and nesting of debris
364	6	
363	7	
362	8	
361	9	
360	10	rebar and trash debris
359	11	
358	12	
357	13	
356	14	wood and metal debris, voids
355	15	
354	16	
353	17	
352	18	
351	19	rubble and concrete fragments, nesting, open voids 6 to 12 inches, caving
350	20	

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LOG OF BORING

PROJECT IC 07034 Mnoian
DRILL DATE 9/1/2009 - 9/10/2009
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LOGGED BY MH/JAI
DRILL TYPE Auger
DIAMETER 36 inch

SURFACE ELEVATION 370 feet
DRILLING CONTRACTOR Malcomb Drilling
SURFACE CONDITIONS On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
350	20	concrete fragments with rebar, nesting, open voids
349	21	metal and trash debris and rebar, concrete fragments 18 to 30 inches in dimension,
348	22	
347	23	
346	24	steel pipe, slab fragments, voids and nesting of debris, rubble and concrete fragments greater than 36 inches in dimension, caving
345	25	
344	26	
343	27	
342	28	bricks, wire, rubble, Silty Sand matrix
341	29	
340	30	rubble and concrete fragments larger than 24 inches, nesting, voids
339	31	
338	32	
337	33	
336	34	conduit with wires
335	35	
334	36	rebar and concrete fragments larger than 48 inches, nesting, voids, caving
333	37	
332	38	Water seeping into boring, water is flowing within a rubble layer and is perched ontop of a Clayey layer, heavy seep on 9/3 becoming a trickle on 9/10 ▽
331	39	
330	40	Sandy Clay, dark brown

IRVINE GEOTECHNICAL Inc

LOG OF BORING

PROJECT IC 07034 Mnoian
DRILL DATE 9/1/2009 - 9/10/2009
LOG DATE 9/1/2009 - 9/10/2009
LOGGED BY MH/JAI
DRILL TYPE Auger
DIAMETER 36 inch

SURFACE ELEVATION 370 feet
DRILLING CONTRACTOR Malcomb Drilling
SURFACE CONDITIONS On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
330	40	concrete fragments with rebar, nesting, open voids
329	41	metal and wood debris and rebar, concrete fragments 18 to greater than 24 inches in dimension, caving, nesting, voids
328	42	
327	43	
326	44	
325	45	
324	46	Clayey Sand matrix, compact, tight, shearing and breaking of rubble fragments
323	47	
322	48	concrete rubble with no soil in matrix, fragments larger than 12 inches, caving, voids
321	49	
320	50	steel pipe, abundant rebar, caving, nesting, abundant voids
319	51	Slab longer than 3 feet,
318	52	
317	53	
316	54	Concrete rubble fragments in Silty Sand and Clayey Sand matrix, well graded, drastic reduction in rebar and oversize material, dark grey brown
315	55	A/C fragments, rubble lup to 12 inches, weak horizontal layering, no voids,
314	56	
313	57	Horizontal layering, tight, well graded
312	58	
311	59	
310	60	

LOG OF BORING

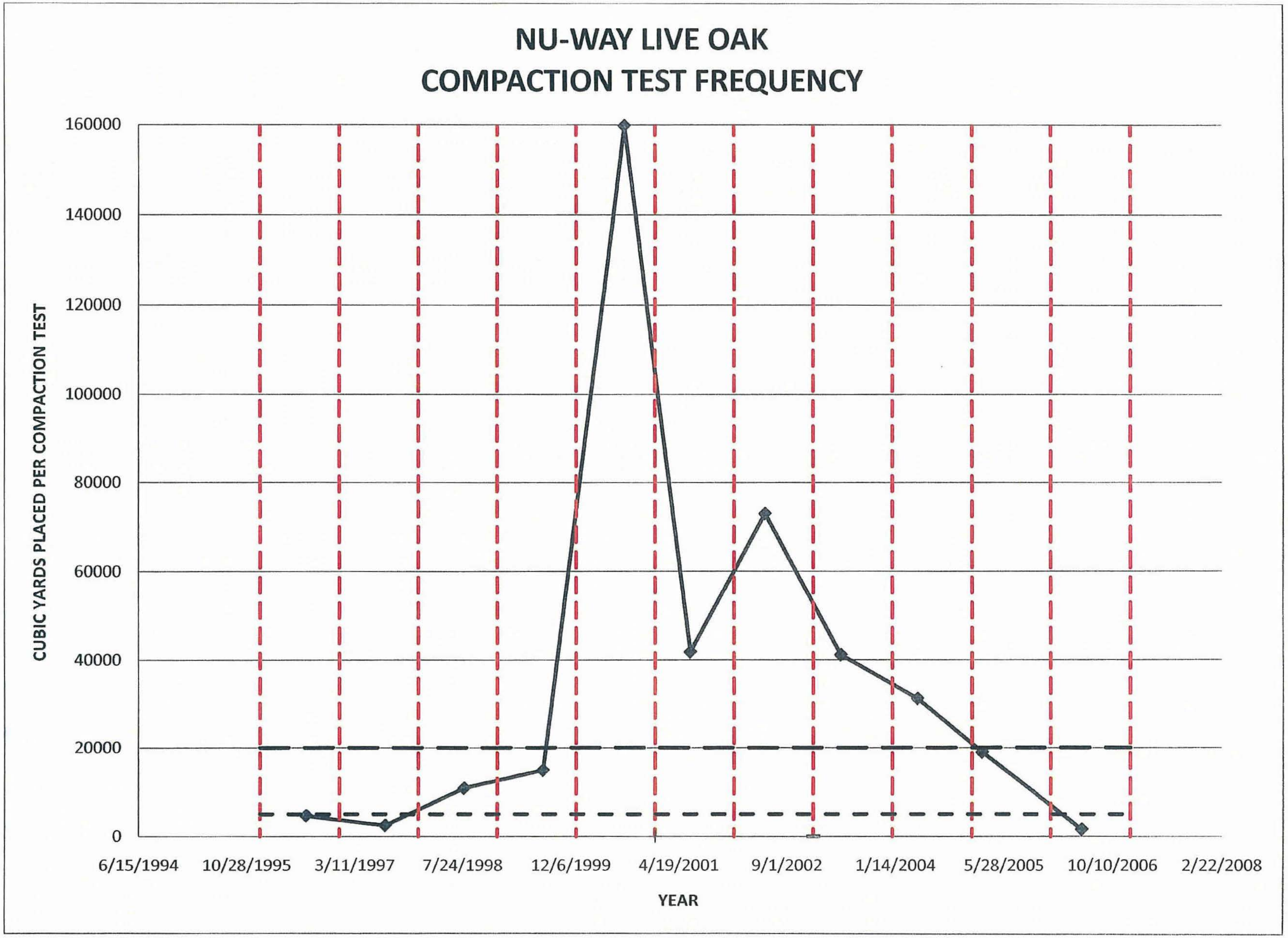
IRVINE GEOTECHNICAL Inc

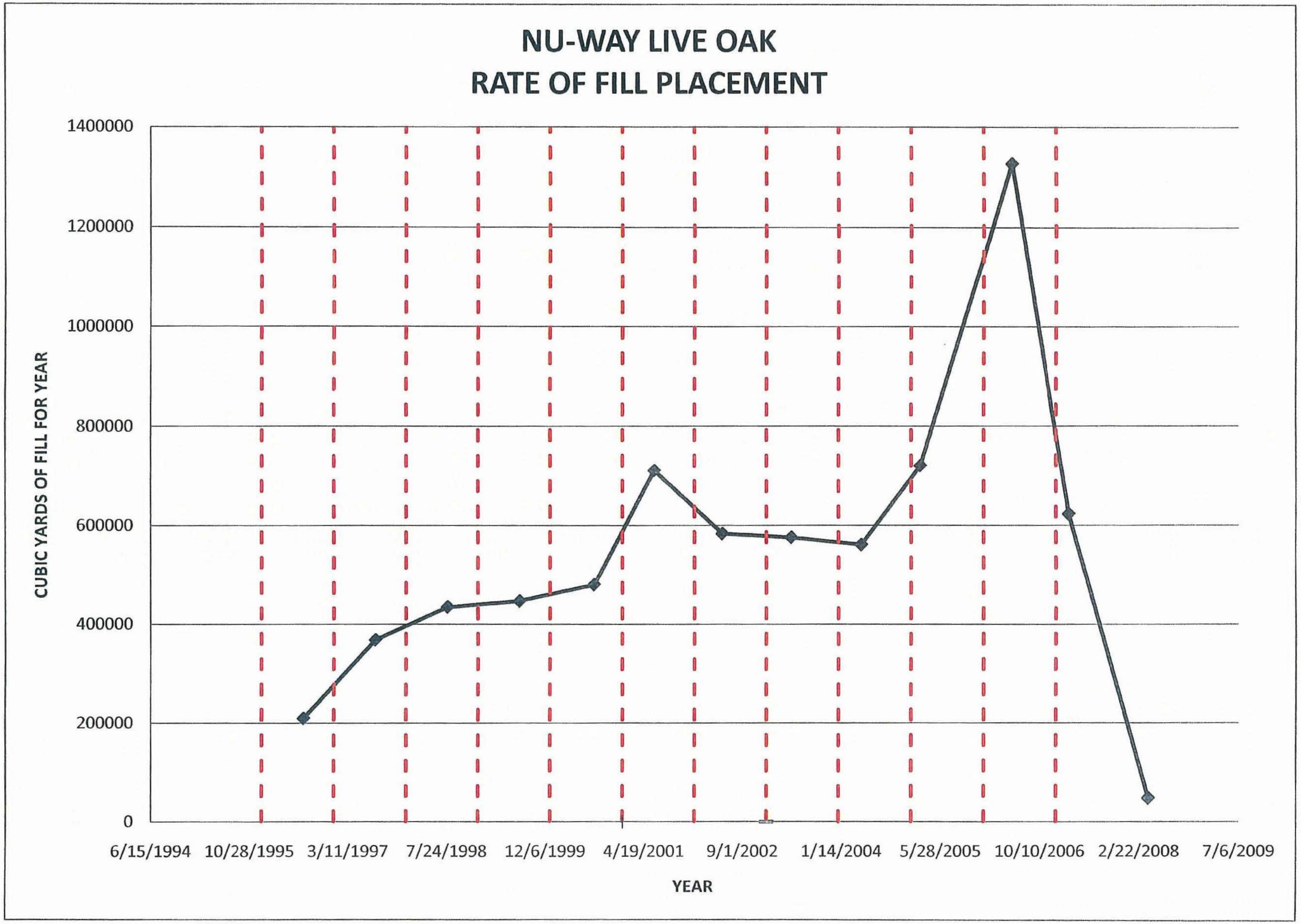
PROJECT IC 07034 Mnoian
DRILL DATE 9/1/2009 - 9/10/2009
LOG DATE 9/1/2009 - 9/10/2009
LOGGED BY MH/JAI
DRILL TYPE Auger
DIAMETER 36 inch

SURFACE ELEVATION 370 feet
DRILLING CONTRACTOR Malcomb Drilling
SURFACE CONDITIONS On Bench 3 within Test Pit 2

BORING 1

Elevation (feet)	Depth (feet)	LITHOLOGIC DESCRIPTION
310	60	Silty Sand and Clayey Sand matrix, dark grey, moist, tight, weak horizontal layering, well graded
309	61	
308	62	wire mesh, silty sand with cobbles,
307	63	
306	64	asphalt fragments, mixed with rubble, silty sand and clayey sand matrix, horizontal layering visible, well graded
305	65	
304	66	
303	67	
302	68	
301	69	
300	70	**END Boring at 7.0 feet**


NU-WAY LIVE OAK
COMPACTION TEST FREQUENCY
CUBIC YARDS PLACED PER COMPACTION TEST
160000
140000
120000
100000
80000
60000
40000
20000
0
6/15/1994
10/28/1995
3/11/1997
7/24/1998
12/6/1999
4/19/2001
9/1/2002
1/14/2004
5/28/2005
10/10/2006
2/22/2008
YEAR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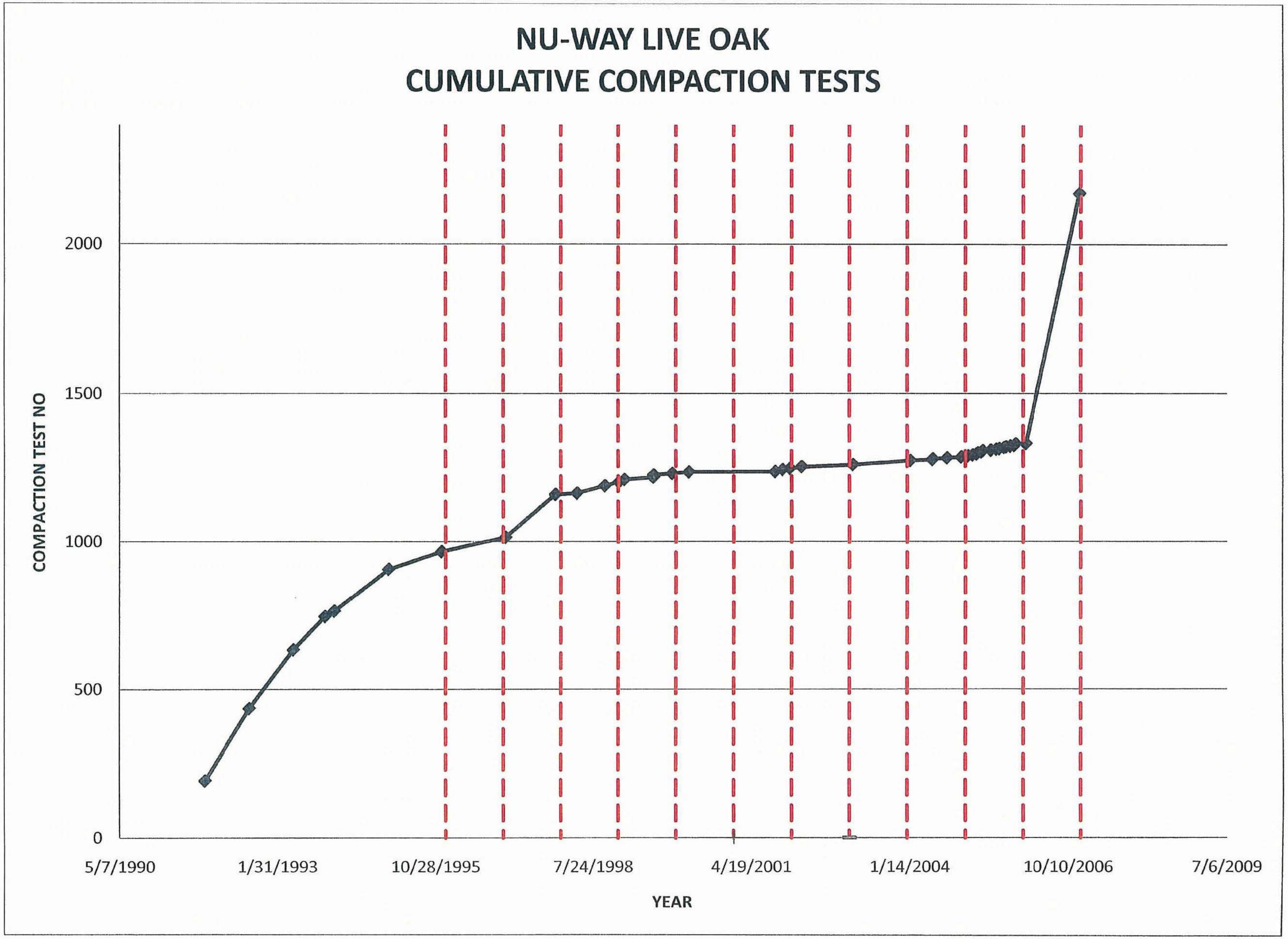
RATE OF FILL PLACEMENT
1400000
1200000
1000000
800000
600000
400000
200000
0
CUBIC YARDS OF FILL FOR YEAR
6/15/1994
10/28/1995
3/11/1997
7/24/1998
12/6/1999
4/19/2001
9/1/2002
1/14/2004
5/28/2005
10/10/2006
2/22/2008
7/6/2009
YEAR

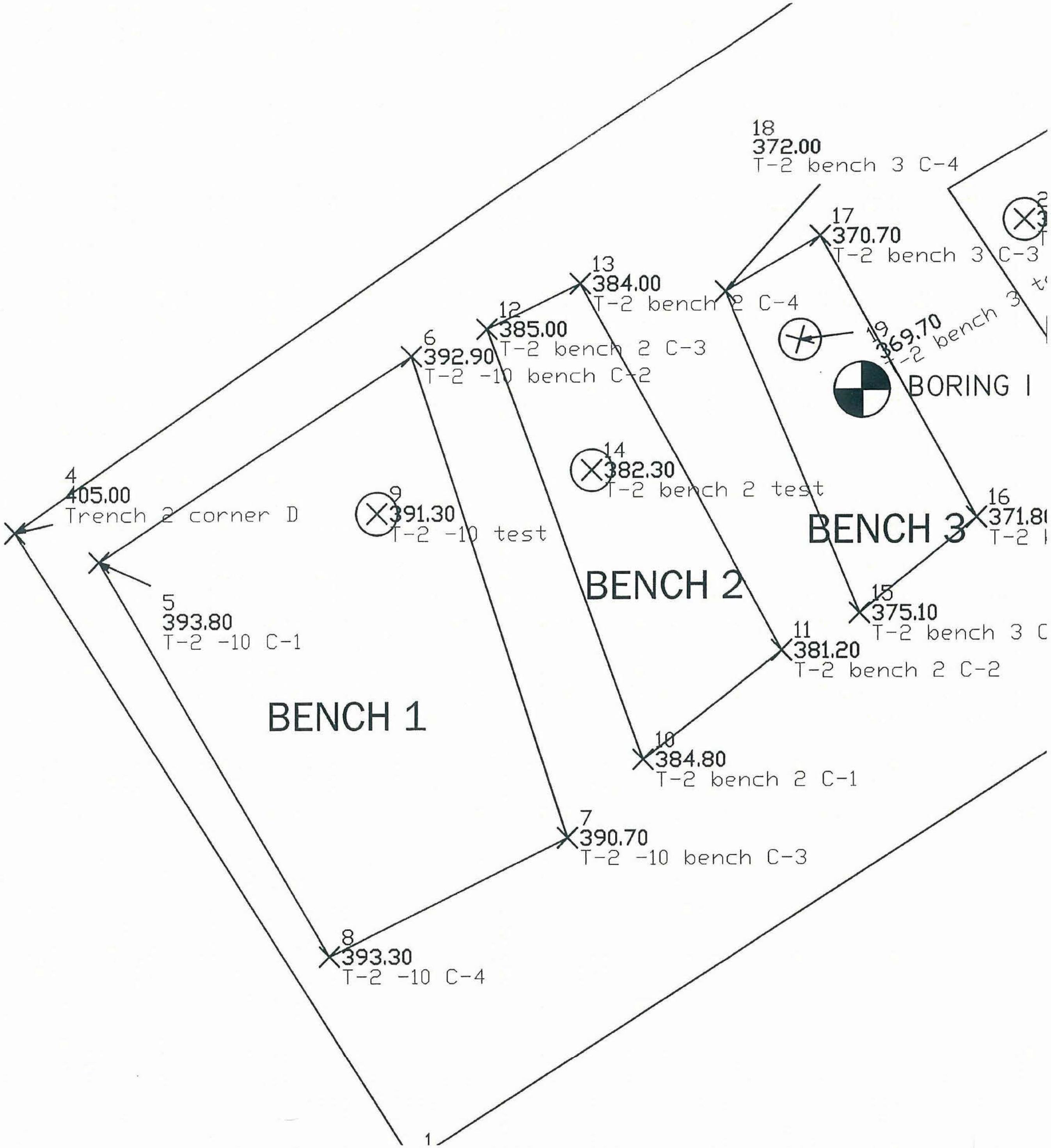
0916.0445



NU-WAY LIVE OAK
CUMULATIVE COMPACTION TESTS
COMPACTION TEST NO
0
500
1000
1500
2000
5/7/1990
1/31/1993
10/28/1995
7/24/1998
4/19/2001
1/14/2004
10/10/2006
7/6/2009
YEAR

0916.0446

0916.0447


18
372.00
T-2 bench 3 C-4
17
370.70
T-2 bench 3 C-3
13
384.00
T-2 bench 2 C-4
12
385.00
T-2 bench 2 C-3
19
369.70
T-2 bench 3
6
392.90
T-2 -10 bench C-2
BORING I
14
382.30
T-2 bench 2 test
4
405.00
Trench 2 corner D
9
391.30
T-2 -10 test
16
BENCH 3
5
393.80
T-2 -10 C-1
BENCH 2
15
375.10
11
381.20
T-2 bench 2 C-2
BENCH 1
10
384.80
T-2 bench 2 C-1
7
390.70
T-2 -10 bench C-3
8
393.30
T-2 -10 C-4
1

PCL XL error

Subsystem: GE_VECTOR

Error: GEEmptyClipPath Warning: IllegalMediaSize

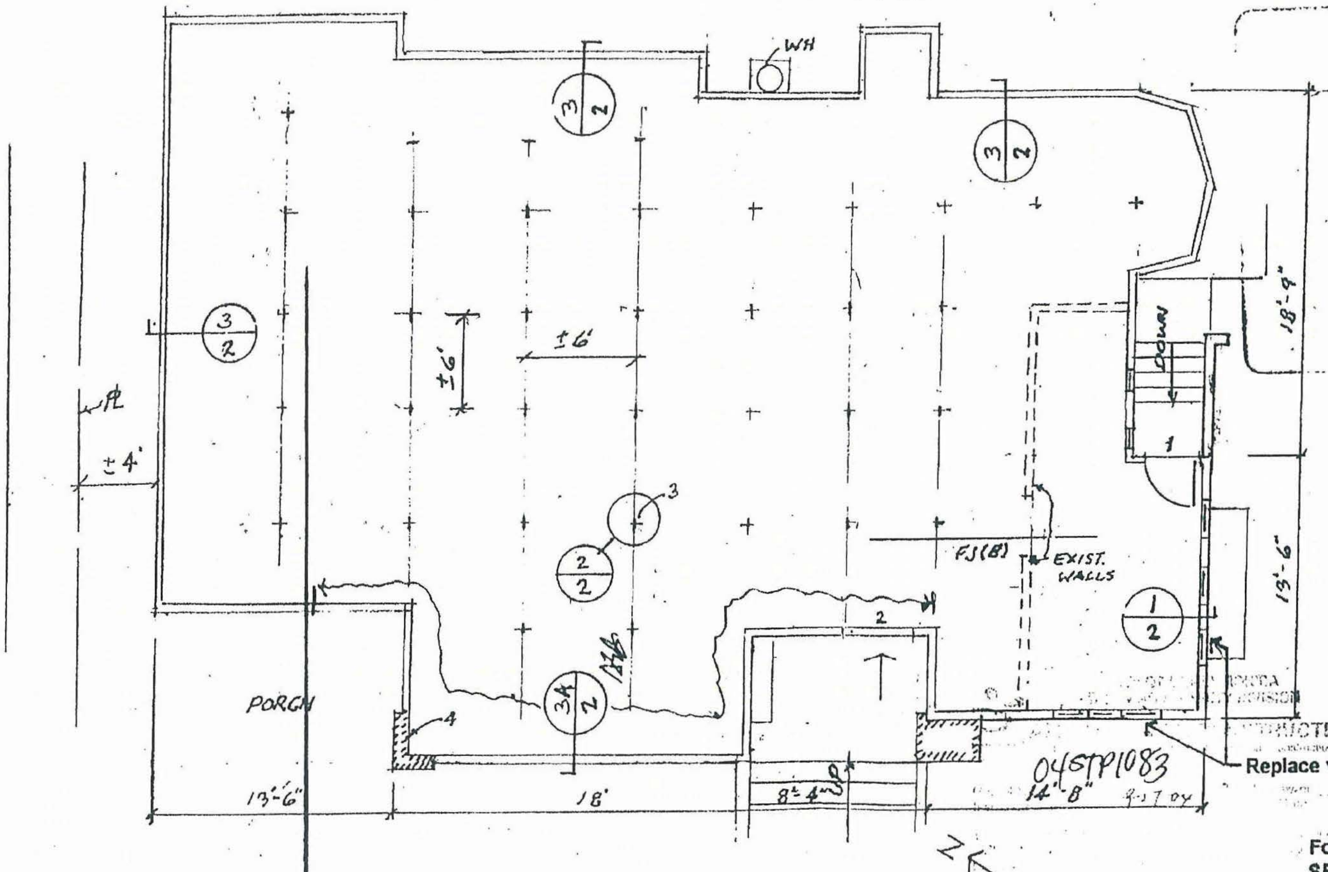
0916.0449

BUILDING ANALYSIS

BUILDING CODE USED: 2002 CBC
OCCUPANCY GROUP: R
TYPE OF CONSTRUCTION: V
STORIES: 1

Scope of work- bolt down house to existing foundation where 6" min. Stem wall and min. 12"X 12" footing exists. Where stem wall and footing are deficient, provide new as shown in detail 1/2 and 3/2

CITY OF SANTA MONICA
City Planning Division
8-16-04
Any changes to these plans submitted to City Planning ... such changes may require ... approvals. Additional ... may be applicable prior to issuance of a Certificate of Occupancy
PLANS
APPROVED



1 Exist. entrance to basement

2 Front entrance above
3 Existing piers
4 Repair exist brick exterior
Remove & Replace

REGISTERED PROFESSIONAL ENGINEER
STANLEY E. SCHOLL
No. 14527
Exp 3-31-05
CIVIL
STATE OF CALIFORNIA

Replace windows

Foundation Bolting/strengthening
SF Residence
2511 Beverly Street
Santa Monica

BS PLANCHECK APPROVED
1/3
8-16-04

Stanley E. Scholl, P.E.
Civil and Structural Engineering
312 Pico Blvd., Santa Monica, 90405
310 466-8835

Foundation Plan

CUST SET 04STP1083 9/27/04

TECHNICAL REPORT

SEISMIC REFLECTION AND BOREHOLE SEISMIC INVESTIGATION: NU-WAY LIVE OAK LANDFILL RECLAMATION, NORTHEAST OF THE INTERSTATE 605 FREEWAY AND LIVE OAK AVENUE INTERSECTION, IRWINDALE, CALIFORNIA

Prepared for:

Irvine Geotechnical
145 N. Sierra Madre Blvd., Suite 12
Pasadena, CA 91107

Prepared by:

Wilson Geosciences, Inc.
Altadena, California 91001-2117
626 791-1589
wilsongeo@earthlink.net

January 2008

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WILSON GEOSCIENCES
Engineering and Environmental Geology

January 24, 2008

Jon A. Irvine, PE CEG
Irvine Geotechnical
145 N. Sierra Madre Blvd., Suite 12
Pasadena, CA 91107

Subject: **SITE EVALUATION: Seismic Reflection and Borehole Seismic Investigation: Nu-Way Live Oak Landfill Reclamation, Interstate 605 Freeway and Live Oak Avenue, Irwindale, California**

Dear Mr. Irvine:

INTRODUCTION, SCOPE OF WORK, AND OBJECTIVES

Irvine Geotechnical requested a geophysical survey investigation as a part of an evaluation of the Nu-Way reclaimed (backfilled) aggregate mine north of Live Oak Avenue, Irwindale, California (Figure 1). Current elevations within the backfill area range from approximately 360-feet amsl on the west to 410-feet amsl on the east. Based on information provided by Irvine Geotechnical, sand and gravel mining was well underway by two property owners at the site by 1964 and continued into the mid-1970s when there is evidence that materials were being placed in the open pit by slurry pipe (silt-size material) and by dumping along the edges of the mine. By 1990/1991 a nearly flat bottom was present near elevation 280-feet above mean sea level (amsl). Subsequently larger backfill debris (not placed using conventional geotechnical engineering controls and compaction methods) raised the landfill area elevation to about 370- to 400-feet amsl, after which a 10-foot (±10-feet) clean fill cap was placed over the debris fill. It is understood that the reclaimed mine has a potential for future use as a commercial development.

Materials being mined at quarries within the City are called Older Alluvium and are a mixture of silt, sand, gravel, cobbles, and boulders. In general the alluvial fan deposits are coarser at the northern edge of the City and finer at the southern edge due to the reduction in stream velocity as slope is reduced; the Nu-Way site lies about midway along the alluvial fan. The stratigraphy of the San Gabriel River alluvial fan varies, with lens-shaped deposits merging both up and down the gradient and laterally, creating a series of discontinuous sand/gravel/cobble/boulder layers in some ways similar to an inert landfill sequence.

The Wilson Geosciences Inc. (WGI), in cooperation with *TERRA PHYSICS*, proposed an original scope of work described as a phased program to (1) evaluate the seismic velocity characteristics of the materials underlying the ground surface over the reclaimed mine using seismic reflection geophysics (Task 1) and (2) assist Irvine Geotechnical in the interpretation of drilling and possible down-hole geophysics (performed by others) data to describe the stratigraphy and in-place density-related properties of the backfill reclamation materials (Task 2).

WGI's proposed to conduct an evaluation including the following four scope items, (1) review the available mine-related topographic and mining data pertaining to the site, (2) plan and conduct a minimum of two, and possibly as many as six, seismic reflection survey lines (from the options presented below) to attempt to define the backfill stratigraphy and velocities, (3) evaluate information collected by others (drilling data, core samples, and processed down-hole geophysical logs), and (4) prepare a report describing the study scope, data, analysis, and results. Seismic reflection survey profiles were conducted along two lines. Planned geophysical logs to be performed in conjunction with the drilling were not done and WGI added a borehole seismic survey to this study scope, with the survey being performed in the Irvine Geotechnical Becker Hammer boring B-4 (Task 3). Following the field surveys and during data interpretation, WGI was provided various data sets that were considered in preparing this report, which are described in subsequent sections. The report has two appendices: Appendix A contains Figures 1 through 5 referred to in the report; and Appendix B is the seismic reflection and borehole seismic velocity survey report prepared by *TERRA PHYSICS* (2007).

The project objectives associated with these three tasks were to delineate (1) boundaries between subsurface material changes within the backfilled gravel pit and (2) boundary with natural soils beneath the pit. It was anticipated that near horizontal seismic reflections would indicate these boundary changes and that steeper reflections may represent the gravel vertical wall boundary with the native soils. The ability to delineate such boundaries could assist in: (1) estimating the volume of the fill material of various types and (2) assessing possible constraints on future construction at the site.

SEISMIC REFLECTION AND BOREHOLE SURVEYS

Scope, Survey Design, and Procedures

TERRA PHYSICS (2007) conducted the seismic reflection and borehole geophysical surveys; their report is in Appendix B. Their scope of work, survey design, and survey procedures are presented there along with their summary of results. That information is not repeated in this report, but is drawn upon to address the study objectives, specifically to describe the landfill/backfill layering and distribution, and the landfill bottom configuration along the two profile lines.

TERRA PHYSICS collected seismic reflection data along the two profile lines in areas discussed in advance with Irvine Geotechnical (Figure 2). In the proposal and planning stage WGI discussed the advantages of seismic reflection over refraction due to the likely presence of lower velocity backfill materials (likely placed without engineering controls) underlying controlled compacted backfill placed in the past several years. This velocity inversion creates hidden layers not visible using the seismic refraction technique. The survey plan (line lengths, geophone spacing, offset energy point distances) took this probable lower velocity material into account along with the fact that this is a noisy site with the I-605 Freeway traffic, local truck traffic, and constant plant operations (e.g., rock crushing and conveyor belts) nearby. Also, the target top-of-older alluvium depth (maximum backfill thickness) was estimated at about 200 feet so that the survey was designed to reach a 25 percent greater depth. No other site-specific subsurface data were provided prior to conducting the surveys.

During the survey and in subsequent initial data analysis it was clear that the site was underlain by at least one high velocity zone as determined from seismic refraction interpretation. This information is normally applied to a reflection survey to establish a velocity profile to assist in converting the reflection arrival times to actual depth below the ground surface. *TERRA PHYSICS* judged that this velocity data was not appropriate to use for depth conversion and that it would be prudent to suspend analysis until a borehole seismic velocity survey by *TERRA PHYSICS* was completed.

General Results and Findings

The two seismic reflection profiles delineated boundaries that correlate well with reported conditions at the same approximate elevations in the backfill materials and with the top of native older alluvium at the base on the backfill (Irvine Geotechnical, 2007; Zeiser, 1991). Seismic reflection and velocity characteristics at the selected locations are related to in-situ densities of the subsurface inert material, and variations of same, filling the sand and gravel mine. Borehole velocity survey results in boring B-4 (Figure 1 and Table 1) defined the vertical compressional and shear wave velocity profiles at this location; these four velocity zones correlate with the seismic reflection profiles and with the Becker Hammer boring uncorrected blow counts for B-4 and B-1 near the seismic reflection profiles, as well as allowing calculation of Poisson's Ratio for the landfill velocity zones and underlying older alluvium (discussed in Appendix B).

Table 1 – Borehole Seismic Wave Velocities Measured in Boring B-4

Velocity Zone-- [Computed Poisson's Ratio]	Compressional Wave Velocity [Feet/Second]		Shear Wave Velocity [Feet/Second]	
	Value (with uncertainty)	*Probable Range (rounded)*	*Value (with uncertainty)*	*Probable Range (rounded)*
1---[0.387]	2050 ± 6%	1930-2175	880 ± 10%	790-970
2—[0.441]	4250 ± 8%	3910-4590	1380 ± 8%	1270-1495
3—[0.336]	5150 ± 10%	4635-5665	2560 ± 15%	2170-2945
4—[0.318]	6200 ± 7%	5765-6635	3200 ± 12%	2815-3585

Ambient noise was a factor in the quality of seismic data gathered at the site. In particular, high amplitude noise from the I-605 freeway and processing plants prevented the identification of coherent seismic shear waves at both horizontal geophones in the borehole velocity survey. However, by using the information in this report for survey planning and execution, future seismic reflection and borehole velocity surveys can be used to define internal and bottom-of-backfill boundaries at the Nu-Way site. Survey design is an important factor affecting depth of penetration for seismic reflection and the reflector detail that can be discerned in the profiles. Future surveys would be designed with a smaller geophone separation and shorter offset energy input distances, because the subsurface velocities were higher than initially anticipated. A specific seismic reflection data processing procedure was developed in parallel with, but separate from, this study that should improve the results from future seismic reflection surveys.

COMPARISON OF IRVINE GEOTECHNICAL BECKER HAMMER DRILLING RESULTS TO BOREHOLE VELOCITY SURVEY

Irvine Geotechnical provided the Becker Hammer uncorrected blow counts for borings B-1 through B-6; results from borings B-4 and B-1 were analyzed since they are close to the seismic reflection Profile Nos. 1 and 2. A borehole seismic velocity survey was conducted in boring B-4. WGI used the uncorrected blow count vertical profiles to compare with the borehole seismic velocity profiles and with the seismic reflection profiles near boring B-4. An Excel file containing the individual blow counts for B-4 and B-1 was analyzed to determine the approximate average blow count values for the various subsurface zones.

TERRA PHYSICS collected borehole compressional and shear wave velocity profile data from Irvine Geotechnical boring B-4, which is compared to the Becker borings B-4 and B-1 uncorrected blow counts (Figures 3 and 4). The seismic velocity survey revealed four distinct compressional and shear wave velocity zones and showing progressively increasing velocity with depth in contrast to assumptions underlying the seismic reflection survey field data collection plan described above. Figures 3 and 4 show depth below ground surface (bgs) along the left vertical axis and adjusted compressional wave travel time in milliseconds along the top horizontal axis; approximate elevations are shown along the right vertical axis. The compressional seismic velocities are (from shallowest to deepest): Layer 1 2050 feet per second (fps); Layer 2 2250 fps; Layer 3 5150 fps; and Layer 4 6200 fps. The shear seismic velocities are (from shallowest to deepest): Layer 1 880 feet per second (fps); Layer 2 1380 fps; Layer 3 2560 fps; and Layer 4 3200 fps.

The boundaries between shear wave velocity zones are shown by the horizontal yellow dashed lines. This velocity data is overlain by the uncorrected Becker blow count plots from boring B-4 (Figure 3) and boring B-1 (Figure 4) with the blow counts shown in the bold type (zero through 200 blows per foot) along the top horizontal axis. Geologic interpretation of the blow counts suggests possible internal backfill zone "contacts" where the blow counts change character; these are shown by the horizontal brown (B-4) and blue (B-1) dashed lines. There is a reasonable correlation between the shear wave velocity zone boundaries and the blow count changes as shown where the near coincidence of the yellow and brown/blue dashed lines are near the various depths. At approximately 163-feet bgs in boring B-4 a lower blow count layer with more consistent counts appears to indicate the native underlying older alluvial sands and gravels. We would expect the compressional wave velocity to decrease at this depth, but that is not observed in the data because the velocity would not have dropped below the water saturation velocity, i.e. about 5000 fps.

COMPARISON OF IRVINE GEOTECHNICAL CROSS-SECTIONS WITH SEISMIC REFLECTION PROFILES

Figures 5A and 5B show the positive correlation between the seismic reflection Profile Nos. 1 and 2 and their corresponding portions of the initial "preliminary" Irvine Geotechnical (2007) cross-sections C and A (referred to as C-C' and A-A'). Since the vertical scales of the seismic reflection profiles are different above and below elevation 350-feet, it was necessary to perform some scale adjustments to match the cross-sections and the profiles. The cross-sections were set over the profiles as "transparencies" to reveal the contacts over the original geophysical interpretations. The cross-sections were adjusted to the proper horizontal scale by matching the Irvine Geotechnical and Zeiser borings in the two data sets. Then the cross-section elevations above and below 350-feet were adjusted proportionately (expanded above 350-feet and compressed below 350-feet) to roughly match the vertical scale of the profile over these intervals. To highlight the cross-section contacts they were traced using rather broad lines intended to show form and shape rather than imply more accuracy or precision than the data justify. The original geophysical annotations are shown in the same colors so that the similarities and differences can be seen. It is important to note that the original geophysical interpretations were made considering the generalized stratigraphy at B-4 provided by Irvine Geotechnical, the Becker uncorrected blow counts for B-4 and B-1, and the borehole seismic velocity data; this was done before the preliminary Irvine Geotechnical (2007) cross-sections were provided so that the two interpretations are independent. For clearer and more detailed views of the seismic reflection profiles, including annotations and estimated seismic reflection uncertainties, please see Appendix B.

Generally the original geophysical interpretation matches the cross-section interpretation between B4 (south of about station 750) and B-1 on Profile No. 1 and west of B-4 on Profile No. 2. Areas that differ include the areas (1) south of B-1 and north of B-4 on Profile No. 1 and (2) east of B-4 on Profile No. 2. In the areas north of about station 750 on Profile No. 1 and east of station 750 on Profile No. 2 we interpreted either un-mined older alluvium near the surface or a poorly bedded coarse-grained rubble fill that was dumped over the eastern edge of the pit before or during the period when the rubble fill was placed between the older silt fill and the clean fill cap. Based on aerial photograph interpretations by others received after the original geophysical interpretations, it appears that this area was excavated (we are uncertain of the depth) as early as 1960 and was filled with rubble-type material beginning in 1975-1976. The cross-sections clearly show this area was excavated, so that we believe this is a less well-layered rubble fill probably with some larger blocky material (e.g., concrete construction debris).

SUMMARY COMMENTS AND CONCLUSIONS

1. Based on the comparison of borehole seismic velocity and uncorrected Becker Hammer blow counts, it appears that (a) there is a strong correlation at the locations tested and (b) there is a similar backfill "stratigraphy" in each boring at a separation distance of about 650-feet.

2. Even though the seismic reflection planning was based on a conceptual model of (a) relatively low seismic velocity non-engineered reclamation backfill and (b) the likelihood of a velocity inversion (higher density over lower density), the reflection method interpretations matched well the preliminary geotechnical cross-section constructed from past landfill maps and borings.

3. Seismic velocity is positively correlated with earth material unit weight/bulk density. The seismic velocity structure of the portion of the Nu-Way reclamation area surveyed indicates there is an increase in seismic velocity with depth and that the backfill properties (compressional and shear wave velocities, blow counts, and *TERRA PHYSICS* computed Poisson's Ratio) of the four velocity zones are similar to sand and gravel-rich alluvial fan materials and potentially superior to other examples of inert backfill.[1]

[1] For a recent project in San Diego County, *TERRA PHYSICS* seismic refraction surveys within an active business park (high technology single and multi-story offices) underlain by up to 40-feet of backfill documented in several borings (engineered and placed in the 1970s) showed a compressional wave velocity in the range of 2700 to 3400 fps (with some areas 1200 to 2000 fps). Another project over a several decades old landfill in the San Fernando Valley area showed compressional wave velocities in the range of 1380 to 2300 fps for non-engineered construction and residential waste, with one lower layer at 3550 fps. While this is anecdotal supporting evidence, it may be possible that the velocities shown in Figures 3 and 4 are indicative of in-place backfill densities higher than would have been assumed for a landfill reclaimed without consistent engineering controls (e.g., continuous compaction testing and engineering inspections).

4. The study objectives were met with regard to delineation of boundaries between subsurface material changes within the backfill, and between the backfill and the natural soils beneath the pit. Steeper sloping surfaces, possibly indicating materials pushed over the edges of the pit, indicated a transition between areas of more and less coherent seismic reflectors. With sufficient coverage the seismic reflection data could assist in estimating the thickness and volume of the various backfill types.

REFERENCES CITED

Irvine Geotechnical, 2007, Becker Hammer blow counts for borings B-1 through B6; stratigraphy descriptions and preliminary cross-sections A, B, and C; and site maps with boring locations and topography for 1991 and 2007.

TERRA PHYSICS, 2007, Seismic Reflection and Borehole Seismic Velocity Surveys to Delineate Subsurface Backfill Material and Underlying Native Soil Boundaries Nu-Way Reclaimed Aggregate Mine Landfill - Irwindale, California, dated January 7, 2008

Zeiser Geotechnical, Incorporated, 1991, Preliminary Geotechnical Investigation–Owl Rock Quarry Site, Irwindale, California, PN90346-00, dated February 4, 1991.

If you have questions about this report, please contact me (626 791-1589) at your convenience.

Sincerely,
WILSON GEOSCIENCES, INC.

Kenneth Wilson
Principal Geologist
P.G. #3175, C.E.G. #928

APPENDIX A
Figures 1 through 5

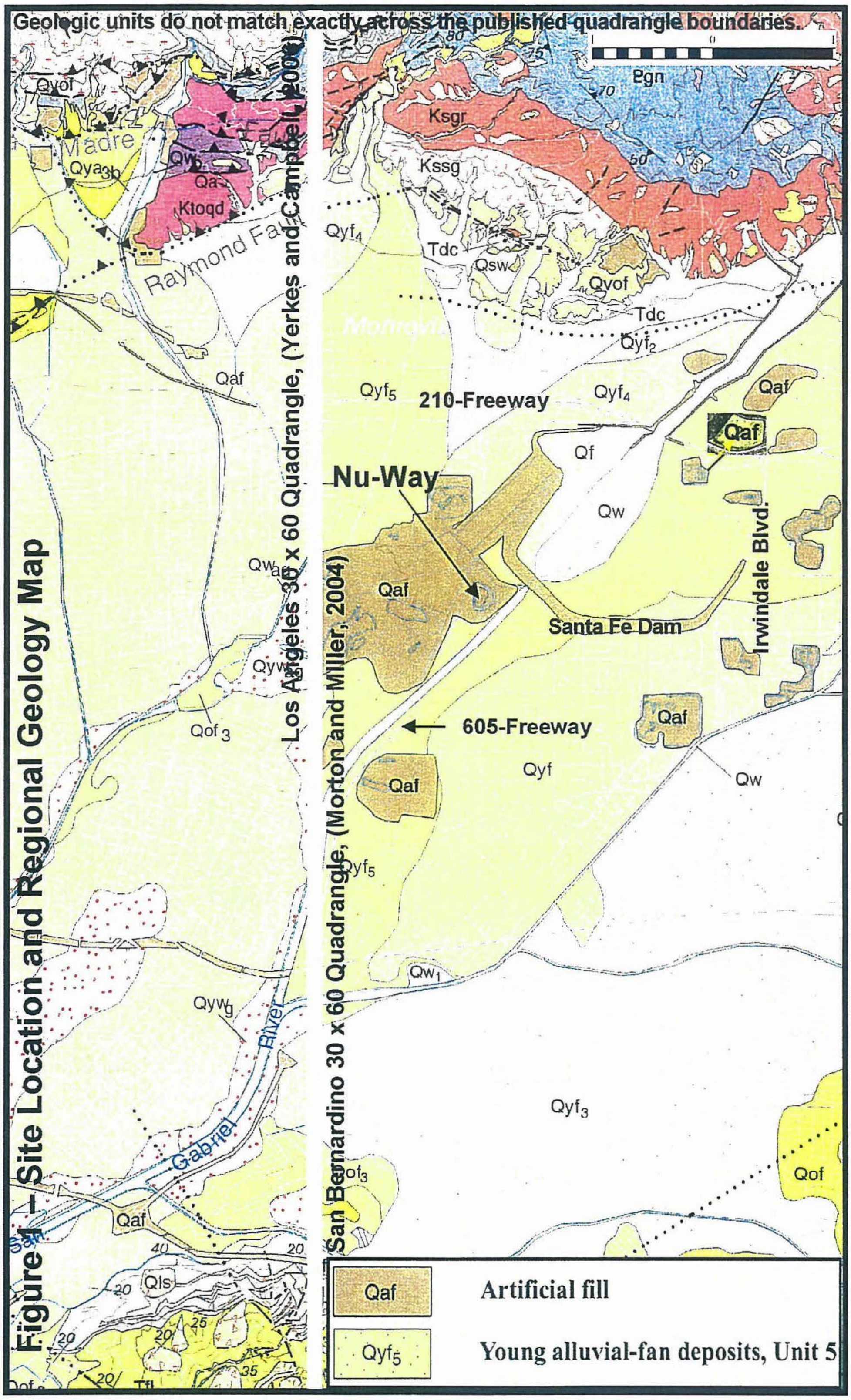


Figure 1 – Site Location and Regional Geology Map
Geologic units do not match exactly across the published quadrangle boundaries.
Los Angeles 30 x 60 Quadrangle, (Yerkes and Campbell, 2005)
San Bernardino 30 x 60 Quadrangle, (Morton and Miller, 2004)
Raymond Fault
Madre
Qvof
Qya3b
Qw
Qa
Ktoqd
Ksgr
Kssg
Tdc
Qsw
Qvof
Qyf4
Qyf2
Qyf5
210-Freeway
Qaf
Qf
Qw
Nu-Way
Santa Fe Dam
Irwindale Blvd.
605-Freeway
Qyf
Qof3
Qyw
Qw1
Qyf3
Qof
San Gabriel River
Qls
Qaf
Artificial fill
Qyf5
Young alluvial-fan deposits, Unit 5

FIGURE 2 – SEISMIC REFLECTION PROFILE NOS. 1 AND 2, AND BECKER BORINGS B-4 AND B-1

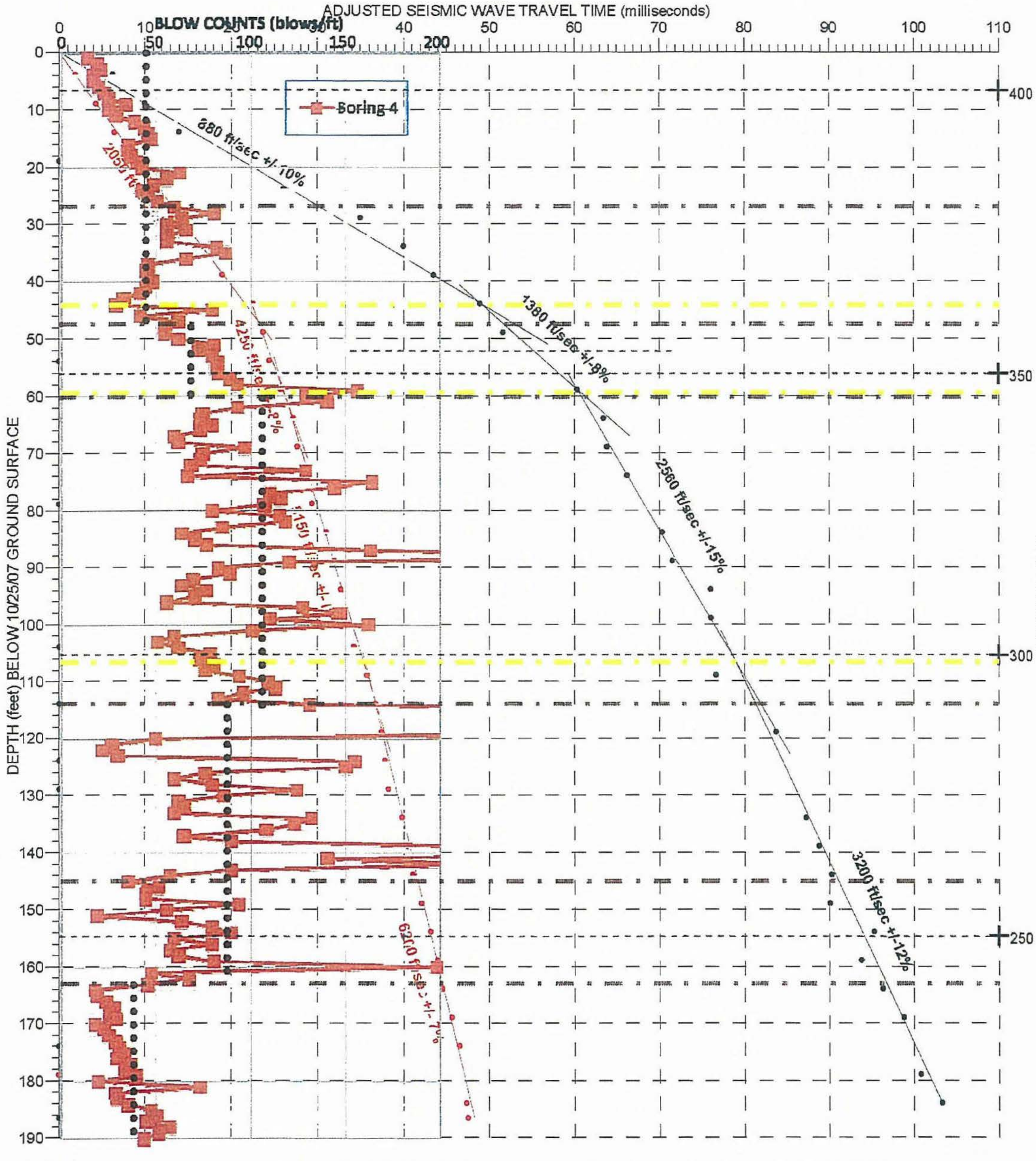


FIGURE 3 - COMPARISON OF DOWNHOLE VELOCITY SURVEY WITH BECKER BLOW COUNTS FROM B-4

- COMPRESSIONAL WAVE DATA
- SHEAR WAVE DATA
- INTERPRETED SEISMIC VELOCITY (feet/second) WITH ESTIMATED UNCERTAINTY.

- Interpreted Blow Count Change
- Shear Wave Velocity Change

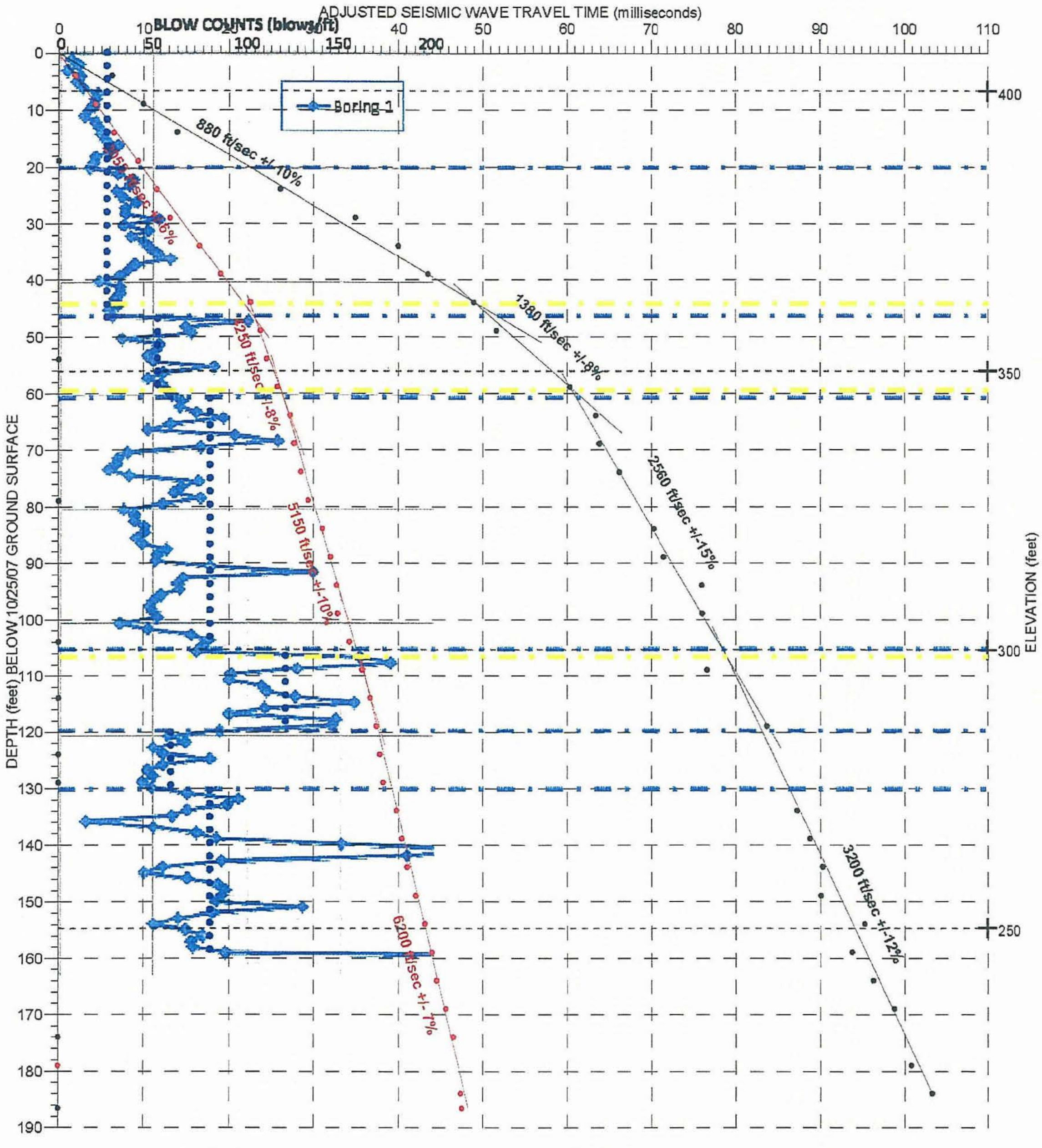


FIGURE 4 - COMPARISON OF DOWNHOLE VELOCITY SURVEY WITH BECKER BLOW COUNTS FROM B-1

- COMPRESSIONAL WAVE DATA
- SHEAR WAVE DATA
- INTERPRETED SEISMIC VELOCITY (feet/second) WITH ESTIMATED UNCERTAINTY.

Interpreted Blow Count Change
Shear Wave Velocity Change

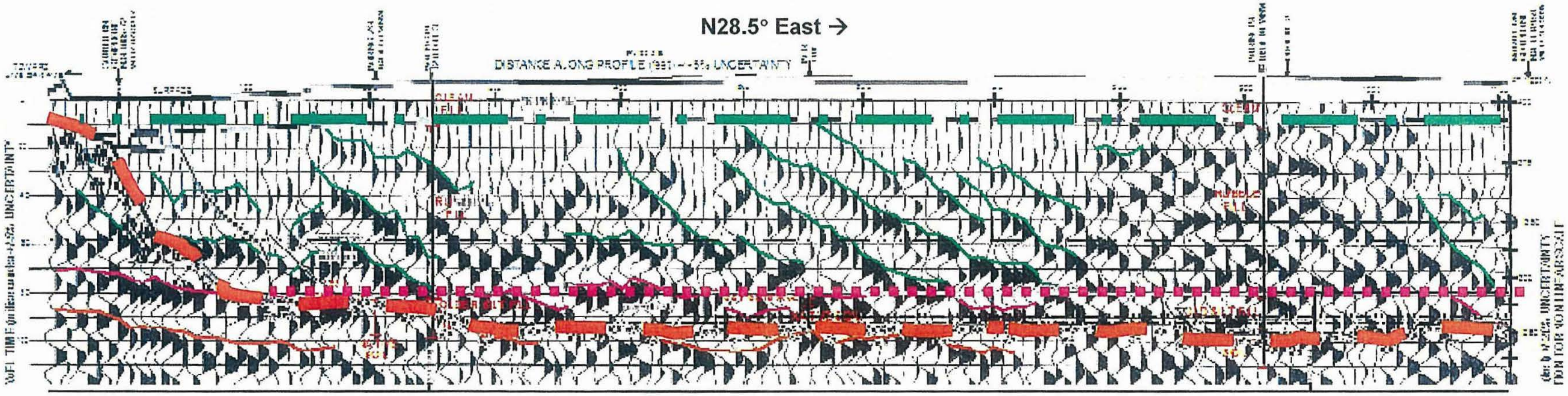


FIGURE 5A - SCHEMATIC CROSS-SECTION C-C' COMPARED TO PROFILE NO. 1

EXPLANATION

thin green line	Rubble Fill	Geophysical Interpretation without Prior Reference to the Cross-Sections
thin magenta line	Top of Older Silt Fill	
thin orange line	Top of Older Alluvium	
thick green bar	Top of Rubble Fill	Approximate Tracing of Superimposed Irvine Geotechnical Cross-Section Contacts
magenta squares	Top of Older Silt Fill	
thick orange bars	Top of Older Alluvium	

NOTES

Portions of the Irvine Geotechnical (2007) cross-sections C-C' and A-A' were adjusted to the proper horizontal scale, then elevations above and below 350-feet were adjusted proportionately to roughly match the vertical scale of the seismic reflection Profile Nos. 1 and 2, respectively. For clearer and more detailed views of the cross-sections and seismic reflection profiles with annotations, please see Appendices A and B.

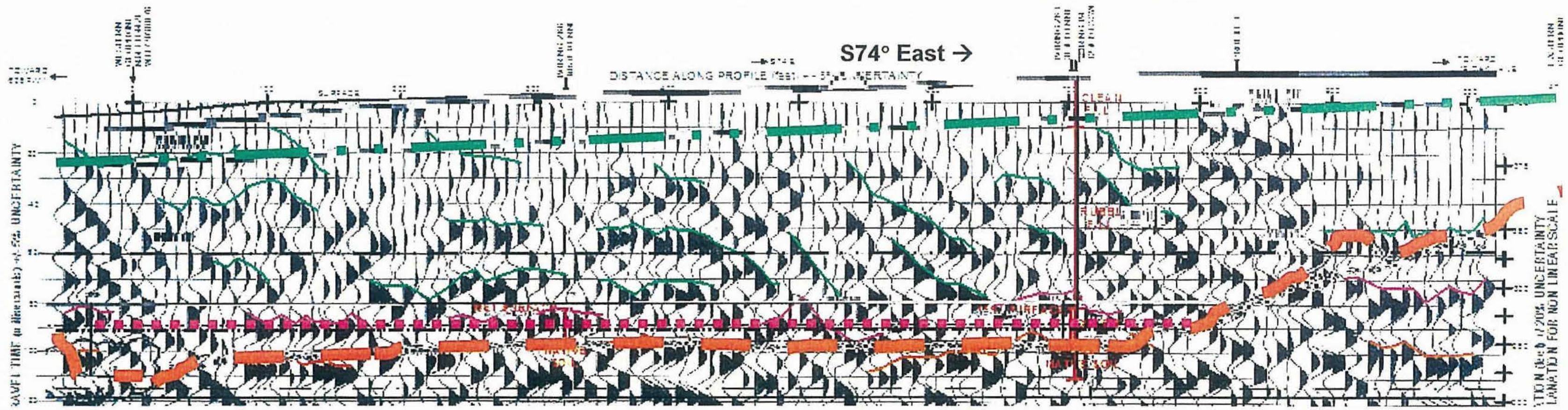


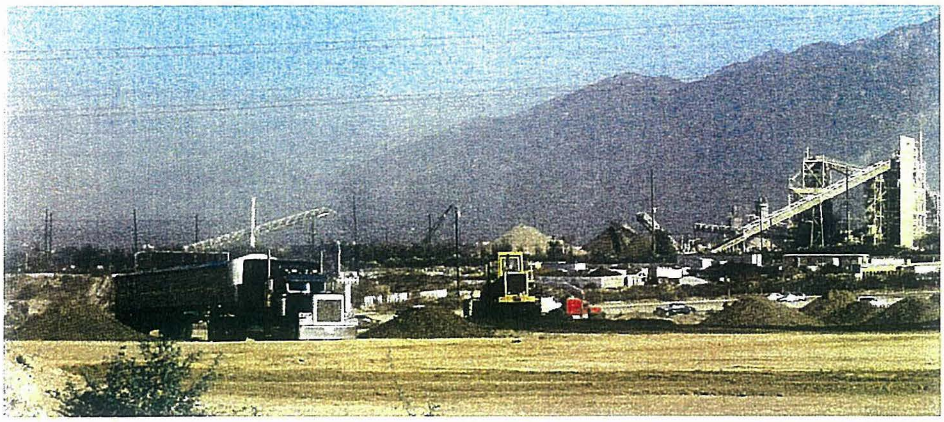
FIGURE 5B - SCHEMATIC CROSS-SECTION A-A' COMPARED TO PROFILE NO. 2

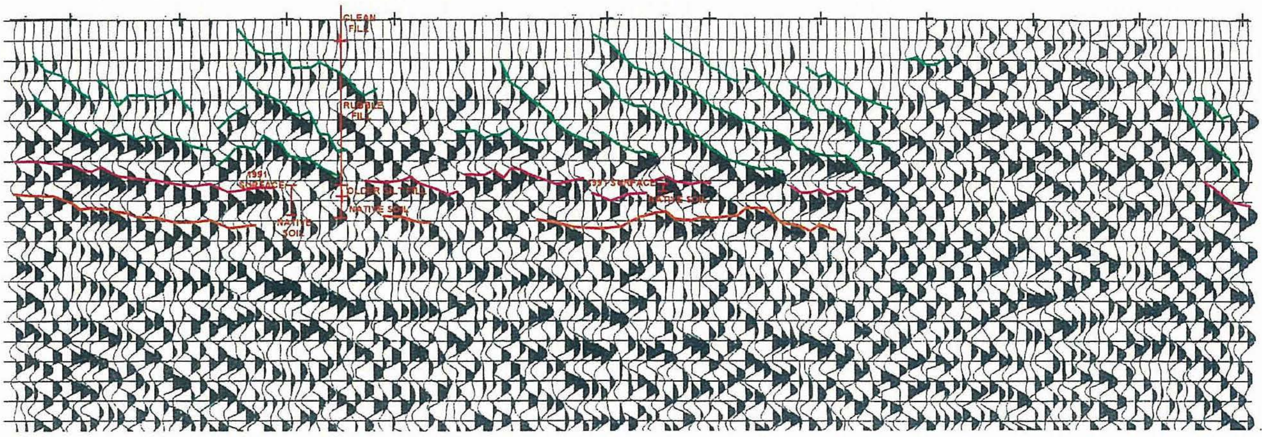
0916.0463

APPENDIX B

TERRA PHYSICS (2007) Seismic Reflection and Borehole Seismic Velocity Survey Report

SEISMIC REFLECTION AND BOREHOLE SEISMIC VELOCITY SURVEYS TO DELINEATE SUBSURFACE BACKFILL MATERIAL AND UNDERLYING NATIVE SOIL BOUNDARIES NU-WAY RECLAIMED AGGREGATE MINE LANDFILL - IRWINDALE, CALIFORNIA





GP 886

Prepared For:
Mr. Ken Wilson, CEG
WILSON GEOSCIENCES
1910 Pinecrest Drive
Altadena, CA 91001

Prepared By:
Mr. Kerry Hennon, PGp-886
TERRA PHYSICS
28841 Base Line Road
Highland, CA 92346

TERRA PHYSICS Project No.: 07-20
January 7, 2008

GEOPHYSICAL SURVEY LIMITATIONS

Geophysical exploration is not an exact science, only an additional tool used to locate subsurface material boundaries and measure their physical properties. *TERRA PHYSICS* is not a guarantor of the services provided, but agrees to perform services in a professional and non-negligent manner and according to information and data available to us. Users of this report should recognize the extreme difficulty in locating undocumented, subsurface material boundaries due to factors such as changing stratigraphy and hydrology and the proximity of near-surface sources of vibrational and electrical noise.

Data and results presented in this report were compiled considering the cited existing geological data and the current surveys. Geophysical interpretation of subsurface conditions from the surface measurements are not unique. These results represent reasonable descriptions of the geological conditions and are presented for information only. The results should be verified by direct investigation methods. Complex subsurface geology may prevent reliable extrapolation of these results away from their original measurement locations.

TERRA PHYSICS reserves the right to review this report's results when additional information concerning this investigation is available in the future.

TERRA PHYSICS 28841 Base Line Road Highland, CA 92346 phone/fax (909)862-0626
TERRAPHYSICS@AOL.COM

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TERRAPHYSICS@AOL.COM

1.0 INTRODUCTION

This report describes the seismic reflection survey designed to delineate subsurface boundaries between various inert fill materials and the native soil at the Nu-Way Landfill on Live Oak Lane, Irwindale (Figure 1). The survey is in support of the on-going Wilson Geosciences project to characterize subsurface conditions at the site. Report Section 1 describes the survey scope and design, and Section 2 explains the results. Sections 3 and 4 describe all survey procedures and list the cited references, respectively.

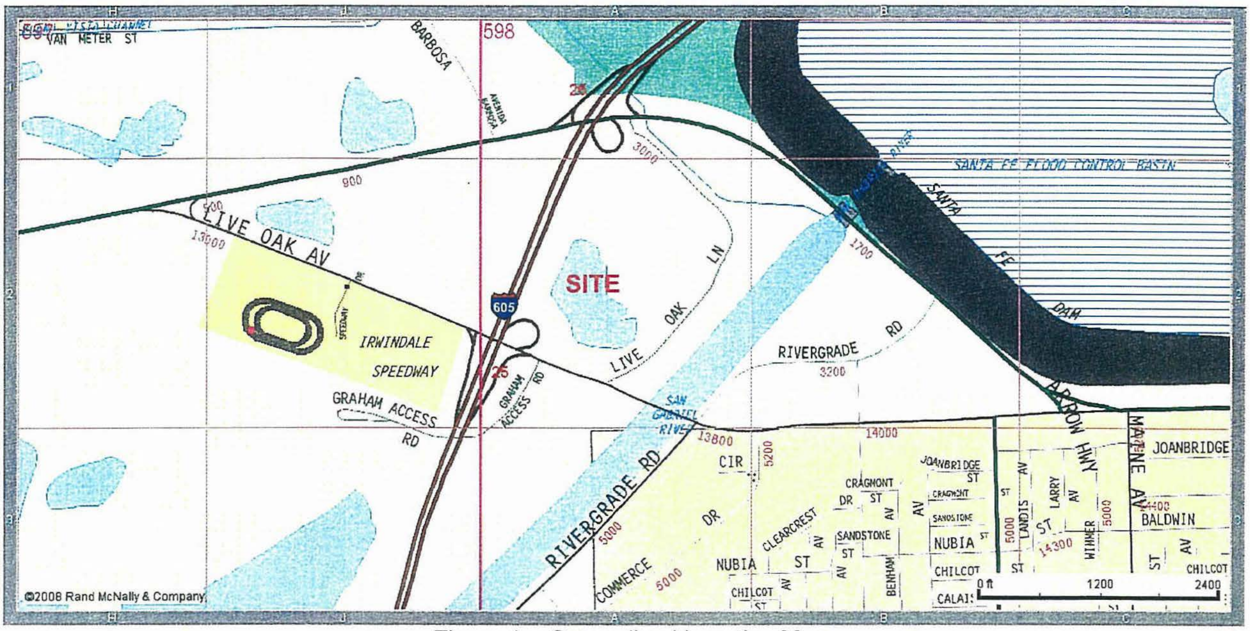

Figure 1 – Generalized Location Map

1.1 Scope Of Work

The survey work scope is to evaluate the seismic velocity characteristics as related to in-situ densities of the subsurface inert backfill materials placed as part of the reclamation of the previous sand-gravel quarry. The specific objectives are to delineate boundaries in the backfill materials and the top of native soil.

1.2 Survey Design

This survey consisted of two distinctly different seismic tests; two surface seismic reflection lines and one borehole seismic velocity profile. Seismic methods were selected for this survey because seismic velocity changes occur between materials with different densities (Burger, 1992). Seismic energy generated at source points along the surface creates waves that travel downward into the ground. Propagation velocities of these waves are directly related to material density. Some of the seismic energy is reflected or refracted back to the surface by boundaries between geological or backfill materials with different acoustic properties. During survey planning discussions with Irvine Geotechnical and Wilson Geosciences, it was anticipated that uncontrolled backfilling practices likely lead to less dense materials in the deeper subsurface. This would create a velocity inversion (denser over less dense backfill) and negate advantages of surface seismic refraction method. The surface seismic reflection method was selected. Also, the target bottom-of-backfill depth was estimated at about 200 feet so that the survey was designed to reach a 25 percent greater depth. No other site-specific subsurface data were provided prior to conducting the surveys.

1.2.1 Surface Seismic Reflection

Wilson Geosciences located the seismic reflection survey profiles as two, approximately perpendicular, lines passing through proposed building footprints as provided by Irvine Geotechnical. During the surface seismic reflection survey, the reflected seismic waves are detected by an array of surface geophones (vibration sensors) and recorded. The entire array is then moved along each profile (Profiles 1 and 2) and the next set of measurements is made. Reflection analysis of the data provides detail lateral and vertical resolution of the topography of the velocity zones. It was anticipated that this site will produce only short segments of reflectors within the backfill that would vary in amplitude and frequency because of how the backfill was placed and compacted.

Figure 2 shows (as black lines) the geophone arrays used to record the data. Shorter red lines represent locations of the final processed data sections. Offset between these lines are caused by the processed data traces being plotted midway between geophones and source locations. The red line should be used to relate interpreted boundaries to other site features. Figures 3 (Irvine Geotechnical, 2007) and 4 (Zeiser Geotechnical, Incorporated, 1991) show the processed section locations on the March 2007 and 1991 topography maps.

Looking North At The North End Of Profile 1

Profile 1 was 1157 feet long, oriented N 28.5 degrees E and may extend from the original quarry pit south perimeter northward past the bench area shown in Figure 4. Profile 2 was 1088 feet long, oriented S 74.0 degrees E, and may extend to the same eastern bench area.

Lateral resolution was increased by placing geophones close together at 17.0 feet intervals. Data quality was enhanced by three design factors. Source points were located at 17.0 feet intervals (same as geophone spacing) so the processed sections would be 12 fold (a measure of signal to noise ratio). A 500 pound weight drop generated waves with large enough amplitude to overcome constant background vibrational noise from the surrounding processing plants. Combining 12-15 individual drops to form a single data record increased the true seismic signal amplitude while minimizing the randomly occurring background noise. The seismic source offsets to the nearest and farthest sensors were 102 and 493 feet. This geometry was designed for about 250 feet of penetration (about 25% deeper than the initial quarry pit depth estimate). Table 1 describes the seismic measurement locations.

Looking West From Profile 2 Center

TABLE 1
SEISMIC SURVEY LOCATIONS

ID	LATITUDE (degrees)	LONGITUDE (degrees)	MEASUREMENT LENGTH/DEPTH (feet)	ORIENTATION (degrees)
Profile 1-Processed Section South End	34.1084252	117.9769074	1157	N 28.5 E
Processed Section North End	34.1112607	117.9751425		
Profile 2-Processed Section West End	34.1114811	117.9781917	1088	S 74.0 E
Processed Section East End	34.1106918	117.9747826		
Boring B1	34.1090787	117.9763212	180	---
Boring B4-Borehole Velocity Survey	34.1108855	117.9757761	190	---

NOTES: Coordinates in NAD83 datum for use with most GPS instruments and have an estimated uncertainty of ±2 feet.

1.2.2 Borehole Seismic Velocity Profile

Vertical borehole seismic profiles of compressional and shear wave velocities were measured by generating seismic waves with hammer hits on the surface and recording their arrival times with geophones at many depths within a boring (Section 3.2). This test was conducted in Boring B4 (Figure 2) located about 12 feet south-southwest of 707 feet along Profile 2. The wave source point was located 10.0 feet southwest of the boring. Data were recorded by the geophones every five feet from the surface to the maximum accessible depth of 187.5 feet. Depths were measured relative to the ground surface (elevation of 407 feet) as of October 25, 2007. The resulting compressional velocity profile was used for the many geometric corrections applied to the reflection data (Section 3.3).

2.0 RESULTS SUMMARY

The *TERRA PHYSICS* seismic survey (Figure 1) at the Nu-Way Landfill on Live Oak Lane in Irwindale satisfied the objectives of evaluating the seismic velocity characteristics as related to in-situ densities of the subsurface inert material filling the sand-gravel quarry. The borehole velocity survey in Boring B4 (Figure 2) defined the vertical compressional and shear wave profiles. Two reflection profiles delineated boundaries in the backfill materials and the top of native soil.

After completion of the *TERRA PHYSICS* seismic surveys data collection, Irvine Geotechnical (2007) provided: 1) blow count data for Becker Borings B1 through B6 and 2) a generalized stratigraphy within the landfill indicated as surficial Clean Fill underlain by Rubble Fill that was placed on the 1991 quarry surface on top of an older Silt Fill. The uncorrected blow counts show widely fluctuating values that indicate the backfill materials are not homogeneous laterally and vertically presumably because of variations in compaction and the presence of various sized concrete fragments. The underlying native soils are also non-homogeneous, Borings B3 and B4 show a blow count decrease (less dense) relative to the backfill materials, but the other four borings show no general trend.

2.1 Borehole Velocity Survey

The compressional and shear wave velocities are described in Figure 5 and Table 2. The results show four distinct velocity zones with boundaries that agree well with changes in the uncorrected blow counts, but do not correlate well with the generalized stratigraphy boundaries. Poisson's Ratio was calculated from the velocities. Other dynamic elastic moduli can be calculated once the bulk density of the backfill material is known.

TABLE 2
CORRELATION OF SEISMIC VELOCITIES WITH STRATIGRAPHY AND BLOW COUNTS

DEPTH RANGE[1] (feet)	ELEVATION RANGE[1] (feet)	COMPRESSIONAL VELOCITY (feet/second)	SHEAR VELOCITY (feet/second)	POISSON RATIO	LITHOLOGY[2]	UNCORRECTED BLOW COUNTS[2]
0 – 45	407 – 362	2050 ± 6%	880 ± 10%	0.387	Clean/Rubble Fill	10 - 85
45 – 60	362 – 347	4250 ± 8%	1380 ± 8%	0.441	Rubble Fill	25 - 155
60 – 111	347 – 296	5150 ± 10%	2560 ± 15%	0.336	Rubble Fill	50 - >200
111 – 187	296 – 220	6200 ± 7%	3200 ± 12%	0.318	Rubble Fill Older Silt Fill Native Soils	15 - >200

Notes: 1. Depths measured relative to the ground surface at the boring (not the casing top). Interpreted depths between velocity zones have an estimated uncertainty of ±2 feet.
2. Stratigraphy and blow counts from Irvine Geotechnical (2007).

In general, the deeper three velocities are much faster than expected for a compacted fill which usually is less than 3500 feet/second. The relatively large velocity uncertainties and large amplitude blow count changes both indicate the backfill material is not homogeneous.

In the backfill material, the change from Clean to Rubble fill did not produce recognizable velocity or blow count changes. The boundary between the upper two velocity zones coincides with a sudden increase in blow counts from less than 50 (depths of 36-45 feet) to 50-

80 (below 45 feet) which indicates an increase in material density. The Rubble Fill encompasses three velocity zones and blow counts that vary irregularly from 30 to more than 200. The change from Rubble Fill to older Silt Fill did not produce recognizable velocity or blow count changes. Saturation below 160 feet did not generate the characteristic compressional velocity of 4800-5400 feet/second (speed of sound in water) because the dense material velocity at that depth was already much higher. Shear waves are not affected by saturation changes. In the native soil, the expected decrease in blow counts occurred about 10 feet deeper than where the native soil was indicated by Zeiser (1991) at 153 feet. A similar blow count decrease was seen at Boring B1 but not at the other four borings. Neither velocity decreases in the native soils so there is no evidence of a less dense material.

2.2 Seismic Reflection Survey

This survey consisted of two approximately perpendicular reflection lines, each more than 1000 feet long and a maximum penetration of about 350 feet. (Penetration was deeper than designed because the backfill material was much faster than anticipated.) Profiles 1 and 2 processed data sections (location and depth uncertainties are ±5% and ±20%, respectively) with interpreted subsurface boundaries and lateral features are shown in Figures 6 and 7. Interpretation of the sections was an iterative process undertaken with Wilson Geosciences. It is important to note that the right side vertical axes on Figures 6 and 7 have two different scale values because two velocity zones were used to convert travel time to depth. In the upper 50 feet (elevations 350-400 feet), the borehole velocity used was 2050 feet/second (dry, Clean/Rubble Fill) which yields a vertical scale of 1 inch to about 42 feet. Below elevation 350 feet, an average (5000 feet/second) of the three deeper borehole velocities were used which yields a vertical scale of I inch to about 92 feet.

Boring B4 borehole velocity survey shows velocity changes correlate to blow count changes not the stratigraphy changes provided. Reflection segments highlighted in Figures 6 and 7 are discontinuous as expected for the backfill material. The PURPLE reflectors were assigned as the top of Older Silt Fill (1991 quarry surface) and the ORANGE reflectors native soil by correlating Borings B1 and B4 blow counts with the highlighted reflectors. GREEN shallower reflectors show possible backfill layering. Deeper reflectors within the native geology below the landfill are not considered in this report.

Profile 1 (oriented north 28.5 degrees east) may not have encountered the original quarry's southern perimeter as shown in Figure 4 because there is no steeply dipping quarry bottom reflector trending northward. Irvine Geotechnical Geologic Cross Section C shows the southern quarry perimeter was trimmed to a 1:1 slope in 1991 which is about 50 feet further south than the original edge shown in Figure 4. No coherent reflectors were identified between distances of about 710 to 1070 feet. This feature is interpreted as the unmined bench area extending out into the quarry from the eastern perimeter (Figure 4).

Profile 2 (oriented south 74 degrees east) did not detect the quarry's western perimeter in agreement with Irvine Geotechnical Cross Section A because there are no steeply dipping reflectors trending eastward. The same unmined bench area crossed by Profile 1 is interpreted to cause the lack of coherent reflectors between about stations 770 and 900 feet.

3.0 DESCRIPTION OF SEISMIC SURVEY EQUIPMENT AND PROCEDURES

ta collection, processing, and interpretation described in this section were in accordance with *TERRA PHYSICS* (2000) Geophysical Survey Procedures which follows California Department of Health Services (2000) guidelines. Borehole velocity test also followed ASTM D4428 (2000) for crosshole measurements. There are no ASTM standards for surface seismic reflection surveying.

3.1 Equipment

Equipment consists of a seismic wave source, borehole, and surface geophones for sensing the seismic waves, and a recording system. Seismic waves can be generated with a variety of sources (hammer, projectile impact, weight drop, and vibrator). The borehole velocity survey used a 20 pound hammer hitting a 1 foot diameter metal plate lying on the surface. For the reflection survey, creating seismic waves with amplitudes much larger than the background vibrational noise was of paramount importance. Therefore, a 500 pound weight dropped 3.4 feet onto a 2.5 x 2.5 metal plate lying on the ground. Frequency spectra of data recorded by geophones about 20 feet for these sources show frequencies as high as 100 Hz were generated which was more than adequate for defining thin layers. To enhance the data signal-to-noise ratio, 3 individual hammer hits for the borehole and 12-15 individual weight drop hits for the reflection were stacked together to form each data record. Both sources were fitted with a timing circuit that sends an electrical signal to the recorder at the instant of wave generation (uncertainty ±0.00001 seconds).

For the borehole velocity measurements, a Mark Products model L10-3WD geophone package with ree mutually perpendicular geophones (10 Hz critically damped resonant frequency) was lowered into the boring casing. The package has a side spring that triggered when it hit the bottom cap thus pressing the geophones tightly against the casing to maximize coupling and improve data quality.

For the reflection measurements, seismic waves were sensed by vertically oriented, Mark Products model L-40 geophones (critically damped 40 Hz resonant frequency). The geophones were fitted with small metal spikes that were forcibly pushed into the ground to improve coupling and thus data quality. Geophones were connected with Mark Products cables.

The geophones' electrical signals were input to a Geometrics model R-48 (S/N 75168) seismograph running software version V3.2 (Geometrics, 2000). This system has one of the largest dynamic ranges of any engineering seismograph. The system is capable of filtering, processing, displaying, and recording 48 channels of data simultaneously. Data were recorded on the internal hard disk and then transferred to PC for later processing. Hard copy records were made during data collection to evaluate data quality and adjust measurement parameters when necessary.

The system has been maintained and was operated according manufacturer's recommendations. At the beginning of each data collection day, a functional calibration test was performed to check the system's timing line accuracy. An external 100 kHz pulse generator was connected to the geophone input terminal of the seismograph. A record was made of the 0.001 second period pulse. Pulse width compared within ±1% to the timing line spacing.

Reflection profiles were constructed by stretching a measuring tape between the staked ends. ›eophone locations have an estimated uncertainty of ±0.05 feet along the tape. Relative geophone ›levations were measured with a Pentax Total Station (Pentax, Incorporated, 1995) model PTS-III_{10} (S/N 210645) and have an estimated uncertainty about ±0.2 feet. Geodetic coordinates of end points were measured with a Trimble (Trimble, 2001) model Ag-114 (S/N 0224018798) with a Jupiter Systems ALLEGRO field computer (S/N 1019) running 'Landmark' version 4.2 software and have an estimated uncertainty of ± 2 feet. Absolute elevations of profile ends were measured with the Trimble and have an estimated uncertainty of ± 3 feet.

3.2 Borehole Velocity Survey Procedures

The following procedures were performed at five feet intervals from near the casing bottom (187.5 feet). Measurement depths of the geophones were relative to the ground surface at the boring (not the casing top).

- Compressional waves were recorded with vertical hammer hits on a metal plate located 10.0±0.1 feet from the casing center . (The offset distance provided a straight path for wave travel through relatively undisturbed soil.) The largest amplitude compressional waves were recorded by the vertically oriented geophone below depths of ten feet because it is parallel with the hammer hit direction. Shallower than ten feet, waves arrived on the vertically and horizontally oriented geophones at about the same time.

- Horizontal shear waves were initially generated with horizontal hits on the ends of a beam lying on the ground at the same distance of 10 feet from the boring. Hammer hits on the opposite ends of the beam produce shear waves with opposite polarity. High amplitude noise from the 605 freeway and processing plants prevented the identification of any coherent seismic waves on both horizontal geophones. So vertically oriented shear wave arrivals were identified from the vertical hammer hits as much larger amplitude and longer time period waves that follow the compressional wave.

Data quality was continuously monitored during data collection to minimize the detrimental effects of the background vibrational noise (vehicle traffic). Wave arrivals were judged to be good to excellent.

Both wave arrivals were identified on the data records and their travel time from the seismic source measured with an estimated uncertainty of ±0.0001 seconds. The slant travel paths from source to the geophones were calculated within ±0.1 feet. The raw travel times along the slant path were converted to the equivalent times for a vertical travel path so a vertically incident profile can be presented. The data were plotted during acquisition to ensure reliable identification of the shear waves. The data are shown as small symbols (red for compressional and black for shear wave arrivals) in the left graph of Figure 5.

Regression lines fitted along these data points were interpreted to represent subsurface velocity zones. The inverse slopes of these lines (seismic velocities) were rounded to the nearest ten feet/second. The velocity uncertainty range was estimated by the regression line variance caused by ›ata scatter. Both velocities are shown in the left graph of Figures 5. Stratigraphy descriptions and uncorrected blow counts (provided by Mr. Jon Irvine) are shown in the right graphs.

3.3 Surface Seismic Reflection Survey Procedures

.ne following recording parameters empirically determined at many sites with similar geologic settings were used.

- o Record length of 0.512 seconds with 0.0001 second sampling interval provided at least 250 feet of penetration and adequate timing resolution of high-frequency (as high as 100 Hz) signals.
- o A 60 Hz notch filter minimized interference from electrical power lines.
- o Geophone array of 24 single geophones (40 Hz) spaced 17 feet apart provided high spatial resolution of the subsurface reflections.
- o Near and far offsets of 102 and 493 feet from the weight drop provided an estimated penetration of 250 without causing a wide-angle reflection problem.
- o Seismic energy generated at shot points located at every geophone should yield twelve-fold data and improve the signal to noise ratio.

Data recording times were carefully selected to avoid introducing the constant airplane and vehicle traffic vibration noise into the data. There were four rock processing plants near the survey area; nothing could be done to minimize the processing plant noise. We found they were operating everyday between 5:00 and 18:30.

Standard seismic refraction data were recorded along each geophone spread on both profiles in hopes ᶠ empirically measuring subsurface velocities. Refraction analysis with the "SIPT" software version 2.3 ,.Rimrock Geophysics, 1997) showed that no more than 3-4 adjacent data points formed roughly straight line segments that are needed to calculate velocities. This problem was probably caused by the irregular material composition and compaction of the backfill. Subsurface velocity zones were measured successfully in Boring B4 (Section 3.2) and used in the following reflection processing steps. Reflection processing used the "WINSEIS" software (2004).

- o Import Records- Reads raw data files in the seismograph format (SEG-2) and converts them to the "WINSEIS" internal storage format.
- o Trace Edit- Eliminates noisy traces from each shot record.
- o Filter- Digital bandpass filter with frequency corners at 50, 70, 150, and 200 Hz reduces the amplitude of the ground roll (less than 50 Hz) and wind (greater than 200 Hz) noise.
- o Gain- Applies a time-varying gain function with a 0.2 second window to each trace to remove variations in source and geophone coupling.
- o First Arrival Mute- Reduces refraction and wide-angle reflection energy that would otherwise interfere with the subsurface reflections after CMP stack.
- o CMP Sort- Sorts individual data traces from each raw record into Common Midpoint Panels (CMP) where each trace has the same subsurface reflection point.
- o Datum Statics- Adjusts each trace in time to a flat datum of 400 feet using the source and geophone elevations and an average velocity (2,050 feet/second) for the near-surface material.
- o Normal Moveout- Adjusts each individual trace in time so the shot point and geophone are coincident, thus mathematically representing the wave with vertical incidence.

First arrivals with excessive stretch were muted to avoid stacking the refracted energy in with the reflections.

- o Filter- Digital bandpass filter with frequency corners at 30, 50, 90, and 120 Hz reduces normal move out stretch and further reduces the amplitude of the ground roll and wind noise.
- o Surface Statics- Adjusts time zero by the same amount of time for each trace within common-shot and common-receiver gathers (groups) to remove variations in timing caused by variations in source and receiver ground coupling.
- o Residual Statics- Adjust time zero by a limited amount independently for each trace to remove variations in timing which are not surface-consistent.
- o CMP Stack- Calculates a composite trace from all adjusted traces within each CMP.
- o Gain- Applies a time-varying gain function with a 0.1 second window to remove effects of inconsistent stack power.

The final CMP stacked traces were plotted versus their positions along the profile with a high-resolution printer to enhance high-frequency signals (Figures 6 and 7). These sections show distance along the profile (±5% uncertainty) on the horizontal axis at a scale of 1 inch = 80 feet. The sections' left vertical axis shows two-way, seismic wave travel time (±5% uncertainty). The elevation scale was calculated by multiplying the time scale by a two layer velocity model. The right side elevation axis scale is 1 inch is about 42 feet above elevation 350 feet where the average borehole velocity was 2050 feet/second. Below elevation of 350 feet, the scale is 1 inch is about 92 feet where the average borehole velocity was 5000 feet/second. The elevation axis uncertainty is estimated to be ±20% because a two layer average velocity model was used and the initial refraction analysis showed large lateral velocity changes.

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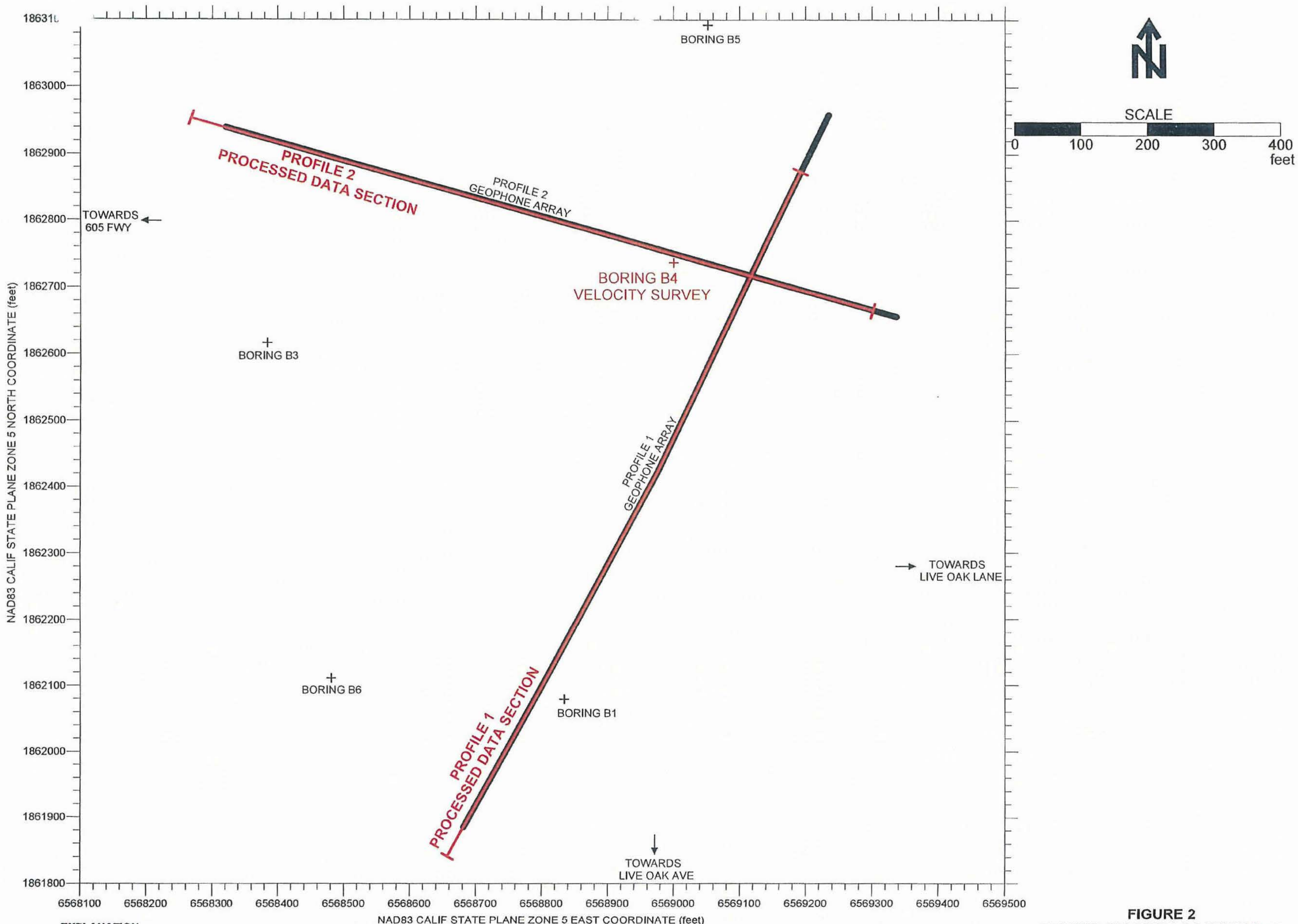


EXPLANATION

DATA TRACES IN THE PROCESSED SECTIONS ARE PLOTTED MIDWAY BETWEEN SEISMIC SOURCE AND GEOPHONE LOCATIONS WHICH CAUSES THE OFFSET BETWEEN THE RED AND BLACK LINES. THE RED LINES SHOULD BE USED TO RELATE INTERPRETED BOUNDARIES ON THE PROCESSED SECTIONS TO OTHER SITE FEATURES.

FIGURE 2
SEISMIC SURVEY LOCATION MAP

NU-WAY LANDFILL - IRWINDALE

TERRA PHYSICS

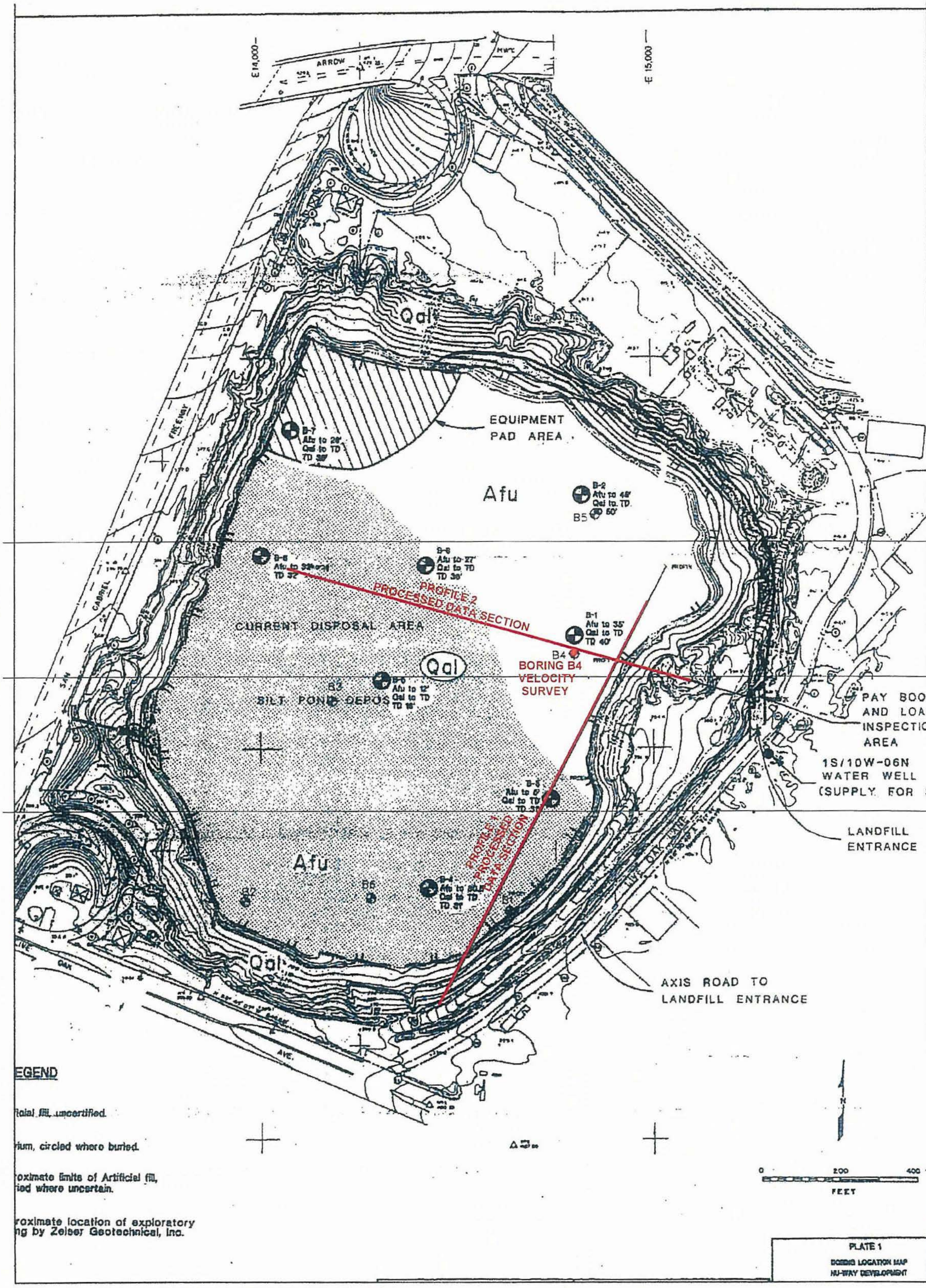
0916.0478

REFERENCE: MARCH 2007 TOPOGRAPHY MAP SUPPLED BY MR. JON IRVINE.

FIGURE 3
CURRENT SITE TOPOGRAPHY MAP SHOWING LOCATIONS OF PROCESSED REFLECTION SECTIONS

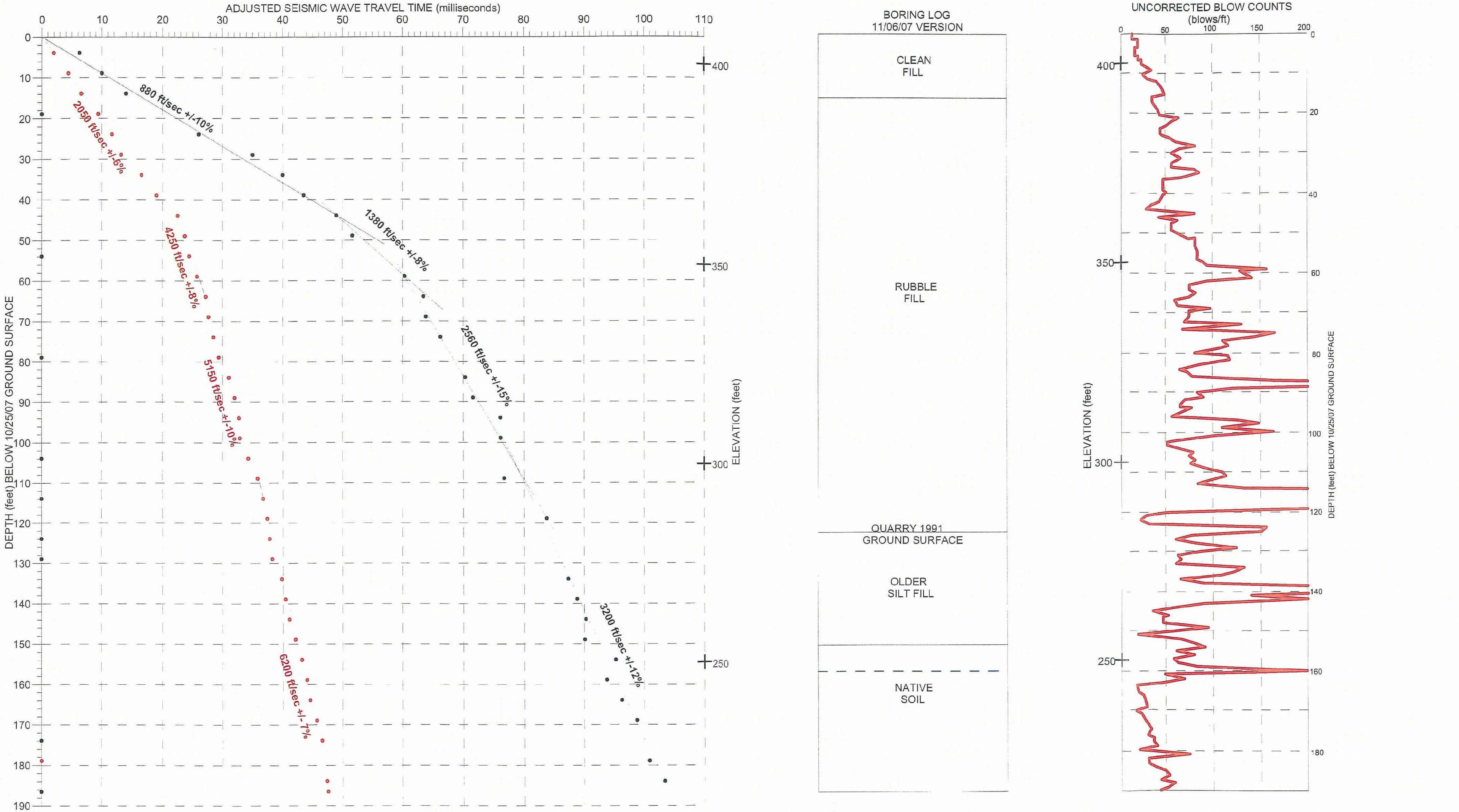
NU-WAY LANDFILL, IRWINDALE

TERRA PHYSICS



REFERENCE: ZEISER GEOTECHNICAL, INCORPORATED 1991 MAP

FIGURE 4
1991 HISTORICAL TOPOGRAPHY MAP SHOWING
LOCATIONS OF PROCESSED REFLECTION SECTIONS
NU-WAY LANDFILL, IRWINDALE
TERRA PHYSICS



EXPLANATION

DATA RECORDED ON OCTOBER 25, 2007 WHEN THE GROUND SURFACE ELEVATION = 407 feet. MEASUREMENT DEPTHS WERE RELATIVE TO THE GROUND SURFACE. SEISMIC WAVES WERE GENERATED 10.0 feet FROM BORING. ORIGINAL TRAVEL TIMES WERE ALONG SLANT DISTANCE PATHS FROM SEISMIC SOURCE TO BOREHOLE RECEIVER DEPTHS. THESE TIMES WERE ADJUSTED TO VERTICAL INCIDENT BY MULTIPLYING THE ORGINAL TIMES BY THE RATIO OF MEASUREMENT DEPTH DIVIDED BY SLANT DISTANCE.

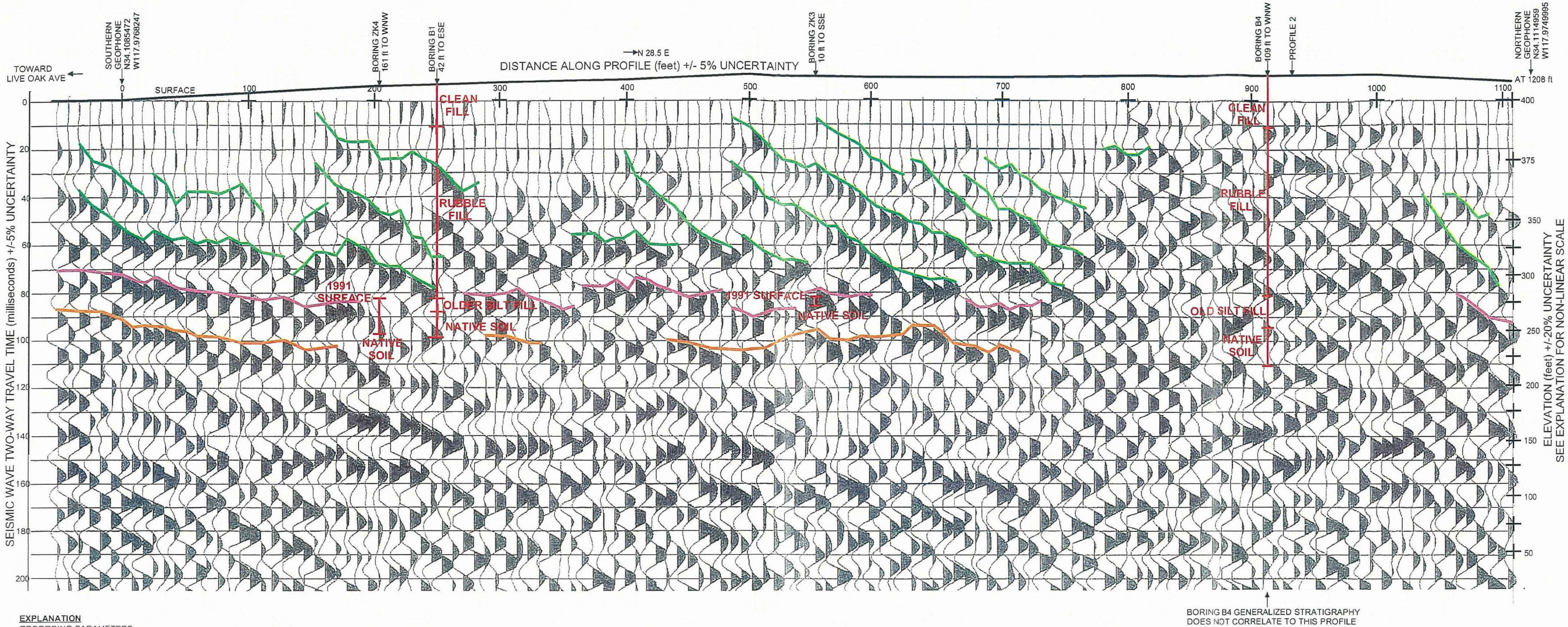
- COMPRESSIONAL WAVE DATA
- SHEAR WAVE DATA

INTERPRETED SEISMIC VELOCITY (feet/second) WITH ESTIMATED UNCERTAINTY.

FIGURE 5
BORING B4
DOWNHOLE VELOCITY SURVEY DATA
AND INTERPRETED VELOCITY MODEL
WITH STRATIGRAPHY CORRELATIONS

NU-WAY LANDFILL, IRWINDALE

TERRA PHYSICS



EXPLANATION

RECORDING PARAMETERS:
SEISMIC WAVES WITH FREQUENCIES AS HIGH AS 100 Hz WERE GENERATED BY STRIKING A 500 pound WEIGHT ON A 2.5 X 2.5 feet METAL PLATE LYING ON THE GROUND. DATA QUALITY WAS IMPROVED BY STACKING 12-15 INDIVIDUAL WEIGHT DROPS FOR EACH DATA RECORD. SEISMIC WAVES WERE SENSED BY 24 GEOPHONES (SINGLE, 40 Hz, CRITICALLY DAMPED) SPACED 17 feet APART. DISTANCE BETWEEN WEIGHT DROP AND NEAREST GEOPHONE WAS 102 feet. DATA RECORDED WITH A 60 Hz NOTCH FILTER.

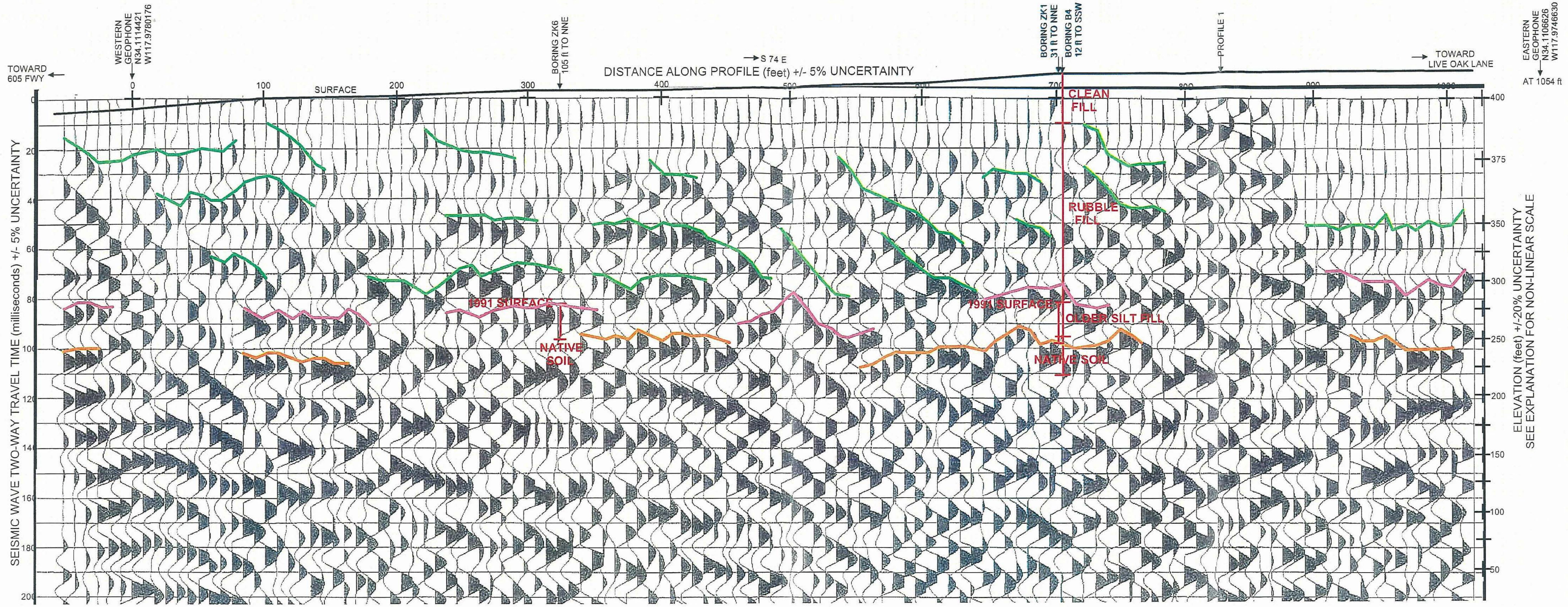
PROCESSING PARAMETERS:
PROCESSING DETAILS ARE DESCRIBED IN REPORT SECTION 3.3. SURFACE DATUM IS 400 feet. BORING B4 VELOCITY SURVEY (FIGURE 5) DETECTED A RELATIVELY LOW VELOCITY (2050 feet/second) FOR THE SURFICIAL DRY, CLEAN FILL. THE THREE UNDERLYING ZONES HAVE MUCH HIGHER VELOCITIES (AVERAGE IS 5000 feet/second) IN THE RUBBLE FILL. THE LARGE VELOCITY CONTRAST FORCED THE PROCESSING TO USE A TWO VELOCITY MODEL. THIS SAME MODEL WAS USED TO CONVERT THE LEFT VERTICAL TRAVEL TIME AXIS TO ELEVATION.
HORIZONTAL SCALE: 1 inch TO 80 feet.
RIGHT VERTICAL ELEVATION SCALE ABOVE ELEVATION OF 350 feet: I inch TO ABOUT 42 feet
BELOW ELEVATION OF 350 feet: I inch TO ABOUT 92 feet

INTERPRETED RESULTS:
REFLECTIONS CAUSED BY SEISMIC VELOCITY/MATERIAL DENSITY CHANGES.
- GREEN BOUNDARIES MAY REPRESENT MATERIAL/COMPACTION CHANGES IN THE RUBBLE FILL.
- PURPLE BOUNDARY MAY REPRESENT THE QUARRY 1991 GROUND SURFACE AND TOP OF OLD SILT FILL.
- ORANGE BOUNDARY MAY REPRESENT TOP OF THE NATIVE SOIL.

FIGURE 6
REFLECTION PROFILE 1
SUBSURFACE MATERIAL BOUNDARIES INTERPRETED FROM THE PROCESSED DATA SECTION

NU-WAY LANDFILL - IRWINDALE
TERRA PHYSICS

0916.0483



EXPLANATION

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PROCESSING PARAMETERS:
PROCESSING DETAILS ARE DESCRIBED IN REPORT SECTION 3.3. SURFACE DATUM IS 400 feet. BORING B4 VELOCITY SURVEY (FIGURE 5) DETECTED A RELATIVELY LOW VELOCITY (2050 feet/second) FOR THE SURFICIAL DRY, CLEAN FILL. THE THREE UNDERLYING ZONES HAVE MUCH HIGHER VELOCITIES (AVERAGE IS 5000 feet/second) IN THE RUBBLE FILL. THE LARGE VELOCITY CONTRAST FORCED THE PROCESSING TO USE A TWO VELOCITY MODEL.
THIS SAME MODEL WAS USED TO CONVERT THE LEFT VERTICAL TRAVEL TIME AXIS TO ELEVATION.
HORIZONTAL SCALE: 1 inch TO 80 feet.
RIGHT VERTICAL ELEVATION SCALE ABOVE ELEVATION OF 350 feet: I inch TO ABOUT 42 feet
BELOW ELEVATION OF 350 feet: I inch TO ABOUT 92 feet

INTERPRETED RESULTS:
REFLECTIONS CAUSED BY SEISMIC VELOCITY/MATERIAL DENSITY CHANGES.
GREEN BOUNDARIES MAY REPRESENT MATERIAL/COMPACTION CHANGES IN THE RUBBLE FILL.
PURPLE BOUNDARY MAY REPRESENT THE QUARRY 1991 GROUND SURFACE AND TOP OF OLD SILT FILL.
ORANGE BOUNDARY MAY REPRESENT TOP OF THE NATIVE SOIL.

FIGURE 7
REFLECTION PROFILE 2
SUBSURFACE MATERIAL BOUNDARIES INTERPRETED
FROM THE PROCESSED DATA SECTION

NU-WAY LANDFILL - IRWINDALE

TERRA PHYSICS