APPENDIX E

Draft Engineering Geotechnical Report

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Geotechnical Engineering Report

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Manchester Cable Landing
Manchester, Mendocino County, California
March 7, 2019
Terracon Project No. NA185199

Prepared for:
BHC Rhodes
Overland Park, KS

Prepared by:
Terracon Consultants, Inc.
Lodi, California

Environmental ■ Facilities ■ Geotechnical ■ Materials

March 7, 2019

BHC Rhodes
7101 College Boulevard, Suite 400
Overland Park, KS 66210

Attn: Mr. Chris Schepmann
        P: (913) 663-1900
        E: chris.schepmann@ibhc.com

Re: Geotechnical Engineering Report
        Manchester Cable Landing
        16001 California Highway 1
        Manchester, Mendocino County, California
        Terracon Project No. NA185199

Dear Mr. Schepmann:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PNA185199 dated January 3, 2019. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of the proposed cable landing project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Patrick C. Dell, Senior Associate   Garret S.H. Hubbart, Principal
Geotechnical Engineer 2186   Geotechnical Engineer 2588
Geotechnical Department Manager   Office Manager

Ryan L. Coe, E.G. 2705
Senior Staff Geologist

Environmental   ■   Facilities   ■   Geotechnical   ■   Materials

REPORT TOPICS

Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the *GeoReport* logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES
SITE LOCATION AND EXPLORATION PLANS
EXPLORATION RESULTS
SUPPORTING INFORMATION

Note: Refer to each individual Attachment for a listing of contents.

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REPORT SUMMARY

Topic [1]	Overview Statement [2]
Project Description	The project includes the construction of an underground/underwater cable system installed using horizontal directional drilling techniques. The cable system will "daylight" in the ocean floor approximately 3,000 feet from the shoreline. According to the information provided by your office, an excavation pit for the horizontal directional drilling equipment will be constructed at a depth of approximately 50 feet below the existing ground surface.
Geotechnical Characterization	Upper 15 feet of soil consisted of loose to medium dense well graded and poorly graded sand with silt. Medium dense silty sand, dense to very dense well graded sand with gravel, silty and clayey sand, and poorly graded sand with clay extended to a depth of about 50 feet. This material from about 15 feet to 50 feet consisted of completely weathered extremely fractured graywacke. Below the completely weathered graywacke, interbedded layers of highly weathered shale and highly weathered extremely fractured (very close fracture spacings) graywacke extended to a depth of 95 feet. From a depth of 95 feet to the maximum depth explored of 226½ feet, the material consisted of strong to very strong unweathered extremely fractured (very close fracture spacings) graywacke rock with abundant clay gouge. Perched groundwater was encountered at a depth of 5 feet below the existing ground surface during our drilling operations.
Earthwork	Excavations for the excavation pit will require heavy excavating equipment due to the very dense nature of the weathered graywacke encountered starting at a depth of about 20 feet below the existing ground surface. A braced or supported dewatered excavation will be required to construct the excavation pit structure.
Below-Grade Structure	We understand that an excavation pit for the horizontal directional equipment will be constructed at a depth of approximately 50 feet below the existing ground surface.
Pavements	We understand only a graveled access road will be constructed for this project.
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics above can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

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INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed cable landing underground excavation pit to be located at 16001 California Highway 1 in Manchester, Mendocino County, California. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil and rock conditions
- Groundwater conditions
- Site preparation and earthwork
- Excavation considerations
- Seismic site classification per 2016 CBC

- Foundation design and construction
- Floor slab design and construction
- Lateral earth pressures
- Pavement design and construction

The geotechnical engineering Scope of Services for this project included the advancement of one test boring to a depth of approximately 226½ feet below existing site grade. In addition, four geophysical surveys were completed along the proposed alignment of the cable.

Maps showing the site and boring locations and the geophysical survey lines are shown in the **Site Location** and **Exploration Plan** sections, respectively. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring log and/or as separate graphs in the **Exploration Results** section.

SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The site is located at 16001 California Highway 1, about 3 miles north of Manchester, California in Mendocino County. The approximate coordinates of the site are 39.014494°N and 123.6897111°W. See **Site Location**

Item	Description
Existing Site Features	Coastal bluff, beach, and rock outcrops.
Current Ground Cover	Bare ground with native grasses and beach.
Existing Topography	The site is situated along a coastal bluff that is about 160 to 170 feet above the mean sea level (MSL), based on Google Earth™. The coastal bluff is relatively flat then dips steeply to the beach at about 30 to 40 degrees with areas of near vertical outcrops.

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	As part of the Request for Proposal (RFP) for this project, we were provided by BHC Rhodes with the proposed boring locations and a description of the project.
Project Description	The project includes the construction of an underground/underwater cable system installed using horizontal directional drilling (HDD) techniques. The cable system will "daylight" in the ocean floor approximately 3,000 feet from the shoreline.
Below-Grade Structures	An excavation pit will be excavated at the proposed boring location to allow HDD equipment to install the cables. We understand the excavation pit will be constructed at a depth of approximately 50 feet below the existing ground surface.
Pavements	Pavements will be constructed to allow access to construction and maintenance equipment.

GEOTECHNICAL CHARACTERIZATION

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project.

The upper 15 feet consisted of loose to medium dense well graded and poorly graded sand with silt. Medium dense silty sand, dense to very dense well graded sand with gravel, silty and clayey sand, and poorly graded sand with clay extended to a depth of about 50 feet. This material from about 15 feet to 50 feet consisted of completely weathered extremely fractured graywacke. Below the completely weathered graywacke, interbedded layers of highly weathered shale and highly weathered extremely fractured (very close fracture spacings) graywacke extended to a depth of

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95 feet. From a depth of 95 feet to the maximum depth explored of 226½ feet, the material consisted of strong to very strong unweathered extremely fractured (very close fracture spacings) graywacke rock with abundant clay gouge.

Perched groundwater was encountered at a depth of about 5 feet below the existing ground surface during our field exploration. It appears that perched groundwater was present in the upper 20 feet. The soil/rock samples obtained from 20 to 50 feet did not appear saturated. At a depth of 50 feet bgs, we switched our drilling method to rock coring and mud rotary, both of which used water or drilling fluid to advance the sampler. Therefore, the depth to actual groundwater could not be accurately determined due to the drilling methods used to advance the boring below the depth of 50 feet.

Site Geology

The site is situated within the Coast Range Geomorphic Province (Coast Range) in Northern California. The mountain ranges and valleys within this region trend northwest and subparallel to the strike of the San Andreas Fault. The Coast Range is generally composed of Cenozoic and Mesozoic sedimentary strata and rocks of the Franciscan Complex[1]. Strata generally dip to the east beneath the alluvium of the Great Valley Geomorphic Province. The Pacific Ocean is west of the Coast Range. The northern coastal portion of the Coast Range is dominated by Tertiary-Cretaceous marine sedimentary and metasedimentary rocks[2].

The San Andreas Fault is a right-lateral strike slip fault that is more than 600 miles long and extends from Manchester to the Gulf of California. The northern San Andreas Fault strikes roughly northwest within the greater coast range mountains. In Mendocino County, it is delineated primarily by the Garcia River Valley. The San Andreas Fault is the major surface expression of the transform plate boundary between the Pacific and North American tectonic plates. The most recent large earthquakes on the San Andreas Fault were the Fort Tejon and San Francisco earthquakes of 1857 and 1906, respectively. The San Andreas Fault is capable of producing magnitude 7.5 to 8 earthquakes under multi-segment rupture scenarios. The section of the San Andreas Fault within the area of the site is documented as accommodating approximately 16 to 25 millimeters of creep annually[3].

The site lies just to the north of Manchester, California near the community of Irish Beach. The excavation pit is located on an approximately 170-foot marine terrace bluff approximately 300 yards from the Pacific Ocean. The San Andreas Fault lies approximately ½ mile to the west of the

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[1] California Geologic Survey, 2002, *Note 36: California Geomorphic Provinces*

[2] California Geologic Survey, 2010, *Geologic Map of California* (map)

[3] California Geologic Survey, 2018, Faulting in California (map)

site and plunges into the Pacific Ocean approximately one-mile south of the site at the mouth of Alder Creek. Multiple fault splays are present within the San Andreas Fault Zone in the Alder Creek area[4].   An exhibit of the San Andreas Fault Zone is located in the attached Supporting Information and shows the portion of the San Andreas Fault which is located near the site. The surficial geology west of the San Andreas Fault is mapped as Tertiary Marine Sediments and Quaternary alluvial and eolian deposits. According to boring logs from several oil and gas wells conducted near Point Arena, subsurface conditions consisted of predominately shale, siltstone, and sands[5] to a depth drilled of 7,780 feet in Sun Well #1-A (API: 0404500005).

Jennings (1960) mapped the surficial geology in the area of the site. Another map by Wagner (1982) covers the area just to the south of the site. The surficial geology at the site is mapped as Pleistocene marine and marine terrace deposits (Qm)[6]. The surrounding geology at the site is mapped as Undivided Cretaceous marine rock (K)[3] and can be interpreted to underly the site. Interpretation from Wagner's map suggest the site could be underlain by rocks of the Franciscan Formation (KJf) consisting of sandstone, shale, conglomerate, greenstone, and metagraywacke[7].

Rocks of the Franciscan Formation frequently occur in the form of a mélange, or mixture, of various rock types. This mélange typically contains larger intact rock blocks within a finer grained, and typically weaker, matrix material. Block sizes can range from gravel and sand-sized particles to particles that are hundreds of meters in diameter[8]. These systems form in the accretionary wedge that forms during subduction of tectonic plates.

The material encountered in our boring is consistent with the mapped geology in the area; however, the subsurface conditions at the site, and within the region of the site, are highly chaotic. The chaotic nature of the geology is due to the depositional environment of the marine sediments that constitute much of the Coast Range, the tectonic history of the site, and the severe faulting that has occurred in the area. A geologic cross-section is provided in the **Site Location and Exploration Plans Section.** The general conditions we anticipate for both the excavation pit and the proposed horizontal direction drill (HDD) alignment consist of a relatively thin mantle of marine terrace deposits and alluvium underlain by marine sedimentary bedrock. Our geologic cross section provided is only intended as a general guide to the potential subsurface conditions based on our interpretation of the mapped geology in the area, and our field investigation.

[4] California Geologic Survey, 1974, Special Studies Zone: Mallo Pass Creek Quadrangle (map)

[5] Division of Oil and Gas and Geothermal Resources (DOGGR), Well APIs: 0404500002, 0404500003, 0404500005

[6] Jennings, C.W., and Strand, R.G., 1960, Geologic map of California : Ukiah sheet: California Division of Mines and Geology, , scale 1:250,000

[7] Wagner, D.L., and Bortugno, E.J., 1982, Geologic map of the Santa Rosa quadrangle, California, 1:250,000: California Division of Mines and Geology, Regional Geologic Map 2A, scale 1:250,000

[8] Medley, E.W., 1994, "The engineering characterization of mélanges and similar block in matrix rocks (bimrocks)"

The summary of the geophysical surveys is provided in the **Supporting Information Section.**

GEOTECHNICAL OVERVIEW

The upper 20 feet of soil encountered in our boring consisted of loose to medium dense sand with varying amounts of silt and gravel. From a depth of about 20 feet to 50 feet, dense to very dense completely weathered bedrock consisting of sand with varying amounts of silt and clay were encountered. If the excavation pit is constructed at a depth of bout 50 feet below the existing ground surface, it will likely bear on highly weathered to completely weathered extremely fractured bedrock. Below a depth of about 20 feet, excavating the bedrock will require heavy equipment capable of dealing with the extremely fractured weathered bedrock. The sides of the excavation will need to be braced or supported in some fashion to prevent sloughing of the sandy materials. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

We understand a graveled access road will be construct for this project. The **Pavements** section addresses the design of pavement systems.

The **General Comments** section provides an understanding of the report limitations.

EARTHWORK

Earthwork is anticipated to include clearing and grubbing, excavations, and fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for construction of the excavation pit and access road.

Site Preparation

Prior to placing fill, existing vegetation and root mat should be removed. Complete stripping of the topsoil should be performed in the proposed construction areas.

The subgrade should be proofrolled with an adequately loaded vehicle such as a fully-loaded tandem-axle dump truck. The proofrolling should be performed under the direction of the Geotechnical Engineer. Areas excessively deflecting under the proofroll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas should either be removed or modified by stabilizing with lime/cement or aggregate base with geogrids. Excessively wet or dry material should either be removed, or moisture conditioned and recompacted.

Excavation Construction

It is anticipated that excavations for the proposed construction can be accomplished with conventional earth excavating equipment within the upper 20 feet. Excavations and HDD operations penetrating the dense to very dense weathered bedrock may require specialized heavy-duty excavating equipment or specialized drilling tools to facilitate break up and removal. Consideration should be given to obtaining a unit price for difficult excavation and drilling in the contract documents.

Due to the presence of loose to medium dense sand present in the upper 20 feet, caving of the excavation should be anticipated. Therefore, shoring and/or lay back of the excavation may be utilized if caving occurs.

The individual contractor(s) is responsible for designing and constructing stable, temporary excavations as required to maintain stability of both the excavation sides and bottom. Excavations should be sloped or shored in the interest of safety following local, and federal regulations, including current OSHA excavation and trench safety standards.

The subsurface soils within the upper 20 feet when dewatered and free from perched water can be considered Type C soils when applying the OSHA regulations. OSHA allows a maximum slope inclination of 1½H:1V for Type C soils in excavations of 20 feet or less.

Flatter slopes may be required if caving soils or seepage is encountered in any excavation. For excavations extending to a depth of more than 20 feet, it will be necessary to have the side slopes designed by a professional engineer.

Soils from the excavation should not be stockpiled higher than 6 feet or within 20 feet of the edge of an open trench or the sides of the excavation. Construction of open cuts adjacent to existing structures, including underground pipes, is not recommended within a 1½ H:1V plane extending beyond and down from the perimeter of the structure. Cuts that are proposed within five 5 feet of other utilities, underground structures, and pavement should be provided with temporary shoring.

It may be necessary for the contractor to retain a geotechnical engineer to monitor the soils exposed in all excavations and provide engineering services for slopes. This will provide an opportunity to monitor the soils encountered and to modify the excavation slopes as necessary. It also offers an opportunity to verify the stability of the excavation slopes during construction.

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Fill Material Types

All fill materials should be inorganic soils free of vegetation, debris, and fragments larger than three inches in size. Pea gravel or other similar non-cementitious, poorly-graded materials should not be used as fill or backfill without the prior approval of the geotechnical engineer.

Imported earth materials for use as engineered fill should be pre-approved by our representative prior to construction. Imported non-expansive soils may be used as fill material for the following:

- general site grading
- foundation areas
- slab-on-grade floor
- pavement subgrade
- foundation backfill
- trench backfill
- exterior slabs-on-grade

Soils for use as compacted engineered fill material within the proposed building/equipment pad area should conform to non-expansive materials as indicated in the following recommendations:

Gradation	**Percent Finer by Weight** **(ASTM C 136)**
3"	100
No. 4 Sieve	50 - 100
No. 200 Sieve	15 - 50
Liquid Limit	30 (max)
Plasticity Index	10 (max)
Maximum Expansive Index*	20 (max)

*ASTM D 4829

We note that we performed a Plasticity Index Test on a sample of the near surface soil which produced a Plasticity Index of 12. Therefore, additional testing may be required to determine if the near surface sandy soils will meet the above specifications. Engineered fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift. Fill lifts should not exceed ten inches in loose thickness.

Fill Compaction Requirements

Recommended compaction and moisture content criteria for engineered fill materials are as follows:

Material Type and Location	Per the Modified Proctor Test (ASTM D 1557)		
	Minimum Compaction Requirement (%)	Range of Moisture Contents for Compaction Above Optimum	
		Minimum	Maximum
On-site sandy soils and Low volume change (non-expansive) imported fill:			
Excavation backfill greater than 5 feet in depth	95	+1%	+3%
Excavation backfill less than 5 feet in depth	90	+1%	+3%
Miscellaneous backfill:	90	0%	+3%
Pavement subgrade beneath aggregate base*:	95	0%	+3%
Aggregate base beneath pavement:	95	0%	+3%
Utility Trenches*:	90	0%	+4%
Bottom of native soil excavation receiving fill:	90	+2%	+4%

*The upper 12 inches beneath graveled access roads should be compacted to 95% of the maximum dry density as determined in the ASTM D1557 test method.

We recommend that compacted native soil or any engineered fill be tested for moisture content and relative compaction during placement. Should the results of the in-place density tests indicate the specified moisture content or compaction requirements have not been met, the area represented by the test should be reworked and retested as required until the specified moisture content and relative compaction requirements are achieved.

Grading and Drainage

All grades must provide effective drainage away from the excavation pit during and after construction and should be maintained throughout the life of the development. Infiltration of water into the excavations should be prevented during construction. Backfill in excavations should be well compacted and free of all construction debris to reduce the potential of moisture infiltration.

Dewatering

During the design phase of the project, additional evaluation of perched water/groundwater and fluctuations in these levels should be performed. Depending on the depth of excavation and seasonal conditions, perched water/groundwater will likely be encountered within the excavations planned on the site.

Excavations that extend below groundwater will involve construction dewatering to maintain excavations in a relatively dry condition. Pumping from sumps may be utilized to control water within excavations. Well points may be required for significant groundwater flow, or where

excavations penetrate groundwater to a significant depth. Excavation contractors are responsible for dewatering the planned excavations.

Excavations and structures that extend below groundwater, including cables, vaults, and manholes, should be designed to resist hydrostatic uplift pressures due to groundwater and would involve waterproofing, as appropriate. Terracon should be notified if excavations are planned to extend below the groundwater levels to verify the stability of the bottom of the planned excavations have an adequate factor of safety.

Earthwork Construction Considerations

It is anticipated that excavations for the proposed construction can be accomplished with conventional earth excavating equipment within the upper 20 feet. Excavations and HDD operations penetrating the dense to very dense weathered bedrock may require specialized heavy-duty excavating equipment or specialized drilling tools to facilitate break up and removal. Consideration should be given to obtaining a unit price for difficult excavation and drilling in the contract documents.

Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of pavement. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to pavement construction.

The perched groundwater table could affect overexcavation efforts, especially for over-excavation and replacement of lower strength soils. A temporary dewatering system consisting of sumps with pumps could be necessary to achieve the recommended depth of over-excavation.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local, and/or state regulations.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically October through April) it may be necessary to take extra precautionary measures to protect subgrade soils.

Wet season earthwork may require additional mitigation measures beyond that which would be expected during the drier summer and fall months. This could include diversion of surface runoff around exposed soils and draining of ponded water on the site. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic.

Earthwork on the project should be observed and evaluated by Terracon. The evaluation of earthwork should include observation and testing of engineered fill, subgrade preparation, and other geotechnical conditions exposed during the construction of the project.

Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the Geotechnical Engineer. Monitoring should include documentation of adequate removal of vegetation and topsoil, proofrolling, and mitigation of areas delineated by the proofroll to require mitigation.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, until approved by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 12-inch thick lift of compacted fill around the below grade structure and every 5,000 square feet in pavement areas. One density and water content test should be performed for 12-inch thick lift for every 50 linear feet of compacted utility trench backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated under the direction of the Geotechnical Engineer. If unanticipated conditions are encountered, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

SEISMIC CONSIDERATIONS

The seismic design requirements for the project are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7-10.

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Description	Value
2016 California Building Code Site Classification (CBC) [1]	C [2]
Site Latitude	39.0145° N
Site Longitude	123.6897° W
S_s **Spectral Acceleration for a Short Period**	2.524g
S_1 **Spectral Acceleration for a 1-Second Period**	1.213g
F_a **Site Coefficient for a Short Period**	1.000
F_v **Site Coefficient for a 1-Second Period**	1.300
S_{Ms} **Maximum Considered Spectral Response Acceleration for a Short Period**	2.524g
S_{M1} **Maximum Considered Spectral Response Acceleration for a 1-Second Period**	1.577g
S_{DS} **Design Spectral Acceleration for a Short Period** [2]	1.683g
S_{D1} **Spectral Acceleration for a 1-Second Period** [2]	1.051g
PGA_M **Peak Ground Acceleration**	0.969g

1. Seismic site classification in general accordance with the *2016 California Building Code*, which refers to ASCE 7-10 with March 2013 errata. The site classification was determined from the MASW geophysical survey which extended to depths of about 90 feet.
2. These values were obtained using online seismic design maps and tools provided by the USGS (http://earthquake.usgs.gov/hazards/designmaps/).

LATERAL EARTH PRESSURES

Design Parameters

For vertical cuts, steeper temporary construction slopes, or unstable soil encountered during the excavation, shoring should be provided by the contractor as necessary, to protect the workers in the excavation. The shoring for the excavations may be cantilevered or may be laterally supported.

For onsite soils in the upper 20 feet and without any hydrostatic pressure: The recommended equivalent fluid pressures and coefficient of friction for foundation elements are presented in the table below. These values are based on an angle of internal friction of 32° and a dry unit weight of 120 pounds per cubic foot (pcf).

Item	On-site Granular Soils
Active Case	37 psf/ft
Passive Case	390 psf/ft
At-Rest Case	56 psf/ft

Item	On-site Granular Soils
Coefficient of friction	0.40

For onsite soils between depths of 20 and 50 feet and without any hydrostatic pressure: We recommended equivalent fluid pressures and coefficient of friction for foundation elements are presented in the following table. These values assume an angle of internal friction of 40° and a dry unit weight of 130 pcf.

Item	On-site Granular Soils
Active Case	28 psf/ft
Passive Case	600 psf/ft
At-Rest Case	46 psf/ft
Coefficient of friction	0.40

The lateral earth pressures herein do not include any factor of safety and are not applicable for submerged soils/hydrostatic loading. A safety factor of 2.0 is considered adequate for allowable passive pressure values. Additional recommendations may be necessary if submerged conditions are to be included in the design.

For the design of braced shoring, we recommend such shoring be designed using a rectangular-shaped distribution of lateral earth pressure of 28H (in psf) (H is the total height of the braced excavation) for excavations between depths of 0 and 25 feet bgs and 24H for excavations between depths of 25 to 50 feet bgs. Terracon should be notified if the excavation is anticipated to be deeper than 50 feet. Sloping ground extending from the back of shoring should be considered as additional surcharge.

The depth of groundwater represents the levels encountered during exploration. The contractor should verify this depth prior to excavation. Terracon should be notified if the excavation is anticipated to be deeper than 50 feet.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads such as soil stockpiles, vehicular traffic, or construction equipment located adjacent to the shoring should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation. As previously mentioned, all shoring should be designed and installed in accordance with state and federal safety regulations.

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PAVEMENTS

General Pavement Comments

Pavement designs are provided for the traffic conditions and pavement life conditions as noted in **Project Description** and in the following sections of this report. A critical aspect of pavement performance is site preparation. Pavement designs noted in this section must be applied to the site which has been prepared as recommended in the **Earthwork** section.

We are providing a pavement section for the graveled access road based on our experience with similar projects and subgrade conditions.

We recommend that the graveled access road consist of 8 inches of compacted Class 2 aggregate base over 12 inches of compacted subgrade. All materials should meet the current Caltrans Standard Specifications, latest edition. If desired to improve the longevity of the graveled road, the surface of the aggregate base may be sealed with an asphalt oil for the first few years to help reduce the potential for moisture migration into the aggregate base.

Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

Pavement Maintenance

The pavement section presented above represents a minimum recommended thickness and, as such, periodic maintenance should be anticipated. Therefore, preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. We recommend that any depressions or ruts be repaired as quickly as possible to reduce the potential for premature pavement deterioration. We recommend stockpiling some aggregate base on site for such repairs. As a minimum and as part of the annual maintenance, depressions or ruts should be repaired prior to the start of the rainy season.

CORROSIVITY

The table below lists the results of laboratory soluble sulfate, soluble chloride, electrical resistivity, and pH testing. The values may be used to estimate potential corrosive characteristics of the on-

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site soils with respect to contact with the various underground materials which will be used for project construction.

Corrosivity Test Results Summary						
Boring	Sample Depth (feet)	Soil Description	Soluble Sulfate (ppm)	Soluble Chloride (ppm)	Electrical Resistivity (Ω-cm)	pH
B-1	1- 2½	Well Graded Sand with Silt	94	28	5044	8.06

The sulfate test results indicate that the soil from boring B-1 classifies as Class S0 according to Table 19.3.1.1 of ACI 318-14. This indicates that the sulfate level is negligible when considering corrosion to concrete.

The chloride test results indicate that the soils have a relatively low chloride content present. According to Table 19.3.1.1 of ACI 318-14, the soil should not be considered an external source of chloride (i.e. sea water, etc.) to concrete foundations. Consequently, chloride classes of C0 and C1 should be used where applicable. C0 is defined as, "Concrete dry or protected from moisture" and C1 is defined as, "Concrete exposed to moisture but not to an external source of chlorides". For the amount of chlorides allowed in concrete mix designs, Table 19.3.2.1 of ACI 318-14 shall be adhered to as appropriate.

Based on the results of the sulfate content test results, ACI 318-14, Section 19.3 does not specify the type of cement or a maximum water-cement ratio for concrete for sulfate Class S0. For further information, see ACI 318-14, Section 19.3.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of

pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

Number of Borings	Boring Depth (feet)	Planned Location
1	226½	Excavation pit

Boring Layout and Elevations: Terracon personnel provided the boring layout. Coordinates were obtained with a handheld GPS unit (estimated horizontal accuracy of about ±10 feet) and approximate elevations were obtained by interpolation Google Earth Pro™.

Subsurface Exploration Procedures: We advanced the boring with a track-mounted rotary drill rig using continuous hollow stem flight augers, ODEX air rotary casing, mud rotary, and rock coring methods. We used hollow stem augers from the ground surface to a depth of 50 feet bgs. At 50 feet bgs, we switched to rock coring that extended to a depth of 95 feet bgs. At 95 feet bgs, we switched to the mud rotary drilling method and continued with that method to the maximum depth drilled of 226½ feet bgs.

We obtained split-barrel samples at depths of 1 foot and 5 feet and at intervals of 5 feet thereafter to a depth of 50 feet bgs. We obtained a split-barrel samples starting again at 95 feet bgs and sampled every 5 feet to a depth of 115 feet bgs. At 120 feet we alternated sampling with the split-barrel sampler and obtaining a grab sample of the cuttings every 5 feet to the maximum depth explored.

In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. We observed and recorded groundwater levels during drilling and sampling. As required by the Mendocino County Environmental Health Department, we backfilled the boring with neat cement grout immediately upon completion of the boring.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring log by a Certified Engineering Geologist. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer and Certified Engineering Geologist. Our exploration team prepared the field boring log as part of the drilling operations. This field log included visual classifications of the materials encountered during drilling and our interpretation of the subsurface conditions between samples. The final boring log was prepared from the field log. The final boring log represents the Geotechnical

Engineer's and Certified Engineering Geologist's interpretation of the field log and include modifications based on observations and tests of the samples in our laboratory.

Geophysical Surveys: A description of the methods of the geophysical surveys is provided in the **Supporting Information Section.**

Laboratory Testing

The project engineer and engineering geologist reviewed the field data and assigned laboratory tests to understand the engineering properties of the various soil and rock strata, as necessary, for this project. Procedural standards noted below are for reference to methodology in general. In some cases, variations to methods were applied because of local practice or professional judgment. Standards noted below include reference to other, related standards. Such references are not necessarily applicable to describe the specific test performed.

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM C127 Standard Test Method for Relative Density (Specific Gravity) of Coarse Aggregate
- ASTM D7012 Standard Test Method for Unconfined Compressive Strength of Intact Rock
- ASTM D1140 Standard Test Method for Determining the Amount of Material Finer than No. 200 Sieve by Soil Washing
- Corrosivity Tests

The laboratory testing program included examination of soil samples by an engineer and geologist. Based on the material's texture and plasticity, we described and classified the soil samples in accordance with the Unified Soil Classification System.

SITE LOCATION AND EXPLORATION PLANS

Contents:

Site Location Plan
Exploration Plan
Site Location Map-Geophysical Survey
Geologic Cross Section A-A'

Note: All attachments are one page unless noted above.

SITE

N

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

N

B1

Boring Location

500 feet

EXPLORATION RESULTS

Contents:

Boring Log (B1)
Corrosivity
Direct Shear (2)

Note: All attachments are one page unless noted above.

PROJECT: Manchester Cable Landing Project

CLIENT: BHC Rhodes
Overland Park, KS

SITE: **State Route 1**
Manchester, CA

LOCATION See Exploration Plan

Latitude: 39.0145° Longitude: -123.6897°

Approximate Surface Elev.: 172 (Ft.) +/-

GRAPHIC LOG	DEPTH / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (Ft.)	FIELD TEST RESULTS	RQD	UNCONFINED COMPRESSIVE STRENGTH (psi)	LABORATORY HP (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	WELL GRADED SAND WITH SILT (SW), trace gravel, fine to medium grained, dark brown, very loose — 1.5 / 170.5+/-					3-3-3 N=6					25	40-28-12	
	WELL GRADED SAND WITH SILT (SW), trace gravel, fine to coarse grained, yellowish red, loose to medium dense — 5.0 / 167+/-	5	▽			8-5-7 N=12					23		
	POORLY GRADED SAND WITH SILT (SP), medium to coarse grained, yellowish red with gray, medium dense	10				7-7-8 N=15					24		
	— 15.0 / 157+/-	15				9-10-12 N=22					18		15
	SILTY SAND (SM), trace gravel, medium to coarse grained, bluish gray with brown, medium dense, completely weathered graywacke — 20.0 / 152+/-	20				6-13-16 N=29					17		
	WELL GRADED SAND WITH GRAVEL (SW), coarse grained, subrounded to rounded, bluish gray, medium dense, completely weathered graywacke — 21.5 / 150.5+/-												
	CLAYEY SAND (SC), trace gravel, medium to coarse grained, bluish gray with brown, dense to very dense, completely weathered graywacke	25				19-50/3"					9		
	— 30.0 / 142+/-	30				18-36-50/3"					13		11
	POORLY GRADED SAND WITH CLAY (SP-SC), medium to coarse grained, bluish gray, very dense, completely weathered graywacke	35				30-21-36 N=57					5		
		40				34-50/5"					5		
		45				31-43-50/3"					6		
	— 50.0 / 122+/-	50				13-16-26 N=42					6		

Stratification lines are approximate. In-situ, the transition may be gradual.
Pocket penetrometer readings taken in clay gouge and clay lenses in samples

Hammer Type: Automatic
Caving was not observed from 50' to 225'

Advancement Method:
  6" ODEX
  HQ Rock Core
  4" Mud Rotary

Abandonment Method:
  Boring backfilled with cement-bentonite grout upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by others.

Notes:

6" ODEX to 50'
Approximately 250 gal. water lost to formation througout drilling
GW appears perched in upper 5 feet
Switch to HQ rock core @ 50'

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Boring Started: 02-05-2019   Boring Completed: 02-08-2019

Drill Rig: D-90   Driller: Bill Bradberry

Project No.: NA185199

GEO SMART LOG-NO WELL NA185199 MANCHESTER CABLE GPJ MODELLAYER.GPJ 3/8/19

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT.

PROJECT: Manchester Cable Landing Project

CLIENT: BHC Rhodes
Overland Park, KS

SITE: **State Route 1**
Manchester, CA

LOCATION See Exploration Plan

Latitude: 39.0145° Longitude: -123.6897°

Approximate Surface Elev.: 172 (Ft.) +/-

DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	SAMPLE TYPE	RECOVERY (Ft.)	FIELD TEST RESULTS	RQD	UNCONFINED COMPRESSIVE STRENGTH (psi)	LABORATORY HP (tsf)	WATER CONTENT (%)
		55		0.6		0			
				1		0			
		60							
62.0	110+/-			3		0			
63.0 **SHALE**, bluish gray, highly weathered, very weak	109+/-	65							
GRAYWACKE, bluish gray, fine-grained, extremely fractured, very close fracture spacing, massive bedding, highly weathered, medium strong, abudant clay gouge				1.5		0			
70.0	102+/-	70							
71.0 **SHALE**, bluish gray, highly weathered, very weak	101+/-			2.5		0			
GRAYWACKE, bluish gray, fine-grained, extremely fractured, very close fracture spacing, massive bedding, highly weathered, medium strong, abudant clay gouge		75							
				2		0			
		80							
				2		0			
		85							
				2.5		15	3,786		
		90							
93.0	79+/-			1.5		0			
SHALE, bluish gray, highly weathered, very weak									
95.0	77+/-	95			31-50/3"			2.5 (HP)	15
GRAYWACKE, bluish gray, fine-grained, extremely fractured, very close fracture spacing, massive bedding, unweathered, strong to very strong rock, abudant clay gouge		100			50/4"				24

GRAYWACKE, bluish gray, fine-grained, extremely fractured, very close fracture spacing, massive bedding, highly weathered, medium strong, abudant clay gouge *(continued)*

Stratification lines are approximate. In-situ, the transition may be gradual.
Pocket penetrometer readings taken in clay gouge and clay lenses in samples

Hammer Type: Automatic
Caving was not observed from 50' to 225'

Advancement Method:
  6" ODEX
  HQ Rock Core
  4" Mud Rotary

Abandonment Method:
  Boring backfilled with cement-bentonite grout upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by others.

Notes:

Switch to Mud Rotary @ 95'

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
902 Industrial Way
Lodi, CA

Boring Started: 02-05-2019

Boring Completed: 02-08-2019

Drill Rig: D-90

Driller: Bill Bradberry

Project No.: NA185199

PROJECT: Manchester Cable Landing Project

CLIENT: BHC Rhodes
Overland Park, KS

SITE: State Route 1
Manchester, CA

LOCATION See Exploration Plan

Latitude: 39.0145° Longitude: -123.6897°

Approximate Surface Elev.: 172 (Ft.) +/-

GRAPHIC LOG	DEPTH / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (Ft.)	FIELD TEST RESULTS	RQD	UNCONFINED COMPRESSIVE STRENGTH (psi)	LABORATORY HP (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	GRAYWACKE, bluish gray, fine-grained, extremely fractured, very close fracture spacing, massive bedding, unweathered, strong to very strong rock, abudant clay gouge *(continued)*	105				50/3"					37		
		110											
		115				20-38-50/2"			2.5 (HP)		17		
		120											
		125				50/5"			2.0 (HP)		29		
		130											
		135				50/5"					22		
		140											
		145				50/1"					28		
		150											
		155				50/4"					14		

Stratification lines are approximate. In-situ, the transition may be gradual.
Pocket penetrometer readings taken in clay gouge and clay lenses in samples

Hammer Type: Automatic
Caving was not observed from 50' to 225'

Advancement Method:
6" ODEX
HQ Rock Core
4" Mud Rotary

Abandonment Method:
Boring backfilled with cement-bentonite grout upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by others.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
902 Industrial Way
Lodi, CA

Boring Started: 02-05-2019

Boring Completed: 02-08-2019

Drill Rig: D-90

Driller: Bill Bradberry

Project No.: NA185199

PROJECT: Manchester Cable Landing Project

CLIENT: BHC Rhodes
Overland Park, KS

SITE: State Route 1
Manchester, CA

LOCATION See Exploration Plan

Latitude: 39.0145° Longitude: -123.6897°

Approximate Surface Elev.: 172 (Ft.) +/-

DEPTH   ELEVATION (Ft.)

GRAYWACKE, bluish gray, fine-grained, extremely fractured, very close fracture spacing, massive bedding, unweathered, strong to very strong rock, abudant clay gouge *(continued)*

Depth (Ft.)	Field Test Results	Water Content (%)
160		30
165	50/1"	19
170		7
175	50/2"	12
180		14
185	50/2"	12
190		24
195	50/1"	
200		24
205	50/2"	16

Stratification lines are approximate. In-situ, the transition may be gradual.
Pocket penetrometer readings taken in clay gouge and clay lenses in samples

Hammer Type: Automatic
Caving was not observed from 50' to 225'

Advancement Method:
6" ODEX
HQ Rock Core
4" Mud Rotary

Abandonment Method:
Boring backfilled with cement-bentonite grout upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by others.

Notes:

WATER LEVEL OBSERVATIONS

▽ *While drilling*

Terracon
902 Industrial Way
Lodi, CA

Boring Started: 02-05-2019

Boring Completed: 02-08-2019

Drill Rig: D-90

Driller: Bill Bradberry

Project No.: NA185199

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL NA185199 MANCHESTER CABLE GPJ MODELLAYER.GPJ 3/8/19

PROJECT: Manchester Cable Landing Project

CLIENT: BHC Rhodes
Overland Park, KS

SITE: State Route 1
Manchester, CA

LOCATION See Exploration Plan

Latitude: 39.0145° Longitude: -123.6897°

Approximate Surface Elev.: 172 (Ft.) +/-

DEPTH   ELEVATION (Ft.)

GRAPHIC LOG	Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (Ft.)	FIELD TEST RESULTS	RQD	UNCONFINED COMPRESSIVE STRENGTH (psi)	LABORATORY HP (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	GRAYWACKE, bluish gray, fine-grained, extremely fractured, very close fracture spacing, massive bedding, unweathered, strong to very strong rock, abudant clay gouge *(continued)*	210									29		
		215				50/2"					9		
		220									26		
	225.0 -53+/-	225				50/1"					7		
	Boring Terminated at 226.5 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.
Pocket penetrometer readings taken in clay gouge and clay lenses in samples

Hammer Type: Automatic
Caving was not observed from 50' to 225'

Advancement Method:
6" ODEX
HQ Rock Core
4" Mud Rotary

Abandonment Method:
Boring backfilled with cement-bentonite grout upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were provided by others.

Notes:

WATER LEVEL OBSERVATIONS

▽   *While drilling*

Terracon
902 Industrial Way
Lodi, CA

Boring Started: 02-05-2019   Boring Completed: 02-08-2019

Drill Rig: D-90   Driller: Bill Bradberry

Project No.: NA185199

CHEMICAL LABORATORY TEST REPORT

Project Number: NA185199
Service Date: 02/26/19
Report Date: 02/27/19
Task:

Terracon

750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client

Manchester Cable Landing Project

Project

BHC Rhodes

Sample Submitted By: Terracon (NA)   **Date Received:** 2/25/2019   **Lab No.:** 19-0199

Results of Corrosion Analysis

Sample Number	
Sample Location	B-1-1
Sample Depth (ft.)	
pH Analysis, AWWA 4500 H	8.06
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	94
Sulfides, AWWA 4500-S D, (mg/kg)	Nil
Chlorides, ASTM D 512, (mg/kg)	28
Red-Ox, AWWA 2580, (mV)	+681
Total Salts, AWWA 2520 B, (mg/kg)	581
Resistivity, ASTM G 57, (ohm-cm)	5044
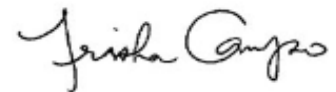
Analyzed By:

Trisha Campo
Chemist

DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS
ASTM D3080

R² = 0.9994

SHEAR STRENGTH

◆ MAXIMUM SHEAR STRESS

		FRICTION ANGLE		COHESION		NORMAL STRESS, psi	NORMAL STRESS, psi	NORMAL STRESS, psi
AT MAXIMUM SHEAR STRESS		32.6	deg	4.5	psi	10.38	20.86	41.70
INITIAL AREA, mm2	2026.8	INITIAL MOISTURE, %						
INITIAL LENGTH, mm	25.40	INITIAL DRY DENSITY, pcf						
SPECIFIC GRAVITY	2.70	INITIAL SATURATION, %						
SG TESTED		INITIAL VOID RATIO						
SG ASSUMED	X	FINAL MOISTURE, %						
LIQUID LIMIT		FINAL SATURATION, %						
PLASTIC LIMIT		FINAL VOID RATIO						
PLASTICITY INDEX		MAXIMUM SHEAR STRESS, psi				10.92	18.10	31.02
SAMPLE TYPE	RECOMPACTED	RATE OF LOADING, in/min				0.0030	0.0030	0.0030
DESCRIPTION	Poorly Graded Sand with Silt							

PROJECT NAME: Manchester Cable Landing

LOCATION: State Rt. 1

JOB NO.: NA185199

DATE: 3/4/2019

BORING NO. 1

SAMPLE NO. 3

DEPTH, feet 10

Terracon

Manchester Cable Landing
State Rt. 1
NA185199
3/4/2019

BORING NO.	1
SAMPLE NO.	3
DEPTH, feet	10

SHEAR STRESS

Legend: Result 1, Result 2, Result 3

Y-axis: SHEAR STRESS, psi (0.0 to 35.0)
X-axis: HORIZONTAL DISPLACEMENT, inch (0.000 to 0.300)

DISPLACEMENT

Legend: Result 1, Result 2, Result 3

Y-axis: VERTICAL DISPLACEMENT, in (-0.015 to 0.020)
X-axis: HORIZONTAL DISPLACEMENT, in (0.000 to 0.300)

Contraction

Dilation

Terracon

DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS
ASTM D3080

R² = 1

SHEAR STRENGTH

◆ MAXIMUM SHEAR STRESS

Y-axis: SHEAR STRESS, psi

X-axis: NORMAL STRESS, psi

		FRICTION ANGLE		**COHESION**		NORMAL STRESS, psi	NORMAL STRESS, psi	NORMAL STRESS, psi
AT MAXIMUM SHEAR STRESS		**41.1**	**deg**	**4.0**	**psi**	10.34	20.78	41.98
INITIAL AREA, mm2	2026.8	INITIAL MOISTURE, %						
INITIAL LENGTH, mm	25.40	INITIAL DRY DENSITY, pcf						
SPECIFIC GRAVITY	2.70	INITIAL SATURATION, %						
SG TESTED		INITIAL VOID RATIO						
SG ASSUMED	X	FINAL MOISTURE, %						
LIQUID LIMIT		FINAL SATURATION, %						
PLASTIC LIMIT		FINAL VOID RATIO						
PLASTICITY INDEX		MAXIMUM SHEAR STRESS, psi				13.00	22.13	40.57
SAMPLE TYPE	RECOMPACTED	RATE OF LOADING, in/min				0.0020	0.0020	0.0020
DESCRIPTION	Well Graded Sand with Gravel							

PROJECT NAME: Manchester Cable Landing BORING NO. 1
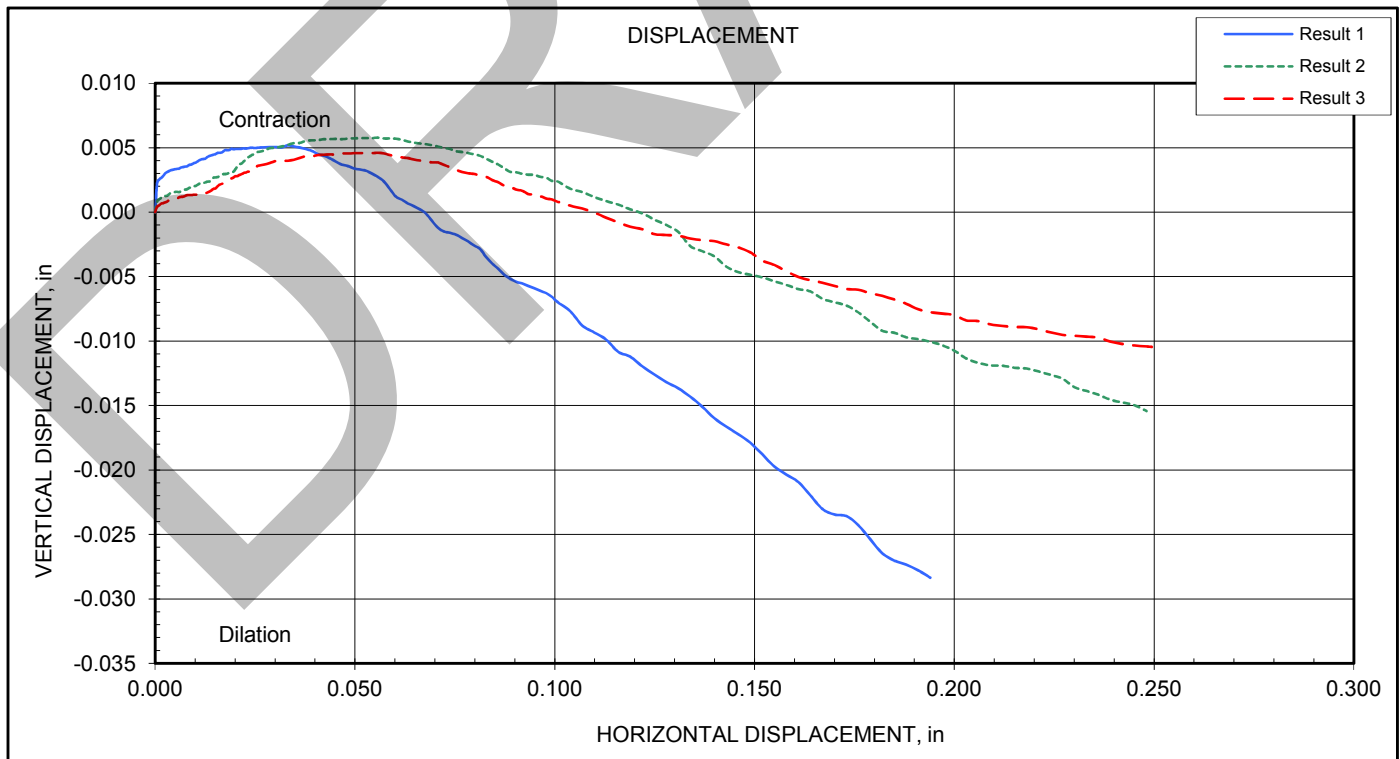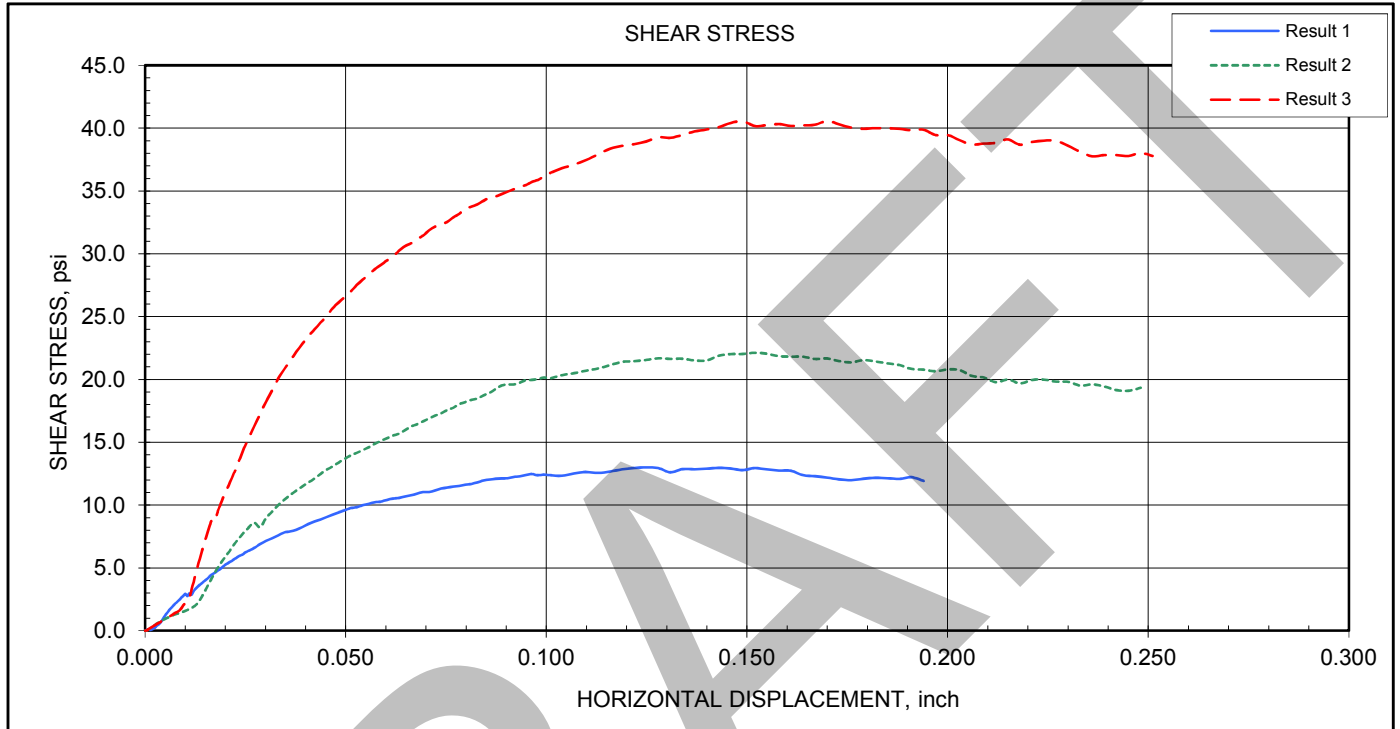
LOCATION: State Rt. 1 SAMPLE NO. 5

JOB NO.: NA185199 DEPTH, feet 20

DATE: 3/4/2019

Terracon

Manchester Cable Landing
State Rt. 1
NA185199
3/4/2019

BORING NO.	1
SAMPLE NO.	5
DEPTH, feet	20

SHEAR STRESS

- Result 1
- Result 2
- Result 3

X-axis: HORIZONTAL DISPLACEMENT, inch
Y-axis: SHEAR STRESS, psi

DISPLACEMENT

- Result 1
- Result 2
- Result 3

Contraction

Dilation

X-axis: HORIZONTAL DISPLACEMENT, in
Y-axis: VERTICAL DISPLACEMENT, in

Terracon

SUPPORTING INFORMATION

Contents:

General Notes
Unified Soil Classification System
Geophysical Investigation Report

Note: All attachments are one page unless noted above.

SAMPLING	WATER LEVEL	FIELD TESTS	

SAMPLING

▌ Rock Core ✋ Grab Sample

☒ Standard Penetration Test

WATER LEVEL

▽ Water Initially Encountered

▼ Water Level After a Specified Period of Time

▼ Water Level After a Specified Period of Time

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

N	Standard Penetration Test Resistance (Blows/Ft.)
(HP)	Hand Penetrometer
(T)	Torvane
(DCP)	Dynamic Cone Penetrometer
UC	Unconfined Compressive Strength
(PID)	Photo-Ionization Detector
(OVA)	Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Descriptive Term (Density)	Standard Penetration or N-Value Blows/Ft.	Descriptive Term (Consistency)	Unconfined Compressive Strength Qu, (tsf)	Standard Penetration or N-Value Blows/Ft.
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

RELATIVE PROPORTIONS OF SAND AND GRAVEL		RELATIVE PROPORTIONS OF FINES	
Descriptive Term(s) of other constituents	Percent of Dry Weight	Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	<15	Trace	<5
With	15-29	With	5-12
Modifier	>30	Modifier	>12

GRAIN SIZE TERMINOLOGY		PLASTICITY DESCRIPTION	
Major Component of Sample	Particle Size	Term	Plasticity Index
Boulders	Over 12 in. (300 mm)	Non-plastic	0
Cobbles	12 in. to 3 in. (300mm to 75mm)	Low	1 - 10
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)	Medium	11 - 30
Sand	#4 to #200 sieve (4.75mm to 0.075mm)	High	> 30
Silt or Clay	Passing #200 sieve (0.075mm)		

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Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	Gravels: More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels: Less than 5% fines [C]	Cu ≥ 4 and 1 ≤ Cc ≤ 3 [E]	GW	Well-graded gravel [F]
			Cu < 4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	Clean Sands: Less than 5% fines [D]	Cu ≥ 6 and 1 ≤ Cc ≤ 3 [E]	SW	Well-graded sand [I]
			Cu < 6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	Silts and Clays: Liquid limit less than 50	Inorganic:	PI > 7 and plots on or above "A"	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K, L, M, N] / Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	Inorganic:	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K, L, M, P] / Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.

[E] $Cu = D_{60}/D_{10}$ $Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
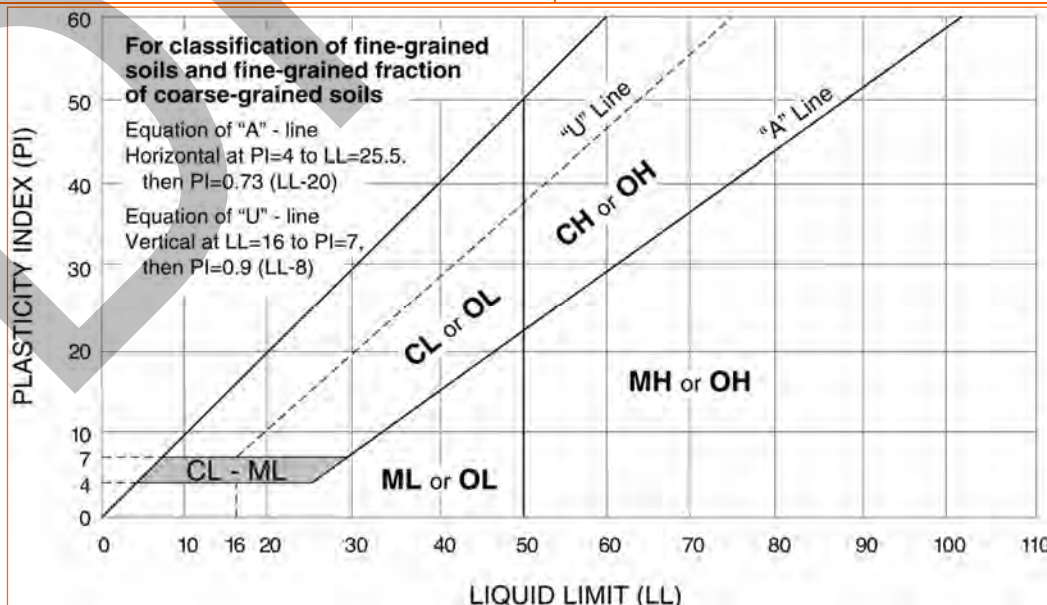
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.

For classification of fine-grained soils and fine-grained fraction of coarse-grained soils

Equation of "A" - line
Horizontal at PI=4 to LL=25.5.
then PI=0.73 (LL-20)

Equation of "U" - line
Vertical at LL=16 to PI=7,
then PI=0.9 (LL-8)

DESCRIPTION OF ROCK PROPERTIES

WEATHERING	
Term	**Description**
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, psi (MPa)**
Extremely weak	Indented by thumbnail	40-150 (0.3-1)
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	150-700 (1-5)
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	700-4,000 (5-30)
Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	4,000-7,000 (30-50)
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	7,000-15,000 (50-100)
Very strong	Specimen requires many blows of geological hammer to fracture it	15,000-36,000 (100-250)
Extremely strong	Specimen can only be chipped with geological hammer	>36,000 (>250)

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Extremely close	< ¾ in (<19 mm)	**Laminated**	< ½ in (<12 mm)
Very close	¾ in – 2-1/2 in (19 - 60 mm)	**Very thin**	½ in – 2 in (12 – 50 mm)
Close	2-1/2 – 8 in (60 – 200 mm)	**Thin**	2 in – 1 ft. (50 – 300 mm)
Moderate	8 in – 2 ft. (200 – 600 mm)	**Medium**	1 ft. – 3 ft. (300 – 900 mm)
Wide	2 ft. – 6 ft. (600 mm – 2.0 m)	**Thick**	3 ft. – 10 ft. (900 mm – 3 m)
Very Wide	6 ft. – 20 ft. (2.0 – 6 m)	**Massive**	> 10 ft. (3 m)

Discontinuity Orientation (Angle): Measure the angle of discontinuity relative to a plane perpendicular to the longitudinal axis of the core. (For most cases, the core axis is vertical; therefore, the plane perpendicular to the core axis is horizontal.) For example, a horizontal bedding plane would have a 0-degree angle.

ROCK QUALITY DESIGNATION (RQD) [1]	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

1. The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

Reference: U.S. Department of Transportation, Federal Highway Administration, Publication No FHWA-NHI-10-034, December 2009
Technical Manual for Design and Construction of Road Tunnels – Civil Elements

March 7, 2019

BHC RHODES
Civil Engineering · Surveying · Utilities
7101 College Boulevard, Suite 400
Overland Park, Kansas 66210-2081

Subject: Geophysical Investigation
             Manchester Cabling Project
             Manchester, California

NORCAL Job # NS195004

Attention: Mr. Christopher Schepmann

This report presents the findings of a geophysical investigation performed by NORCAL Geophysical Consultants, a Terracon Company, for BHC Rhodes at the site of a proposed cable landing along the coastline approximately 3 miles north of Manchester, California. The investigation was authorized under Terracon Inter-Office Services for the Terracon Lodi Office, Project No. NA185199. The fieldwork was conducted during the period January 22 through 24, 2019 by NORCAL Professional Geophysicist David T. Hagin (CA PGp No. 1033) and Senior Geophysical Technician Travis W. Black.

1.0 SITE DESCRIPTION

The site naturally divides itself into two areas, as can be seen on Plate 1. One area is on the beach, at the base of a coastal bluff cliff. The area is a typical coastal site, consisting of a sand, driftwood and brush covered terrain sloping downward to the west. Elevations in the beach area vary from near sea level to about 40 feet (above msl). The other area is atop the bluffs between the top of the cliff and Highway 1. This area is generally flat and covered with grass and low brush, with elevations ranging from about 160 to 175 feet.

Terracon Lodi advanced exploratory boring B-1 (February, 2019) approximately 60 feet east of the eastern end of Line SR-4, as shown on Plate 1. The boring log for B-1, furnished by the Lodi

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office, indicates that the upper 15 feet consists of unconsolidated sandy sedimentary deposits with low blow counts (SPT). Between depths of 15 and 50 feet the logs show completely weathered graywacke with significantly higher blow counts. Below a depth of 50 feet, the boring encountered rock consisting of highly weathered extremely fractured graywacke with thin interlayered beds of highly weathered shale. Below 90 feet, the logs indicate a change to extremely fractured, unweathered, strong graywacke.

Geologic maps (California Geological Survey, 2010) indicate a subsurface consisting of Mesozoic sedimentary and metasedimentary rocks. The nearest active trace of the San Andreas Fault is mapped approximately one-half mile to the southwest of the site and a Holocene/Pleistocene splay to the north of the main trace appears to trend toward the site.

2.0 PURPOSE

The purpose of the geophysical investigation is to characterize the nature of the site bedrock by measuring the seismic P-wave (Vp) and S-wave (Vs) velocity values prior to planned excavation and horizontal drilling for the cable landing. To achieve this goal, we performed a Seismic Refraction (SR) survey to measure the Vp values and a Multichannel Analysis of Surface Waves (MASW) survey to measure the Vs values.

Detailed descriptions of the SR and MASW methods, including the methodology and instrumentation, our data acquisition and analysis procedures, as well as the limitations of the methods are included in Appendix A.

3.0 SCOPE OF WORK

Our scope of work consisted of conducting SR profiling and MASW soundings at four locations; two within the beach area and two on the bluffs, as shown on Plate 1. Terracon Lodi personnel determined the location and orientation of the four SR lines, labeled Line SR-1 through Line SR-4, and the four associated MASW soundings, labeled MASW-1 through MASW-4, shown on Plate 1. Each MASW sounding is centered on the associated SR line.

4.0 SEISMIC REFRACTION RESULTS

4.1 SEISMIC VELOCITY PROFILES

The results of the SR surveys conducted along the four lines shown on Plate 1 are illustrated by the seismic velocity cross-sections (profiles) shown on Plate 2. The profiles depict variations of Vp both laterally and with depth beneath each line. The black line at the top of each diagram represents the ground surface. On each profile the horizontal axis represents the distance (station) in feet from the beginning of the line and the vertical axis represents elevation in feet (above msl). The Vp values are in feet per second (ft/sec), and are represented by color-shaded, labeled contours according to the color bar shown at the bottom of the plate. The same velocity range and color scale was used on all four profiles in order to facilitate their comparison.

4.2 P-WAVE VELOCITIES

The P-wave velocities (Vp) measured by the seismic refraction survey range from about 1,000 ft/sec at the surface to around 7,500 ft/sec at depth. This velocity range can be differentiated into three sub-ranges which we define herein as low, moderate and high. Low Vp range from approximately 1,000 to 4,000 ft/sec and are represented by tan to yellow shading. Vp in this range typically represent surficial soils and poorly consolidated sedimentary deposits or fill. Moderate Vp range from 4,000 to 6,000 ft/sec and are represented by green to blue shading. Vp in this range probably represent more consolidated, cemented or saturated sediments, or possibly deeply weathered rock. High Vp range from 6,000 to about 7,500 ft/sec and are represented by varying shades of maroon. These velocities probably represent bedrock in various degrees of weathering, where the degree of weathering and/or fracturing decreases with increased Vp. The maximum Vp values measured are only moderate, indicating significant weathering and/or fracturing of the local bedrock to the depths measured by this survey

4.3 DISCUSSION

4.3.1 Line SR-1

The seismic velocity profile for Line SR-1 shows a zone of low Vp (1,000 to 4,000 ft/sec) extending across the profile from ground surface to depths varying between 10 and 15 feet. This zone likely coincides with the depth of the unsaturated beach sand at the time of the survey. Underlying the low Vp zone, the profile displays a zone of moderate Vp (4,000 to 6,000 ft/sec)

that ranges from about 5 to 11 feet in thickness and is somewhat thicker near the southern end of the line. This zone likely represent highly weathered bedrock, completely saturated sands or some combination of the two. Below depths of 18 to 23 feet, we note high Vp (up to ~7,500-ft/sec) suggesting moderately weathered/fractured rock extending to the bottom of the profile, at a depth of 80 feet. The seismic velocity zones described above are generally horizontal with a slight low near the center of the line.

4.3.2 Line SR-2

The seismic velocity profile for Line SR-2 displays a range of Vp values similar to that shown on Line SR-1. The descriptions of the various layers are similar to those described in Section 4.3.1, as are the maximum Vp values. The primary difference between the profiles is the sloping nature of both the ground surface and the Vp contours, which both show apparent dip toward the southwest. Traversing the profile from northeast to southwest, the base of the low Vp zone deepens from about 10 feet to 23 feet below ground surface, creating a wedge-shaped lens that likely consists largely of unconsolidated sands. The depth to the top of the high Vp zone along the same traverse varies from around 15 feet to 35 feet in depth. Maximum Vp values on Line SR-2 are somewhat less than 7,000 ft/sec.

4.3.3 Lines SR-3 and SR-4

As shown on Plate 1, we combined Lines SR-3 and SR-4 were conducted end-to-end to form a single long, east-west trending line. As the two lines depict very different velocity structures, combining them allows us to see these changes on a single continuous profile, thus facilitating our analysis. Since the lines do not overlap, the ray trace coverage (resolution) is decreased somewhat where the lines join. However, the combined profile is helpful to better visualize the structure of the subsurface.

The primary feature of note is an apparent vertical offset within the deeper velocity values that suggests a structural element. This feature is visible on the profile as a lateral change in coloring between station 400 and station 500, below a depth of about 35 feet. The proximity of the San Andreas Fault evokes the idea that this feature may be fault-related. Lower Vp values at depth and vertically oriented contour lines within this area suggest the possible presence of fault related features such as a shear zone or possibly fault gouge. In addition, considering that the layering in the upper 35 feet does not appear offset, the feature does not appear to be recent. The maximum values at depth on the eastern portion of the line are similar to the

maximum values measured on the beach lines (6,000 to 7,000 ft/sec). However, the velocity values beneath the western portion of the line are generally lower (3,000 to 5,000 ft/sec) and may indicate a high degree of fracturing, or possibly a change in subsurface materials. High Vp visible near the bottom of the profile at the western end may correlate with high Vp at shallower depth at the eastern portion. This would be supporting the idea of a vertical structural offset.

Borehole B-1 (summarized in Section 1.0) is approximately 60 feet east of the eastern end of Line SR-4. Although this is not close enough for a direct correlation, a comparison between the seismic profile and the borehole log suggests that the low Vp zone correlates with the sedimentary layers, the moderate Vp values correspond to the completely weathered graywacke and the high Vp is associated with the highly weathered rock consisting of interlayered graywacke and shale.

5.0 MASW RESULTS

5.1 1D Vs MODELS

The seismic S-wave velocity (Vs) versus depth graphs shown on Plates 3 and 4 illustrate the results of the MASW surveys for MASW-1 through MASW-4. The horizontal scales on these step plots represent Vs in feet per second (ft/sec) and the vertical scales represent depth in feet below the ground surface (bgs) at the center of the line. The solid blue line on each graph depicts the variation in Vs versus depth. The velocities and depths are also tabulated in Table A through Table D, presented below.

Table A : MASW-1 S-Wave Velocity vs Depth

DEPTH RANGE (FT)	S-WAVE VELOCITY (FT/SEC)
0.0 – 1.6	470
1.6 – 2.7	930
2.7 – 4.1	430
4.1 – 5.8	320
5.8 – 8.0	490
8.0 – 10.7	510
10.7 – 14.1	920
14.1 – 18.3	1,240
18.3 – 70.0	1,800

Table B : *MASW-2 S-Wave Velocity vs Depth*

DEPTH RANGE (FT)	S-WAVE VELOCITY (FT/SEC)
0.0 – 3.0	650
3.0 – 5.1	750
5.1 – 7.7	840
7.7 – 11.0	810
11.0 – 20.3	510
20.3 – 26.7	1,200
26.7 – 34.7	1,400
34.7 – 70.0	1,880

Table C : *MASW-3 S-Wave Velocity vs Depth*

DEPTH RANGE (FT)	S-WAVE VELOCITY (FT/SEC)
0.0 – 2.4	1,090
2.4 – 5.4	1,160
5.4 – 9.1	1,250
9.1 – 13.7	1,180
13.7 – 19.5	870
19.5 – 26.8	730
26.8 – 35.9	1,290
35.9 – 47.2	1,670
47.2 – 61.4	1,390
61.4 – 90.0	2,090

Table D : *MASW-4 S-Wave Velocity vs Depth*

DEPTH RANGE (FT)	S-WAVE VELOCITY (FT/SEC)
0.0 – 1.9	1,130
1.9 – 4.3	1,460
4.3 – 7.3	1,640
7.3 – 11.0	1,470
11.0 – 15.7	1,530
15.7 – 21.6	710
21.6 – 28.9	870
28.9 – 38.0	2,220
38.0 – 49.4	2,580
49.4 – 90.0	3,460

5.2 DISCUSSION

Generally, we note a good correlation between the Vp values measured by the SR survey and the Vs values measured by the MASW survey, noting that Vs values are typically ½ to ¼ of the Vp values for most earth materials. The Vs values tend to increase with increasing depth; however, several Vs inversions (decreasing Vs with depth) are apparent.

The standard method of reporting MASW data is to consider the location of the 1D velocity vs. depth model as the center point of the MASW spread. However, this does not mean that the measured velocity values represent materials solely beneath that location. In fact, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values.

6.0 LIMITATIONS

It should be noted that the seismic refraction technique is based on the assumption that seismic velocity increases with depth. Any layers representing a decrease in velocity with depth, otherwise known as a velocity inversion, will not be defined and will result in the over-estimation of the depth of deeper, higher velocity layers. In addition, relatively thin layers might not be individually resolved and might, instead, be lumped together with other layers. Hard and soft zones within a given seismic layer will tend to be averaged into the velocity of that layer. There is not necessarily a one-to-one relationship between lithologic layers and seismic layers. It is

entirely possible that two different types of material could have the same seismic velocity. Alternatively, a change in velocity can occur within a single lithologic unit. Thus, seismic velocity layers are not necessarily representative of lithologic layers.

Since the accuracy of our findings is subject to the limitations described above, it should be noted that subsurface conditions may vary from those depicted in the final results. A more detailed discussion of the limitations with regard to the seismic refraction method is presented in Appendix A.

7.0 STANDARD CARE AND WARRANTY

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the shallow subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate having the opportunity to provide our services for this investigation. Please do not hesitate to call if you are in need of further geophysical consulting services.

Respectfully,
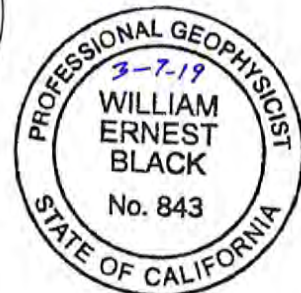
NORCAL Geophysical Consultants, Inc.

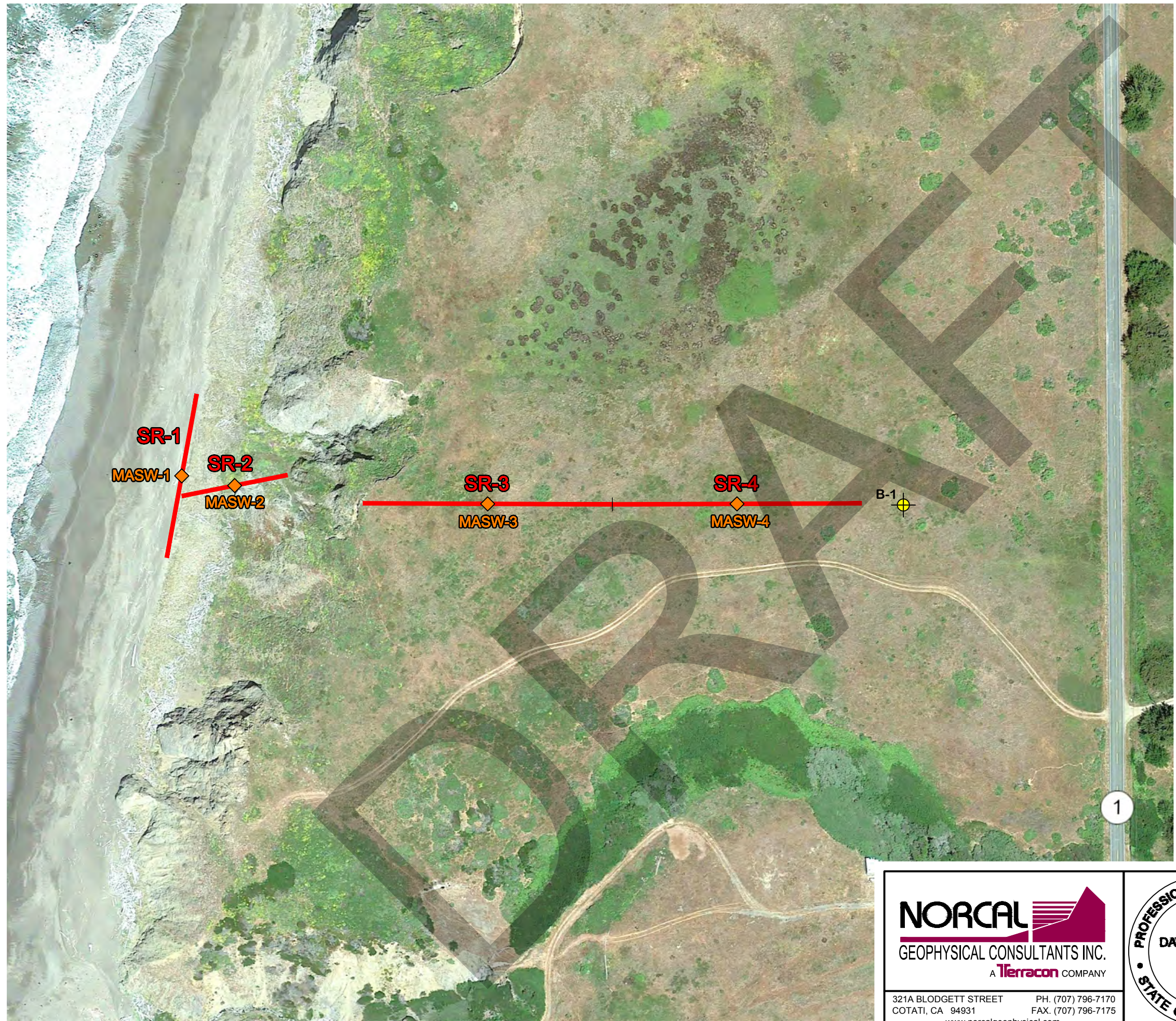David T. Hagin
California Professional Geophysicist, PGp 1033

William E. Black
California Professional Geophysicist, PGp 843

DTH/WEB
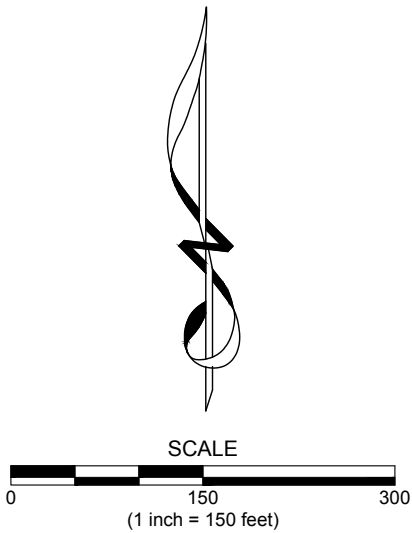Enclosures: Plates 1 through 4, Appendix A

SCALE
0   3000   6000
(1 inch = 6000 feet)

AREA SHOWN AT LEFT

Kinney Rd

Manchester

VICINITY MAP

SR-1
MASW-1
SR-2
MASW-2
SR-3
MASW-3
SR-4
MASW-4
B-1

1

SCALE
0   150   300
(1 inch = 150 feet)

LEGEND

▬▬▬	SEISMIC REFRACTION LINE
◆	MASW SOUNDING LOCATION
⊕	BORING

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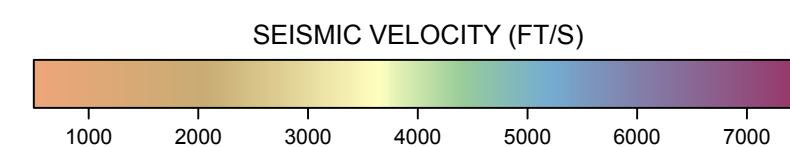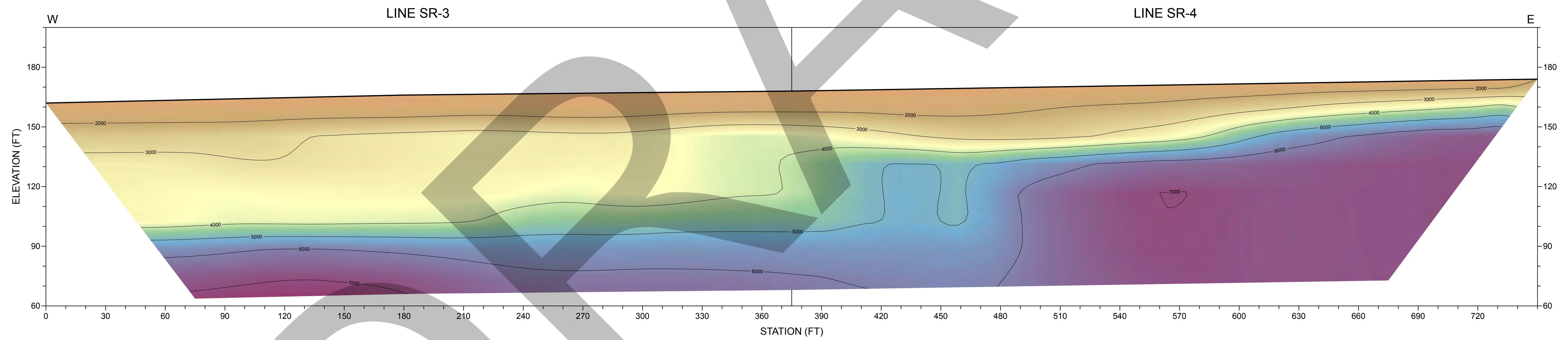
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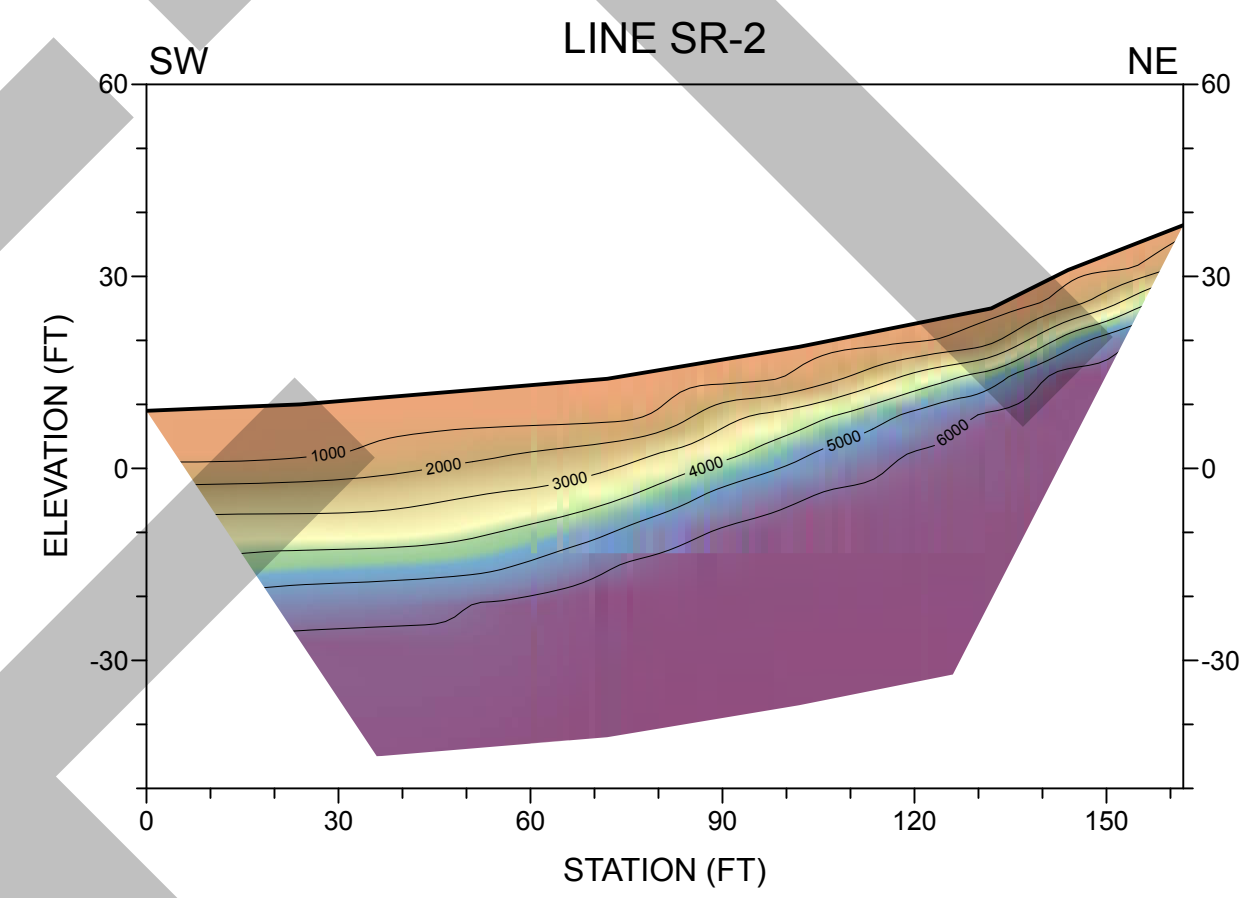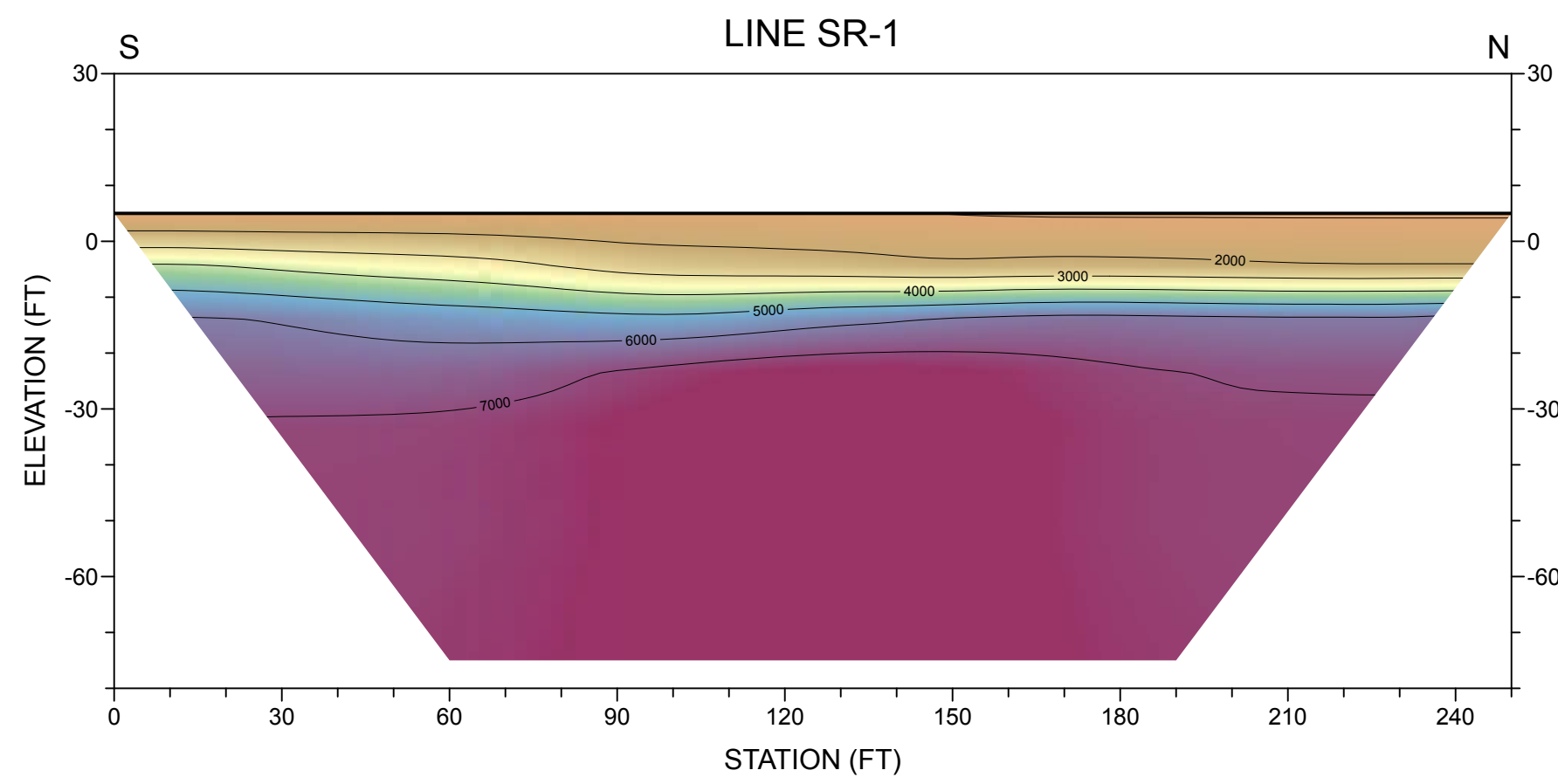DAVID T HAGIN
No. 1033
STATE OF CALIFORNIA

SITE LOCATION MAP
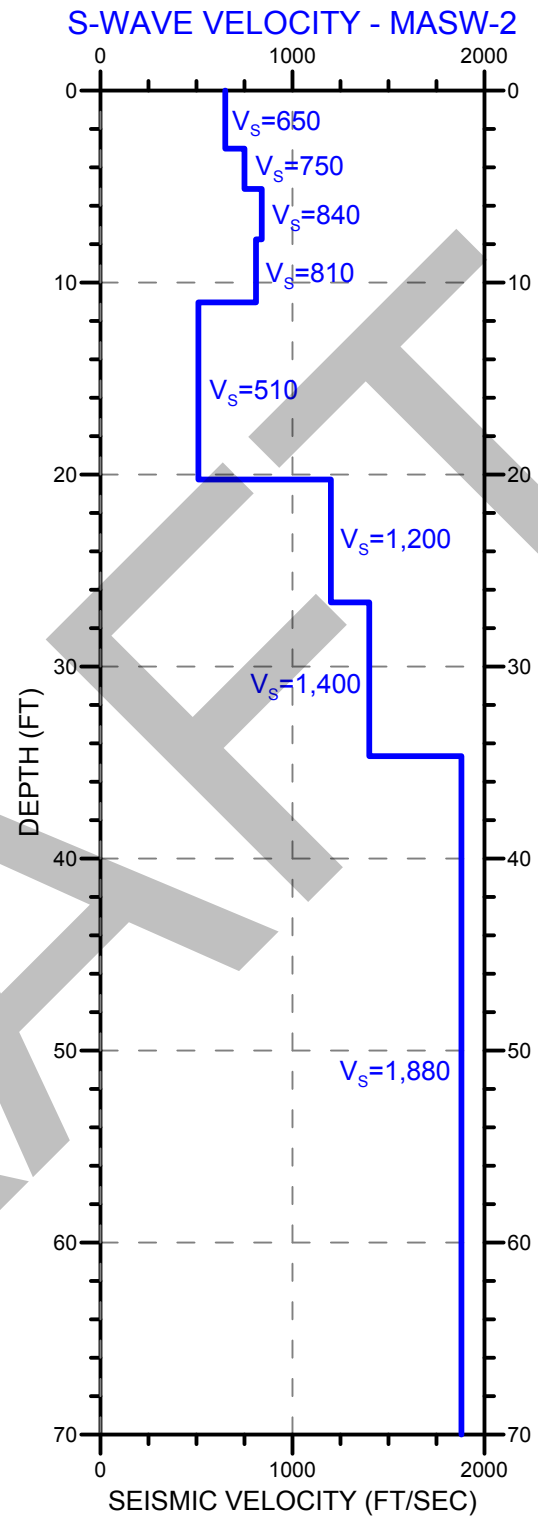GEOPHYSICAL SURVEY
MANCHESTER CABLING PROJECT

LOCATION: MANCHESTER, MENDOCINO COUNTY, CALIFORNIA

CLIENT: BHC RHODES

JOB #. NS195004	DATE: MARCH 2019	PLATE
DRAWN BY: G.RANDALL	APPROVED BY: DTH	**1**
David Hagin	3/7/2019	

LINE SR-1

LINE SR-2

LINE SR-3

LINE SR-4

SEISMIC VELOCITY (FT/S)

1000 2000 3000 4000 5000 6000 7000

SCALE

0   30   60

(1 inch = 30 feet)

STATION (FT)

ELEVATION (FT)

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PROFESSIONAL GEOPHYSICIST
DAVID T HAGIN
No. 1033
STATE OF CALIFORNIA

SEISMIC REFRACTION PROFILES
LINES SR-1 - SR-4
MANCHESTER CABLING PROJECT

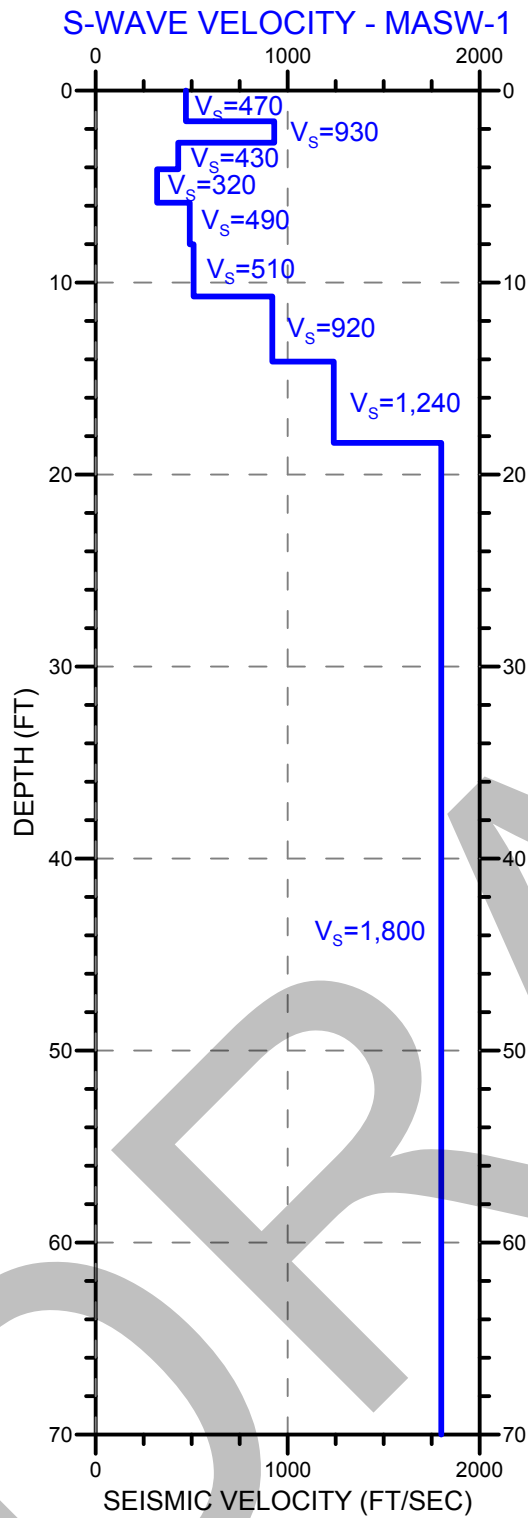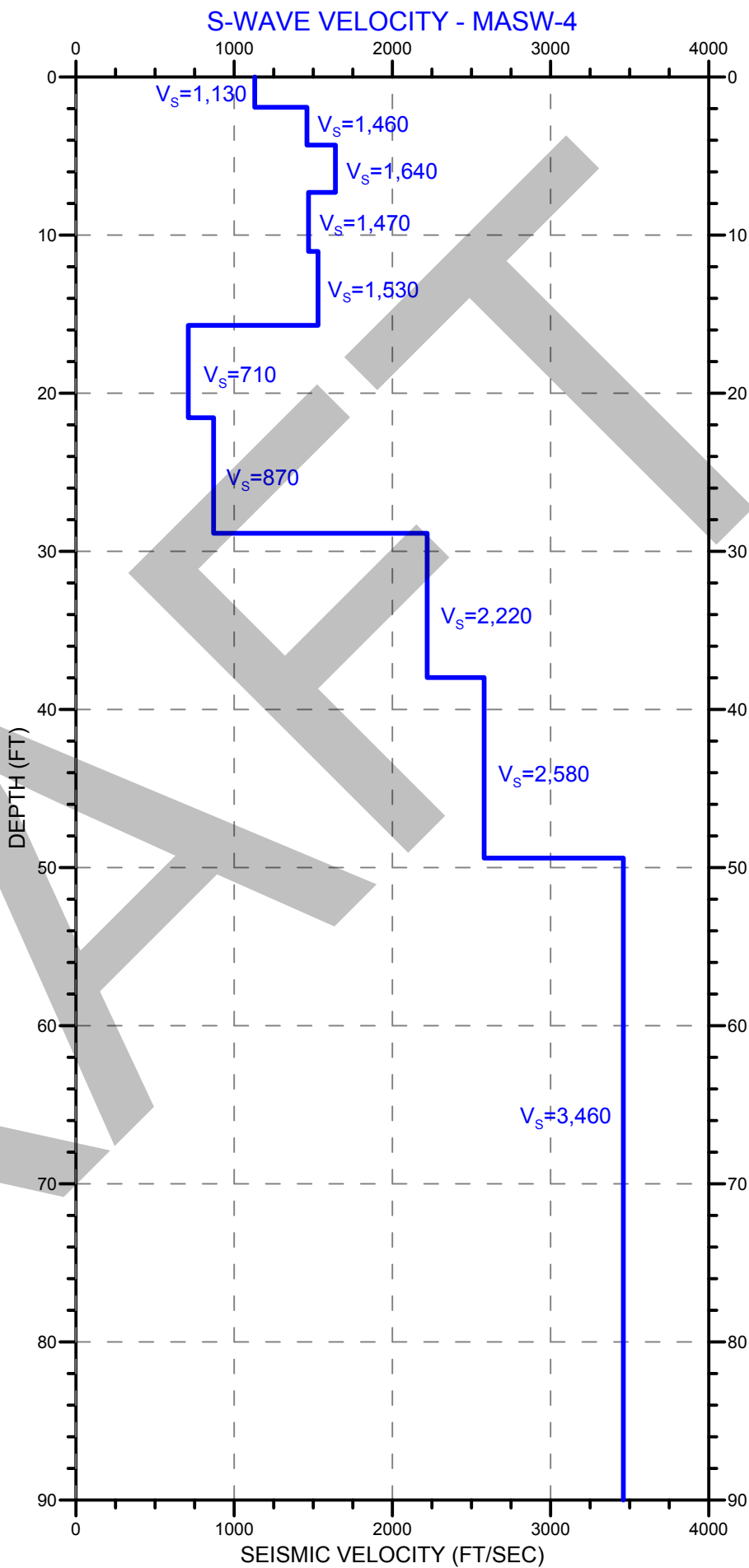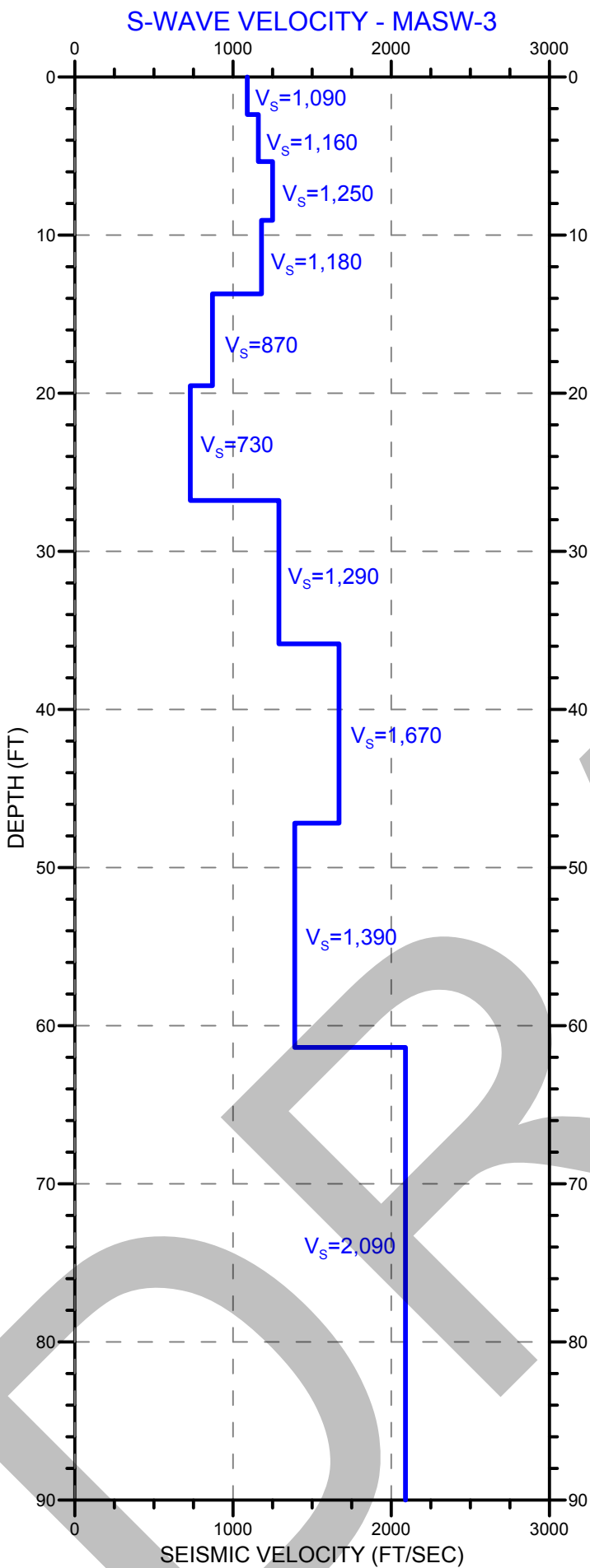LOCATION: MANCHESTER, MENDOCINO COUNTY, CALIFORNIA

CLIENT: BHC RHODES

JOB #: NS195004 | DATE: MARCH 2019

DRAWN BY: G.RANDALL | APPROVED BY: DTH

PLATE
2

S-WAVE VELOCITY - MASW-1

$V_S=470$

$V_S=930$

$V_S=430$

$V_S=320$

$V_S=490$

$V_S=510$

$V_S=920$

$V_S=1,240$

$V_S=1,800$

DEPTH (FT)

SEISMIC VELOCITY (FT/SEC)

S-WAVE VELOCITY - MASW-2

$V_S=650$

$V_S=750$

$V_S=840$

$V_S=810$

$V_S=510$

$V_S=1,200$

$V_S=1,400$

$V_S=1,880$

DEPTH (FT)

SEISMIC VELOCITY (FT/SEC)

LEGEND

S-WAVE VELOCITY (MASW)

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DAVID T HAGIN

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STATE OF CALIFORNIA

MASW SOUNDINGS
MASW-1 - MASW-2
MANCHESTER CABLING PROJECT

LOCATION: MANCHESTER, MENDOCINO COUNTY, CALIFORNIA

CLIENT: BHC RHODES

JOB #: NS195004

DATE: MARCH 2019

DRAWN BY: G.RANDALL

APPROVED BY: DTH

3/7/2019

PLATE

3

S-WAVE VELOCITY - MASW-3

V_S=1,090
V_S=1,160
V_S=1,250
V_S=1,180
V_S=870
V_S=730
V_S=1,290
V_S=1,670
V_S=1,390
V_S=2,090

DEPTH (FT)

SEISMIC VELOCITY (FT/SEC)

S-WAVE VELOCITY - MASW-4

V_S=1,130
V_S=1,460
V_S=1,640
V_S=1,470
V_S=1,530
V_S=710
V_S=870
V_S=2,220
V_S=2,580
V_S=3,460

DEPTH (FT)

SEISMIC VELOCITY (FT/SEC)

LEGEND

—— S-WAVE VELOCITY (MASW)

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STATE OF CALIFORNIA

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Appendix A

GEOPHYSICAL METHODS:

SEISMIC REFRACTION (SR) PROFILING

MULTICHANNEL ANALYSIS OF SURFACE WAVES (MASW) SOUNDINGS

Appendix A

1.0 SEISMIC REFRACTION (SR) PROFILING

1.1 METHODOLOGY

The seismic refraction method provides information regarding the seismic velocity structure of the subsurface. An impulsive (mechanical or explosive) source is used to produce compressional (P) wave seismic energy at the surface. The P-waves propagate into the earth and are refracted along interfaces caused by a downward increase in velocity. A portion of the P-wave energy is typically refracted along this interface and back to the surface where it is detected by sensors (geophones) that are coupled to the ground surface in a collinear array (spread). The detected signals are recorded on a multi-channel seismograph and are analyzed to determine the shot point-to-geophone travel times. These data can be used along with the corresponding shot point-to-geophone distances and elevation data to determine the depth, thickness, and velocity of subsurface seismic layers.

An SR profiling survey provides a measure of the seismic P-wave velocity variations both laterally and with depth. Typically, P-wave velocities (Vp) within rock are significantly higher than in unconsolidated sediments. Within rock, Vp are inversely proportional to the degree of weathering and/or fracturing of the rock. That is, the less weathered and/or fractured the rock the higher its Vp. Within unconsolidated sediments, Vp are directly proportional to their degree of consolidation, cementation and/or moisture content. That is, the higher these parameters are the higher their Vp.

1.2 DATA ACQUISITION

We collected SR data from four seismic lines labeled Line SR-1 through Line SR-4, as shown on Plate 1. Terracon Lodi determined the locations and orientations of the seismic lines. The lengths of the SR lines varied from 162- to 375-ft. Each line consisted of 24 geophones and 5 shot points distributed in a collinear array. A sample distribution of shot points and geophones for Line SR-1 is shown in Figure 1.1. The other lines consisted of a similar distribution with geophone spacings that ranged from 6- to 15-ft.
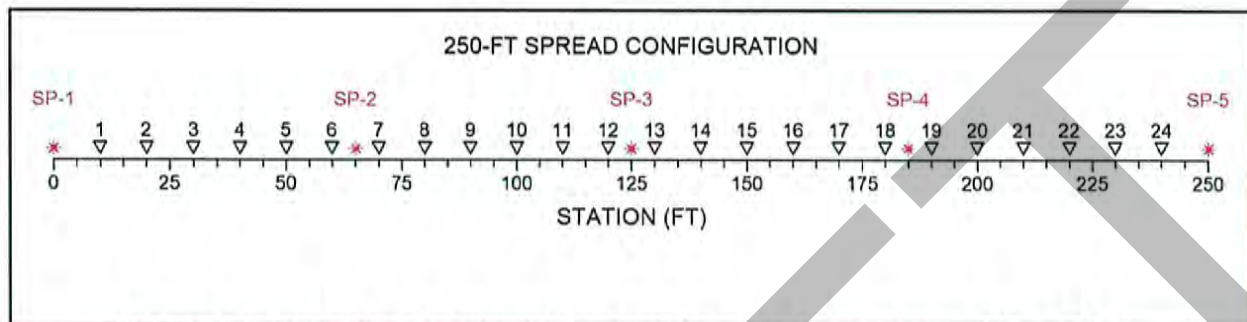
250-FT SPREAD CONFIGURATION

Figure 1.1: *Distribution of shot points and geophones for 250-foot spread (Line SR-1).*

1.3 INSTRUMENTATION

The seismic waveforms produced at each shot point were recorded using a Geometrics **Geode** 24-channel engineering distributed array seismograph (see photo below) and Oyo **Geospace** geophones with a natural frequency of 8 Hz. The geophones were coupled to the ground surface by a metal spike affixed to the bottom of each geophone case. Seismic energy was produced at each shot point by multiple impacts with a 16 pound sledge hammer against a metal strike plate placed on the ground surface. The seismic waveforms were digitized, processed and amplified by the Geode, transmitted via a ruggedized Ethernet cable to a field computer and algebraically summed (stacked) until sufficient signal to noise ratio was achieved. The data were displayed on the computer's LCD screen in the form of seismograms, analyzed for quality assurance and archived for subsequent processing. These images were later used to determine the time required for P-waves to travel from each shot point to each geophone in a given array (spread).

1.4 DATA ANALYSIS

The seismic refraction data were processed using the software package **SeisImager,** written by Oyo Corporation (Japan) and distributed by Geometrics Inc. This package consists of two programs titled **Pickwin,** Version 5.1.1.2 (2013) and **Plotrefa,** Version 3.0.0.6 (2014). For each seismic line we used **Pickwin** to view the seismic records and identify first arriving P-wave

energy at each geophone and to determine the shot point to geophone travel time associated with each arrival. We then used **Plotrefa** to assign elevations to each geophone and to plot the shot point to geophone travel times versus their distance (Station) along the line. A sample Time versus Depth (T-D) graph is shown in Figure 1.2. After examining the T-D graph we assigned velocity layers (1-3) to each travel time and then computed a 2D model using **Plotrefa's** time-term routine. This resulted in a 2D layered cross-section (profile) illustrating seismic velocity versus depth. A sample 2D time-term model is shown in Figure 1.3.
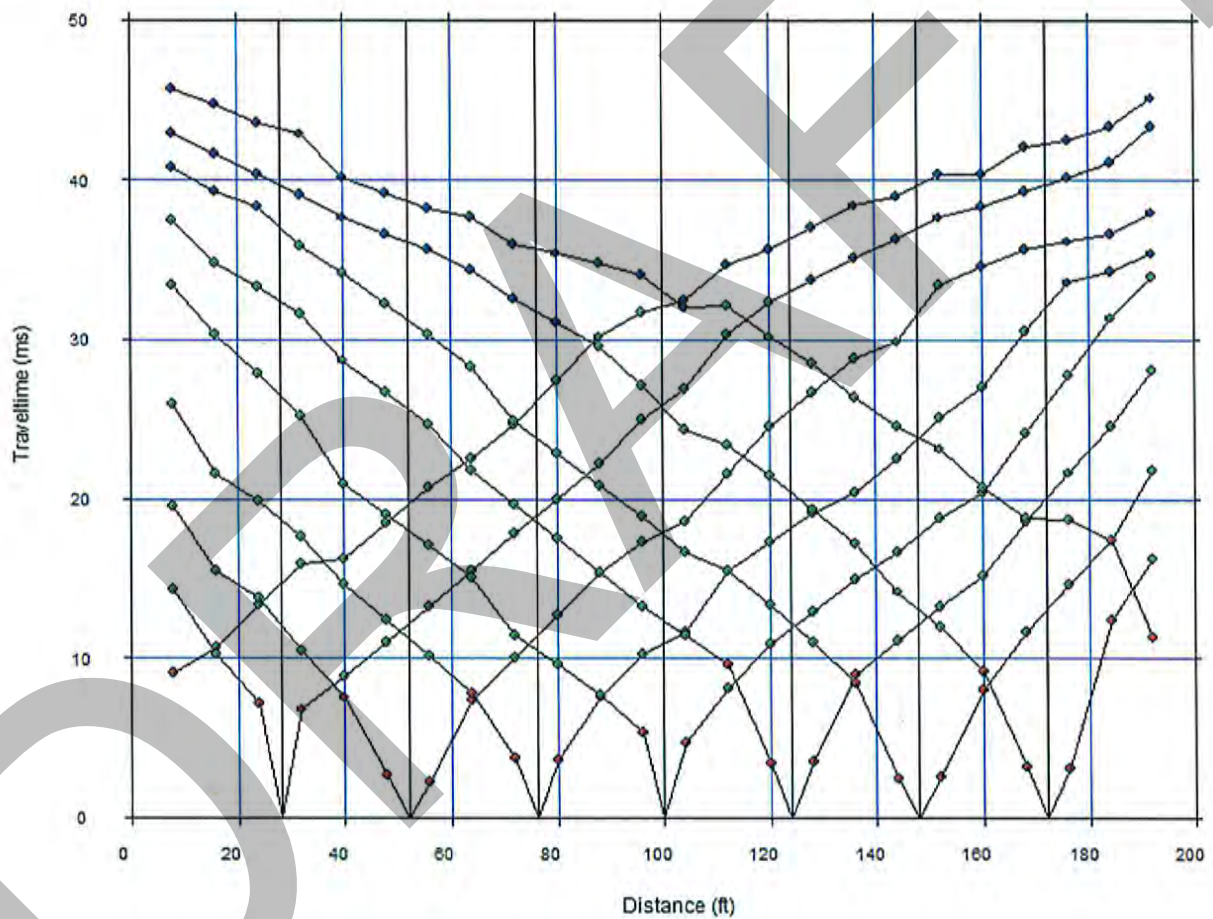
Figure 1.2: *Sample SR Time-Distance Graph. Red circles represent layer 1 (V1), green circles represent V2 and blue circles represent V3.*
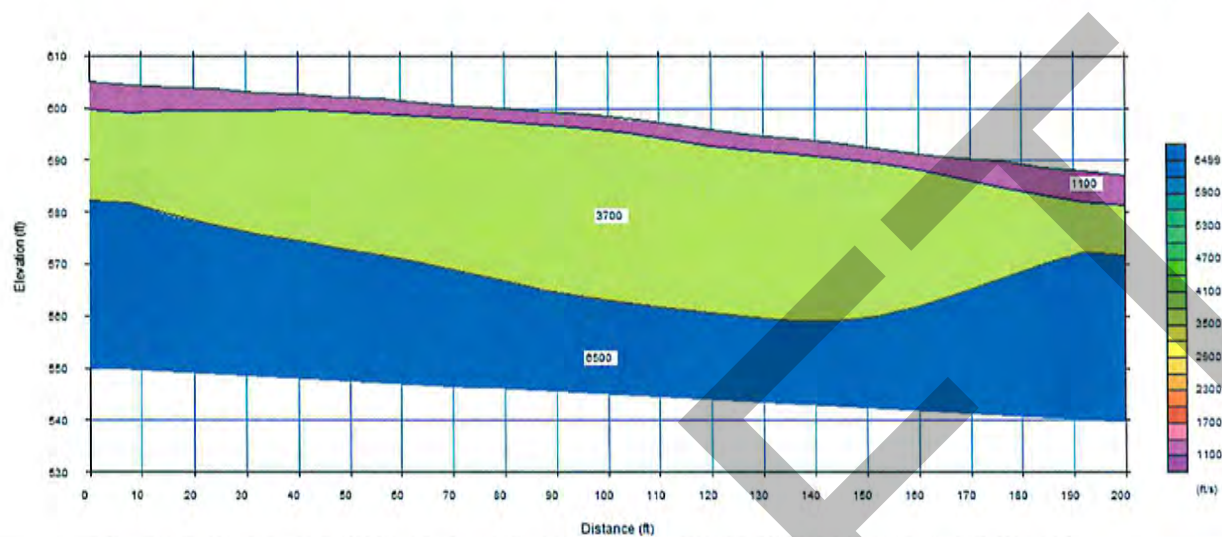
Figure 1.3: *Sample Time-Term Seismic Velocity Model. Velocities are labeled and indicated by the color bar on the right.*

Finally, we used the time term model as input to **Plotrefa's** tomographic routine. This routine divided the input model into cells according to the geophone spacing and depth range and assigned a velocity to each cell. It then used a ray tracing routine to compute synthetic travel times through the model from each shot point to every geophone. The synthetic travel times were compared with the observed travel times to determine the goodness of fit. If the fit was not within certain assigned parameters, the program then adjusted the velocity in each cell and reran the ray tracing. This procedure was repeated through as many as 20 iterations in order to achieve the optimum fit between observed and synthetic travel times. A sample tomographic model is shown in Figure 1.4.
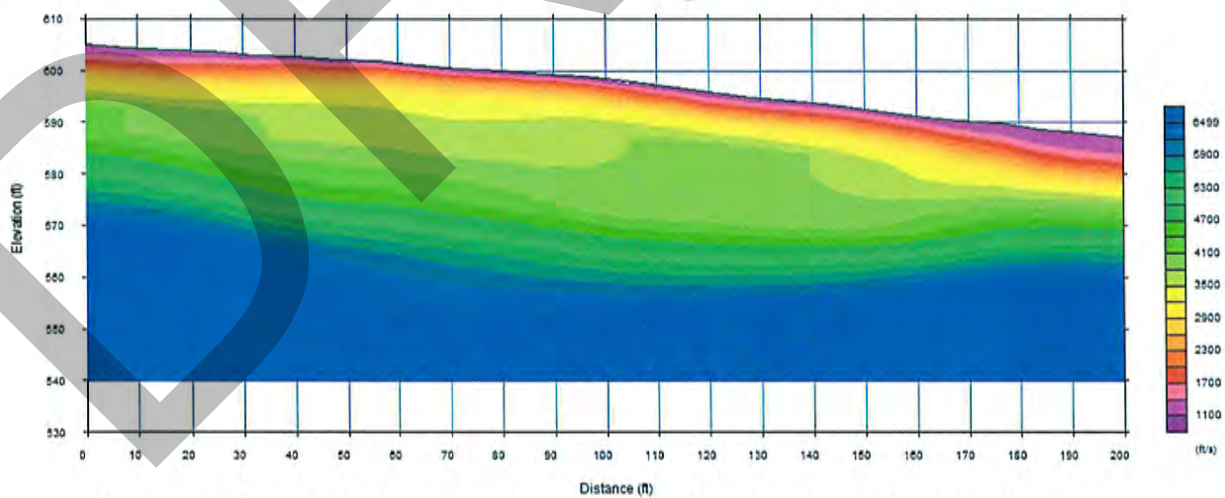

Figure 1.4: *Sample tomographic Inverted Seismic Velocity Model. Velocities indicated by color bar on right.*

Once the tomographic processing was complete, we used the computer program **Surfer 13.0** by Golden Software to construct a color contoured 2D cross-section (profile) illustrating the results for each seismic line.

1.5 INTERPRETATION

The SR profiles described above are models of the subsurface based on P-wave velocities. How these velocities and their subsurface distribution relate to geology is a matter of interpretation. This interpretation can be based on experience and a general knowledge of the local geology. However, the best results are achieved when the models can be correlated with subsurface information provided by other means such as onsite observations, borehole geological and/or geophysical logs, trench logs or projections based on mapped surface geology. This type of information is referred to as "ground truth".

1.6 LIMITATIONS

Based on the physical properties of refraction (Snell's Law), in order for a seismic wave to be refracted back toward the surface the seismic velocity of the upper layer must be less than the velocity of the lower layer. When higher velocities overlie lower velocities, often referred to as a velocity inversion, the seismic energy will be refracted downward and the lower layer will not be detected at the surface. As a result, the calculated depths of any deeper higher velocity layers may be over-estimated. Furthermore, some layers may be truncated, or too thin to detect. These are referred to as "hidden layers."

If the seismic source used for the survey does not produce sufficient energy to propagate through the entire spread at detectable levels, the first arriving P-waves at each geophone may not be visible on the seismic records. Additionally, extraneous seismic energy sources such as wind, ocean waves, traffic or nearby machinery may create "noise" on the recorded waveforms that may mask the first arrivals.

In noisy conditions many "stacks" may be necessary to achieve an acceptable signal to noise ratio. Stacking consists of superposition of waveforms such that the stacked shot energy builds with successive shots whereas the noise tends to cancel itself out due to its random nature.

Another common external noise source is overhead power lines. If the cable is laid out parallel to the lines electrical noise may be induced in the cable. Possible internal noise sources may be faulty geophone connections due to dirt or moisture, or use of an unsuppressed power supply.

Finally, seismic refraction processing algorithms are based on the assumption that the seismic velocity layers are isotropic. That is, that the velocity is uniform within the length and breadth of each layer. Another assumption is that the velocity distribution does not change in a direction transverse to the seismic line. In other words, that there is true 2D symmetry. If these conditions are not met, the actual subsurface conditions will vary from those represented by the seismic model.

Given these limitations, we recognize that a seismic velocity profile represents a model of the subsurface the is subject to interpretation and is conceptual in nature. It is not intended or expected to represent an exact depiction of the subsurface.

2.0 MULTI-CHANNEL ANALYSIS OF SURFACE WAVES (MASW) SOUNDINGS

2.1 METHODOLOGY

When seismic waves are generated at or near the ground surface, both body and surface waves are generated. Body waves expand omni-directionally throughout the subsurface. They consist of both compressional (P) and shear (S) waves. Surface waves (e.g., Rayleigh, Love, etc.) radiate along the ground surface at velocities that are proportional to shear wave velocity (Vs). If a vertical energy source is used, Rayleigh type surface waves are produced. These are commonly referred to as "ground roll" in seismic surveys. Rayleigh waves are characterized by retrograde elliptical motion, and travel at approximately 0.9 times the velocity of S-waves.

Surface waves account for more than two-thirds of the energy produced by vertical seismic energy sources. As a result, surface waves are the most prominent signal on multi-channel seismic records. In addition, surface waves have dispersion properties that body waves lack. That is, different wavelengths have different penetration depths and, therefore, propagate at different velocities. By analyzing the dispersion of surface waves, it is possible to obtain an S-wave versus depth velocity profile. Since Vs is directly proportional to shear modulus, this provides a direct indication in the variation of stiffness (or rigidity) of subsurface materials.

We recorded and analyzed Surface waves using a method referred to as Multi-channel Analysis of Surface Waves (MASW). For this survey, we employed the method to collect and analyze surface wave data using a fixed array of geophones and shot points. This is referred to as a sounding and results in a 1D model depicting variation in S-wave velocity versus depth beneath the center of the array. However, the subsurface conditions underlying the entire length of the array, and for several tens of feet to either side, contribute to the measured velocity values. The

method requires an energy source that is capable of producing ground roll and geophones that are capable of detecting low frequencies (<10 Hz) signals.

2.2 DATA ACQUISITION

We acquired MASW sounding data by deploying an array parallel to each SR line, expanding around the center point of each of the four SR lines, respectively, as shown on Plate 1. The locations are generally along the planned cable right-of-way. At each sounding location, we deployed an array of 24 geophones with 4 shot-points, distributed as shown in Figure 2.1.
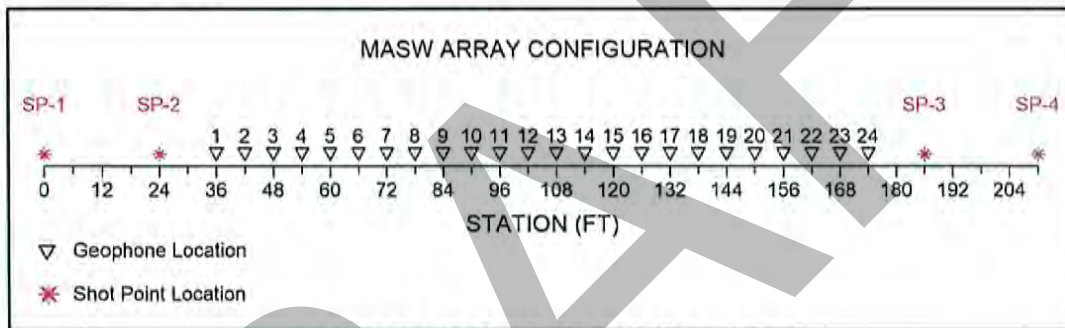

Figure 2.1: *MASW Array Configuration*

Seismic energy was produced at each shot point using a 16-pound sledgehammer striking a metal strike plate on the ground surface. The resulting seismic waveforms were detected by an array of 24 Oyo **Geospace** geophones with a natural frequency of 8-Hz and recorded using a Geometrics **Geode** 24-channel distributed array engineering seismograph. The seismic waveforms were digitized, processed and amplified by the Geode and transmitted via a ruggedized Ethernet cable to a field computer. The recorded data were archived for subsequent processing and displayed on the computer's LCD screen in the form of seismograms for quality assurance analysis.

2.3 DATA ANALYSIS

The seismic wave-traces (shot gathers) recorded at each shot point were analyzed using the computer program **SURFSEIS** developed by the Kansas Geological Survey (Version 5.0, 2016). This interactive program converts the data acquired from all four shot points in a given sounding into a dispersion curve representing phase velocity versus frequency. This curve is then inverted to produce a 1D model indicating S-wave velocity versus depth. The steps involved in this procedure are as follows:

1) The shot gathers are converted to KGS format.
2) Stations are assigned to the geophone and shot point locations.
3) The resulting records are viewed to determine their overall quality. If necessary, portions of the records are muted to remove interference from refractions, reflections and higher mode events.
4) For each formatted (and/or muted) record, the program produces what is referred to as an "overtone plot". This is a colored cross-section indicating phase velocity versus frequency and amplitude. The vertical axis represents phase velocity (increasing upward); the horizontal axis represents frequency (increasing to the right); and signal amplitude is indicated by various colors, with the hottest colors (orange to red to dark brown) representing the strongest signals. Typically, the strongest signals align in a curved pattern with a symmetry similar to a "hockey stick" where the blade is pointing upward at the lower end of the frequency spectrum (higher velocity at greater depth) and the handle projects to the right in the direction of increasing frequencies indicating lower velocities.
5) The overtone plots compiled from the four shot points are reviewed to determine their overall quality and the best among them (possibly all) are merged to form a single overtone. This enhances the overall signal to noise ratio of the survey.
6) The resulting overtone plot is used as a guide in deriving a dispersion curve representing phase velocity versus frequency. This is done by fitting the curve along the center of the hockey stick where the signal to noise ratio is highest.
7) The resulting dispersion curve is inverted through an iterative process to compute a 1D model representing S-wave velocity versus depth.

Following this procedure, we use the computer program *Grapher* 13.0 by Golden Software to plot the 1D seismic velocity versus depth graphs shown on Plate 2. On these step plots, the horizontal axis represents seismic velocity (increasing to the right) and the vertical axis represents depth (increasing downward). The solid blue line depicts the variations in S-wave velocity versus depth for each sounding.