APPENDIX C

GEOTECHNICAL INVESTIGATION AND PALEONTOLOGICAL RESOURCES RECORDS CHECK

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GEOTECHNICAL INVESTIGATION
PROPOSED PHASE 1 MEDICAL OFFICE BUILDING
ST. JOSEPH HOSPITAL
NEC S. MAIN STREET AND W. STEWART DRIVE
ORANGE, CALIFORNIA

Prepared for:
PMB Orange 2 LLC
3394 Carmel Mountain Road, Suite 200
San Diego, California 92121

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Project No. 2981.I

March 10, 2020

GEOTECHNICAL
PROFESSIONALS INC.

March 10, 2020

PMB Orange 2 LLC
3394 Carmel Mountain Road, Suite 200
San Diego, California 92121

Attention: Mr. Peter Jeong

Subject: Report of Geotechnical Investigation
Proposed Phase 1 Medical Office Building
NEC S. Main Street and W. Stewart Drive
Orange, California
GPI Project No. 2981.I

Dear Mr. Jeong:

Transmitted herewith is our report of geotechnical investigation for the subject project. The report presents our evaluation of the foundation conditions at the site and recommendations for design and construction.

We are providing this report in an electronic format. When requested, we will provide wet signed originals for submittal to regulatory agencies.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have any questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

James E. Harris, G.E.
Principal

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TABLE OF CONTENTS

LIST OF FIGURES

FIGURE NO.

APPENDIX A

APPENDIX B

APPENDIX C

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed Phase 1 Medical Office Building (MOB) project located at St. Joseph Hospital in Orange, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed project will consist of a concrete podium type building consisting of 3 levels above grade and 4 to 5 levels of below grade parking. The ground level will consist tenant space, parking entry ramps and Columbia Street extending through the building. The upper 2 levels will consist of tenant space for the MOB. The below grade levels will consist of parking garage. The building will cover a footprint of approximately 44,400 square feet (sf).

Current plans indicate that the structure will be constructed 4 to 5-levels of below grade concrete parking, 1-level of a concrete podium, and 2 levels of a steel frame structure for the MOB over the podium. Based on the number of subterranean levels, we have assumed foundations will extend up to to 53 feet below existing grades.

The proposed site configuration is shown on the site plan, Figure 2.

Based on our experience with similar projects, we assume maximum column loads of approximately 700 to 1,000 kips for the MOB/parking garage.

Our recommendations are based upon the above structural and finish grade information. We should be notified if the actual loads and/or grades differ or change during the project design to either confirm or modify our recommendations. Also, when the project shoring and foundation plans become available, we should be provided with a copy for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical conditions at the site as they relate to the design and construction of the proposed development. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork and design of shoring and foundations.

2.0 SCOPE OF WORK

Our scope of work for this investigation consisted of review of existing information, field exploration, laboratory testing, engineering analysis, and the preparation of this report.

Our field exploration consisted of five exploratory borings. The locations of the subsurface explorations are shown on the Site Plan, Figure 2.

The exploratory borings were drilled using truck-mounted hollow-stem auger drilling equipment to depths of 81 to 101 feet below existing site grades. Details of the drilling and Logs of Borings are presented in Appendix A.

Laboratory soil tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, grain size distribution, fines content, shear strength, consolidation, expansion potential, and soil corrosivity. Laboratory testing procedures and results are summarized in Appendix B.

Soil corrosivity testing was performed by HDR under subcontract to GPI. Their test results are presented in Appendix C.

Terra Geosciences performed seismic shear-wave survey to assess the average shear wave velocity of the subsurface soils. The results of the testing and the report by Terra Geosciences are presented in Appendix C.

Engineering evaluations were performed to provide earthwork criteria, foundation and slab design parameters and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is located at the northeast corner of S. Main Street and W. Steward Drive within the western portion of St. Joseph Hospital in Orange, California. The site is currently occupied by a small single-story motel building, a two-story hospital building, a single-story community clinic building, parking lots, and a street into an existing parking structure. The 3-level parking garage is located along nearly the entire eastern side of the proposed MOB

The site is bounded on the north by an empty lot with small abandoned buildings, on the west by S. Main Street, on the south side by W. Stewart Drive, and on the east by an existing parking garage, a dental office parking lot, and drives within the hospital.

The site topography is relatively flat with ground surface elevations ranging from approximately +155 to +159 feet across the site from south to north.

3.2 SUBSURFACE SOIL CONDITIONS

Our field investigation disclosed a subsurface profile consisting of undocumented fill soils overlying natural soils. Detailed descriptions of the conditions encountered are shown on the Logs of Borings in Appendix A.

We encountered undocumented fill soils to depths of approximately 5 feet or less in our exploratory borings. The fill soils consisted of silty sands, clays, silty clays, and sandy clays. Moisture contents of the fills were observed to be slightly moist to moist.

The natural soils encountered consisted of interbedded layers of firm to stiff sandy clay, silty clays, and clayey silts along with medium dense to dense clayey sands, silty sands and sands. At a depth of approximately 40 to 50 feet, a thick layer of dense to very dense sandy soil was encountered which contained varying amounts of gravel and cobbles. The borings at the southern portion of the site generally contained more gravel and cobbles. In general, the sand layer extended to a depth of 60 to 70 feet where a layer of hard clay was encountered. Interbedded layers of hard clays and very dense sands were extended to the bottom of our explorations. The sandy soils are generally dry to slightly moist. The clays are generally moist to very moist. At depths of 40 to 50 feet near the planned excavation bottoms, the sandy soils vary from medium dense to very dense and, in general, the clayey soils vary from stiff to hard. These natural soils near foundation level generally exhibit moderate to high strength and low compressibility characteristics. Detailed descriptions of the soils are shown on the Logs of Borings in Appendix A.

3.3 GROUNDWATER AND CAVING

Groundwater was encountered at 98 feet deep in one of our borings (B-4). Data published by the State of California indicates that groundwater in the site vicinity is greater than 40 feet below the existing site grades (Reference 1).

Caving was not observed within the small diameter explorations. However, caving and raveling of the dry to slightly moist sandy deposits should be expected.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. The proposed structure can be supported on spread footings with a slab-on-grade provided the geotechnical constraints discussed below are mitigated. The most significant geotechnical issues that will affect the design and construction of the proposed structure are as follows:

- The planned excavation for the subterranean parking levels will remove the undocumented fills across the site. Details are presented in the "Earthwork" sections of this report.

- Based on limited site access, shoring will be required during excavation of the basement level. Intermittent layers of very dense layers of sands with gravels and cobbles exist in the soil profile. Driven or vibrated soldier piles may not be a feasible or economical alternative to drilled holes. The shoring contractor should evaluate the subsurface conditions when planning the installation methods for soldier piles and tieback anchors.

- Groundwater was encountered in one of our explorations at a depth of more than 40 feet below the planned excavations for the subterranean level of the building. We do not anticipate groundwater to adversely impact excavation bottoms, floor slabs, or basement walls.

- Footings for the building can be supported directly on natural soils at the lower subterranean level. Footings for at-grade minor structures can be supported on properly compacted fills.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC DESIGN

4.2.1 General

The site is located in a seismically active area typical of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed building will be in accordance with the California Building Code (CBC), 2019 edition. A Site Class C may be used for the seismic design. This determination is based on a seismic shear wave survey performed by Terra Geosciences under subcontract to GPI. The average shear wave velocity of approximately 1290 feet/second was measured in the subsurface soils to approximately 100 feet below foundation levels. Appendix C contains the results of the seismic shear wave survey.

The seismic code values can be obtained directly from the tables in the building code using the above values and appropriate SEAOC/OSHPD website (Reference 2). The Project Structural Engineer should determine the seismic design method.

4.2.2 Strong Ground Motion Potential

Based on published information presented in Reference 3, the most significant faults in the proximity of the site are the San Joaquin Hills and Puente Hills Faults, which are located approximately 6 to 7 miles from the site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the SEAOC/OSHPD website (Reference 2), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.68g for a modal magnitude 6.5 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (Reference 4) and a site coefficient (F_{PGA}) based on site class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years.

The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated sandy soils. Thus, three conditions are required for liquefaction to occur: (1) a sandy soil of loose to medium density; (2) saturated conditions; and (3) rapid, large strain, cyclic loading, normally provided by earthquake motions.

The site is not located within an area shown as having a potential for soil liquefaction in accordance with the Seismic Hazards Mapping Act as shown in the Orange Quadrangle (Reference 5). Soil liquefaction is not likely to occur at the project site because the soils below the historical high groundwater level are dense to very dense.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in the densification of loose to medium dense sandy soils above the groundwater. The sands below the planned excavation level of the subterranean parking are medium dense to very dense. Ground subsidence below the excavation level will not likely occur during a major seismic event.

4.3 SUBSURFACE DRAINAGE

Groundwater was encountered in one of our explorations at a depth of more than 40 feet below the planned excavations for the subterranean level of the building. A historical high groundwater level as provided by the State (Reference 1) indicates a depth of greater than 40 feet below existing grades. Except for limited layers of clayey soils, the soils in our samples were dry to slightly moist above a depth of 90 feet below existing grades. Based on the above information, we do not anticipate groundwater to adversely impact excavation bottoms, floor slabs, or basement walls.

4.4 EARTHWORK

The earthwork anticipated at the project site will consist of demolition of existing buildings, improvements and pavements, clearing and grubbing, excavation for the subterranean parking, subgrade preparation, and the placement and compaction of fill.

4.4.1 Clearing and Grubbing

Prior to grading, performing excavations, or constructing the proposed improvements, the areas to be developed should be stripped of vegetation and cleared of existing structures, debris, and pavements. Buried obstructions, such as footings, abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during the clearing operation, including organic topsoil, should be removed from the site. If approved by the owner and regulatory agency, inert demolition debris, such as concrete and asphalt may be crushed for reuse in engineered fills outside the planned building areas in accordance with the criteria presented in the "Materials for Fill" section of this report.

If cesspools or septic systems are encountered during grading, they should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to further grading.

4.4.2 Excavations

Excavations at this site will include the subterranean parking excavation, removals of undocumented fills if not removed by the excavation, footing excavations, and trenching for new utility lines.

Based on the preliminary project plans, the minor amount of fill soils within the building limits will be removed during the planned excavation for the subterranean parking levels. For planning purposes, removals below the proposed pad elevation are not required unless the soils are disturbed. The actual depths of removal should be determined in the field during grading by a representative of GPI.

For minor at-grade supported structures, such as screen walls, canopies, or short retaining walls, the existing fills should be removed and the footings should be underlain by at least 2 feet of properly compacted fill. For pavement and hardscape outside the building, the existing soils within at least 1 foot of the existing or finished grade, whichever is lower, should be overexcavated and replaced as properly compacted fill. The actual depths of removals should be determined in the field during grading by a representative of GPI.

Where space is available, the removals for at-grade structures should extend laterally beyond the edge of footing a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e. a 1:1 projection below the edge of footings).

Where not removed by the aforementioned excavations, existing undocumented utility trench backfill remaining below new foundation areas should be removed and replaced as properly compacted fill. This is especially important for deeper fills such as existing sewers and storm drains. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. The removal should extend laterally 1-foot beyond both sides of the pipe. The actual limits of removal will be confirmed in the field. We recommend that known utilities be shown on the grading plan.

Temporary construction excavations may be made vertically without shoring to a depth of 4 feet below the adjacent grade. Though not anticipated due to site constraints, for cuts up to 10 feet, the slopes should be properly shored or sloped back to at least 1:1 (horizontal to vertical) or flatter. For cuts up to 20 feet, the slopes should be properly shored or sloped back to at least 1½:1 or flatter. The inclination is measured from the top to toe of slope, and we do not recommend incorporating a vertical cut at the base of the slope. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of the adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards.

4.4.3 Subgrade Preparation

After removals are complete and prior to placing fills or construction of proposed at-grade structures, the subgrade soils should be scarified to a depth of 12 inches, moisture-conditioned, and compacted to dry densities equal to at least 90 percent of the maximum dry density (95 percent for sandy soils), determined in accordance with ASTM D 1557.

In areas to receive pavements (outside of the structure), the top 12 inches below the pavement base should be scarified, moisture-conditioned, and compacted to a minimum of 90 percent (95 percent for sandy soils) of the maximum dry density in accordance with ASTM D 1557.

4.4.4 Material for Fill

Soils available from on-site excavations, less debris or organic matter, will be suitable for re-use in fills. Clays and silts should not be used for fills behind retaining walls or directly beneath exterior flatwork. Retaining wall backfill and soils within 1-foot of finished grade for exterior hardscape and flatwork should consist of on-site or imported granular and be relatively non-expansive soils as described below. This recommendation is presented graphically in Figure 3.

While not anticipated for the project, imported fill material should be predominately granular (containing no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. of 20 or less). Import or on-site materials used in compacted fills should not contain particles larger than 6 inches in diameter. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain at least one sack of cement per cubic yard and have a maximum slump of 5 inches. When set, such a mix typically has the consistency of compacted soil.

From a geotechnical engineering standpoint, asphalt concrete or portland cement concrete can be incorporated into fills provided that they are crushed to the consistency of aggregate base and thoroughly blended with enough soil to form a well-graded mixture (typically a 3:1 soil to debris ratio). Such material should not be placed within landscape areas. Approval from the owner and City of Orange should be obtained prior to use of the inert materials within the building area.

In areas where open-graded gravel, such as pea gravel or ¾-inch crush rock, is placed, the gravel should be separated from the on-site soils with a suitable non-woven filter fabric, such as Mirafi 140N or equivalent. The purpose of the filter fabric is to reduce the potential for soil particles to migrate into the void spacing of the gravel.

4.4.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent (95 percent for sandy soils) of the maximum dry density, determined in accordance with ASTM D1557. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-8 inches
Heavy loaders or vibratory rollers	8-12 inches

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

Fills consisting of the on-site clays should be placed at a moisture content of 1 to 3 percent over the optimum moisture content in order to achieve the required compaction and reduce the potential for future swelling. On-site or imported granular fills should be placed at a moisture content of 0 to 2 percent over the optimum moisture content. The moisture content of the sandy soils encountered in the upper 50 feet of the explorations at the site was generally below the optimum moisture content. The clayey soils encountered in the upper 50 feet of the explorations at the site are typically over optimum conditions.

Once moisture conditioned and properly compacted, the exposed clayey soils should not be allowed to dry out prior to covering. A representative of GPI should confirm the moisture content of the subgrade soils immediately prior to placement of concrete or additional fill.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

4.4.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. Neither shrinkage nor subsidence is anticipated to be a major factor on the project because of the significant soil export. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be verified during grading.

4.4.7 Trench/Wall Backfill

Utility trench and wall backfill, consisting of the on-site materials (trenches only) or imported sand, should be mechanically compacted in lifts. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Moisture conditioning of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. GPI should observe and test trench and wall backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain one sack of cement per cubic yard and have a maximum slump of 5 inches. Within building areas, the slurry should contain two sacks of cement per cubic yard.

4.4.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

4.5 FOUNDATIONS

4.5.1 Foundation Type

The proposed structure may be supported on conventional isolated and/or continuous shallow footings, provided the subsurface soils are prepared in accordance with the recommendations given in this report. All building footings should be supported on competent natural soils. Footing bottoms should be moistened immediately prior to placement of concrete.

4.5.2 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the natural soils, a static allowable net bearing pressure of up to 5,500 pounds per square foot (psf) may be used for both continuous footings and isolated column footings bearing.

The bearing pressures provided below are for dead-load-plus-live-load, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

For any at-grade minor structures, a static allowable net bearing pressure of up to 2,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings bearing on properly compacted fill consisting of on-site soils.

4.5.3 Minimum Footing Width and Embedment

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure for the building.

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTNG* EMBEDMENT (inches)
5,500	120	24
5,000	96	24
4,500	72	24
4,000	60	24
2,500	24	24
2,000	24	18
1,500	18	18

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction.

A minimum footing width of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf.

4.5.4 Estimated Settlements

We calculated settlements based on the assumed structural loads. Based on our analyses, the total static settlement under a maximum column load of 1000 kips supported on natural soils at 40 to 50 feet below existing grades, is expected to be on the order of 1-inch or less. Differential settlement between similarly loaded adjacent footings is anticipated to be on the order of ½-inch.

Total settlements of less than 1-inch are anticipated for any minor at-grade structures supported on natural soils or compacted fill derived from on-site soils. Differential settlement between similarly loaded adjacent footings is anticipated to be on the order of ½-inch.

The above estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations. If the structural loads change as the design of the project progresses, we should be notified in order to confirm the estimated settlement values provided above.

4.5.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the footings are poured tight against competent natural soils or compacted fill. These values may be used in combination without reduction.

4.5.6 Foundation Concrete

Laboratory testing by HDR (Appendix C) on samples provided by GPI indicates soluble sulfate contents of 4 to 142 mg/kg (0.001± to 0.014± percent by weight). For the 2019 CBC, foundation concrete should conform to negligible sulfate exposure per the requirements outlined in ACI 318, Section 4.3.

4.5.7 Footing Excavation Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve all footing excavations.

4.6 RETAINING STRUCTURES AND SHORING

Basement walls, cantilever retaining walls, and temporary shoring are planned for the site. The following recommendations are provided for walls up to 15 feet tall and shoring that does not extend more than 50 feet in height. We recommend that conventionally constructed walls be backfilled with sandy (granular) soils.

4.6.1 Basement and Retaining Walls

Active pressure may be used in the design of the subterranean walls if the total movement of the wall is sufficient to mobilize the active pressure (yielding at least ½-inch laterally in 10 feet of wall height). For cantilever walls with level, drained backfill comprised of granular soils, the magnitude of active pressures is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). For cantilever walls retaining level, drained, undisturbed native soils, the magnitude of active pressures is equivalent to the pressures imposed by a fluid weighing 45 pcf.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures imposed by a fluid weighing 65 pounds per cubic foot should be used for <u>drained, natural</u> soils.

The following pressures should be used to design the basement walls if they are waterproofed and designed to resist hydrostatic pressure below the design groundwater elevation. At-rest pressures imposed by a fluid weighing 94 pounds per cubic foot should be used for <u>undrained natural</u> soils for basement walls.

To account for seismic load an additional lateral earth pressure equal to 25 pcf (equivalent fluid pressure distribution) should be added to the above active pressures. If the walls are designed using the above at-rest pressure, the added seismic load plus at-rest pressures can be limited to 70 pcf for <u>drained natural</u> soils. The added seismic load plus at-rest pressures can be limited to 96 pcf for <u>undrained natural</u> soils.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. Surcharge loads from the adjacent parking structure needs to be considered if the proposed basement wall extends below the zone of influence of the adjacent foundations. The zone of influence can be defined as the area below an imaginary 1:1 line extending downwards from the bottom of the nearest footing.

In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

Construction equipment, such as cranes, concrete trucks, or loaders can impose lateral surcharge loads if they are supported adjacent to the basement walls (or shoring). Therefore, surcharge effects from such equipment will need to be evaluated on a case-by-case basis and, if needed, the walls locally reinforced to support the surcharge from such loads.

The recommended pressures for the drained condition are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydrostatic pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall. For retaining walls constructed adjacent to temporary shoring, a composite geotextile drain may be used with a manifold-type collection drain at the design groundwater level. A representative of GPI should observe and approve wall drains prior to placement of wall backfill.

The Structural Engineer should specify the use of select, granular wall backfill on the plans for walls that are to be conventionally backfilled. Wall footings should be designed as discussed in the "Foundations" section.

4.6.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, such as along the property limits, shoring will be required. Based on current plans, shoring is anticipated along all sides of the project site. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied-back with earth anchors. The tie-back anchors will require permission and be subject to limitations from the adjacent property owners and the City of Orange. Utilities in the adjacent streets should be considered when planning the shoring. Rakers providing support to the soldier piles from inside the excavation would be an option if tie-backs are not allowed in any areas of the site.

The shoring contractor should evaluate the subsurface conditions when planning the installation methods for soldier piles and tieback anchors. Because of intermittent layers of very dense layers of sands with gravels and cobbles in the upper 80 feet of the soil profile, driven or vibrated soldier piles may not be a feasible or economical alternative to drilled holes. The presence of very dense sands as well as gravel and cobbles should be considered when evaluating the alternatives for soldier piles.

For cantilever shoring with level backfill consisting of the on-site soils, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). For restrained shoring, such as soldier piles with tied-back earth anchors, a trapezoidal apparent earth pressure envelope may be used. The magnitude of the maximum pressure may be taken as 28H in pounds per square foot (psf) where H is the total height of the excavation being shored for the basement walls. The trapezoidal distribution is shown on Figure 4, Lateral

Earth Pressures for Tie-Back Shoring. It should be noted that the provided lateral earth pressures assume a fully drained condition and do not include hydrostatic pressures.

In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected.

Shoring should also be designed for adjacent building loads of the parking structure and construction equipment in a similar manner as basement walls.

For design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the excavation may be taken to be 600 pounds per square foot at the excavated surface, up to a maximum of 6,000 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils. While not anticipated due to the potential for significant cobbles at the excavation level, if the soldier piles are driven or vibrated into place, the design width of the soldier piles (effective pile diameter) used in calculations should be equal to the actual width of the flange of the soldier piles.

While not anticipated to be feasible, driving of soldier piles to improve production or minimize ground vibration should only allow predrilling down to the design elevation of the excavation bottom. A continuous flight auger should be utilized to enable reversing the auger to minimize the removal of soil during the process. If soil is removed during the predrilling process, the resulting void should be backfilled with 1½ sack sand-cement slurry. The diameter of the auger used for predrilling should not exceed 80 percent of the maximum depth of the soldier pile beam section.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier pile and the retained earth may be taken as 0.35. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean mix concrete and the retained earth. In addition, provided the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 500 pounds per square foot.

Continuous lagging will be required between the soldier piles. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. We recommend that the voids between the lagging and retained earth be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot, provided the soldier beam spacing is 8 feet or less.

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation. The anchors should extend at least 20 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities. The capacities of anchors should be determined by testing of the initial anchors as outlined in a following paragraph. For design purposes, it may be estimated that conventional drilled cast-in-place friction anchors will develop an average friction value of 700 pounds per square foot. Post-grouted anchors typically obtain greater capacities compared to gravity grouted anchors. In general, the obtained capacity of post-grouted tie-back anchors is primarily a function of construction methods and experience of the specialty contractor along with local site conditions. The capacity of tie-back anchors should be determined through a performance specification. Ultimately, it is the contractor's responsibility to obtain the required pullout capacity, which may require extensive post grouting and/or field modifications. A design friction value of 2,000 pounds per square foot for post-grout anchors has been used by contractors on other projects. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on-center, group action reduction in the capacity of the anchors need not be considered.

The anchors may be installed at angles of 15 to 45 degrees below the horizontal. Caving of the anchor holes should be prevented with the installation method selected. For friction gravity, grouted anchors (non post-grouted), the anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. The annular space around the tie-back tendons should not be backfilled until after anchor testing. If caving is a concern in the sandy deposits, the portion within the active wedge may be backfilled with sand and only enough cement to allow placement by pumping. Additional tendons may be required if the active wedge portion is filled to complete the 200 percent tests discussed below.

At least 10 percent of the total anchors should be selected for quick 200 percent tests. At least one anchor per row should be tested for 24 hours. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the post grouting or anchor length should be increased until satisfactory test results are obtained. When the extent of the shoring program is known, we should review the recommended test program and make modifications as necessary. For the 24-hour 200 percent tests, the total deflection during loading should not exceed 12 inches. The deflection after the 200 percent test load has been applied should not exceed 0.75-inch during the 24-hour period. If the anchor movement after the 200 percent load has been applied for 10 hours is less than 0.5 inch, and the movement over the previous 4 hours has been less than 0.1-inch, the test may be terminated. For the quick 200 percent tests, the total deflection should not exceed 12 inches. The deflection after the 200 percent test load has been applied should not exceed 0.25 inch during a 30-minute period.

The remaining anchors should be pretested to at least 150 percent of the design load. The total deflection during the test should not exceed 12 inches. The rate of creep under the 150 percent load should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading. After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10 percent from the design load, the load should be reset until the target load is achieved.

Anchor testing should be performed by the contractor and observed by GPI. The contractor shall provide the necessary test equipment, including an independent fixed reference point (i.e., tripod) for placement of the dial gage for measuring anchor deflections during tensioning. Prior to testing, the contractor shall supply current calibration records of the hydraulic jack to be used for testing. Calibration records should be signed by a California registered professional engineer and be within 3 months prior of the start of testing.

It is difficult to accurately predict the amount of deflection of the shored embankment. It should be realized, however, that some deflection will occur. Adjacent to city right-of-way, the shoring should be designed to limit deflection to 1-inch. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures. We recommend limiting the lateral deflection of shoring adjacent to the parking structure or other buildings to ½-inch.

Driven/vibrated soldier piles, while not anticipated at this project, should be limited to areas beyond 20 feet from existing buildings, and to a greater distance where adjacent structures appear to be sensitive to vibration or settlement. Ground vibrations could be monitored when driving/vibrating soldier piles adjacent to sensitive structures. A seismograph should be used to measure peak particle velocities (PPV) at the ground surface of the structures of concern. We suggest a maximum allowable PPV of 0.5 inches per second be used as a threshold value unless a lower value is required by the adjacent property owners or hospital operations. Measures should be taken to reduce vibrations if PPV limits are exceeded. Such measures could include altering the predrilling methods or changing to the installation of the soldier piles in a drilled and grouted hole.

In areas of the site where tie-back anchors for the temporary shoring will not be feasible, an option would be the use of rakers to support the temporary shoring. Based on the characteristics of the in-place soils at the planned subgrade level determined during our initial investigation, we recommend an allowable bearing pressure of 3,000 pound per square foot (psf) for the raker footings with a minimum embedment of 18 inches. Based on our analyses, the same frictional capacity values can be used for resisting upward components of the raker loads as recommended for resisting downward loads. Raker footing excavations should be cleaned of loose soils and observed by a representative of GPI prior to placing concrete.

We recommend performing a detailed survey of the improvements to be supported above the planned shoring prior to and during the shoring installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles. We suggest weekly readings during the excavation and for the first three weeks after achieving the bottom of the excavation. After that time, the readings should be performed every other week until the completion of the basement walls.

4.7 CONCRETE FLOOR SLABS

Slab-on-grade floors should be supported on a non-expansive (Expansion Index of 20 or less), undisturbed natural or compacted granular soils as discussed in the “Placement and Compaction of Fill” section.

Although not anticipated for the subterranean parking level, a moisture vapor retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (wood, vinyl, tile, etc.). Polyolefin in 15-mil thickness should be covered by a layer of clean sand (less than 5 percent by weight passing the No. 200 sieve) having a minimum thickness of 2 inches. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water-cement ratio for the concrete used for the floor slab, effective sealing of joints and edges (particularly at pipe penetrations) as well as excess moisture in the concrete. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of the floor surface prior to placing moisture-sensitive floor coverings.

4.8 CORROSION

Soil corrosivity testing was performed by HDR under subcontract to GPI. The corrosivity test results are presented in Appendix B. The on-site soils should be considered moderately corrosive to buried metals. If additional corrosion consultation is required, a corrosion engineer such as HDR should be consulted.

4.9 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on imported non-expansive compacted fill if differential heave is not acceptable. The use of clayey soils within the upper 12 inches of exterior flatwork subgrade is not recommended. Prior to placement of concrete, the subgrade should be prepared as recommended in “Subgrade Preparation” section.

4.10 STORMWATER INFILTRATION

Stormwater infiltration in soils retained by basements walls should not be permitted. Infiltration at a sufficient depth below the bottom of the foundations or below the basement walls would be required. The sandy soils encountered during our investigation directly below the proposed foundation level have characteristics, which are generally suitable for on-site subsurface infiltration of stormwater. However, there is a consistent layer of hard clay at a depth of approximately 15 to 25 feet below the foundation level. This clay material will likely be relatively impermeable and will cause mounding of groundwater within the influence of foundations. Extending the infiltration beyond the clay layer with dry wells is not feasible due to the measured groundwater depth of 98 feet and the County requirement to not infiltrate within 10 feet of the seasonally high groundwater (Reference 6). For this reason, it is our opinion that stormwater infiltration at the site is not considered feasible for this project.

4.11 PAVED AREAS

Preliminary pavement design has been based on an assumed R-value of 10 for the clayey near-surface soils. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of the existing soils. The subgrade soil conditions will need to be confirmed at the conclusion of rough grading.

		SECTION THICKNESS (inches)	
PAVEMENT AREA	**TRAFFIC INDEX**	**ASPHALT/PORTLAND CONCRETE**	**AGGREGATE BASE COURSE**
Asphalt Concrete			
Automobile Parking	4.0	3.0	6
Automobile Drives	5.0	3.0	9
Truck Drives	6.0	3.0	13
Portland Cement Concrete			
Automobile Parking	4.0	6.0	4
Automobile Drives	5.0	6.5	4
Truck Drives	6.0	7.0	4
Parking Structure	5.0	5.0	---

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation".

The pavement base course should be compacted to at least 95 percent of the maximum dry density (ASTM D 1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter-inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials (except Processed Miscellaneous Base).

The above recommendations are based on the assumption that the base course will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course and subgrade, which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.12 SURFACE DRAINAGE

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings.

4.13 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork and shoring installation during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by PMB Orange 2 LLC and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by GPI during grading, excavation, and foundation construction. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If others perform construction phase services, the client and new geotechnical firm must accept full responsibility for all geotechnical aspects of the project, including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
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REFERENCES

1. California Geological Survey, 1997, Seismic Hazard Zone Report for the Orange 7.5-minute Quadrangle, Orange County, California, Seismic Hazard Zone Report 011.

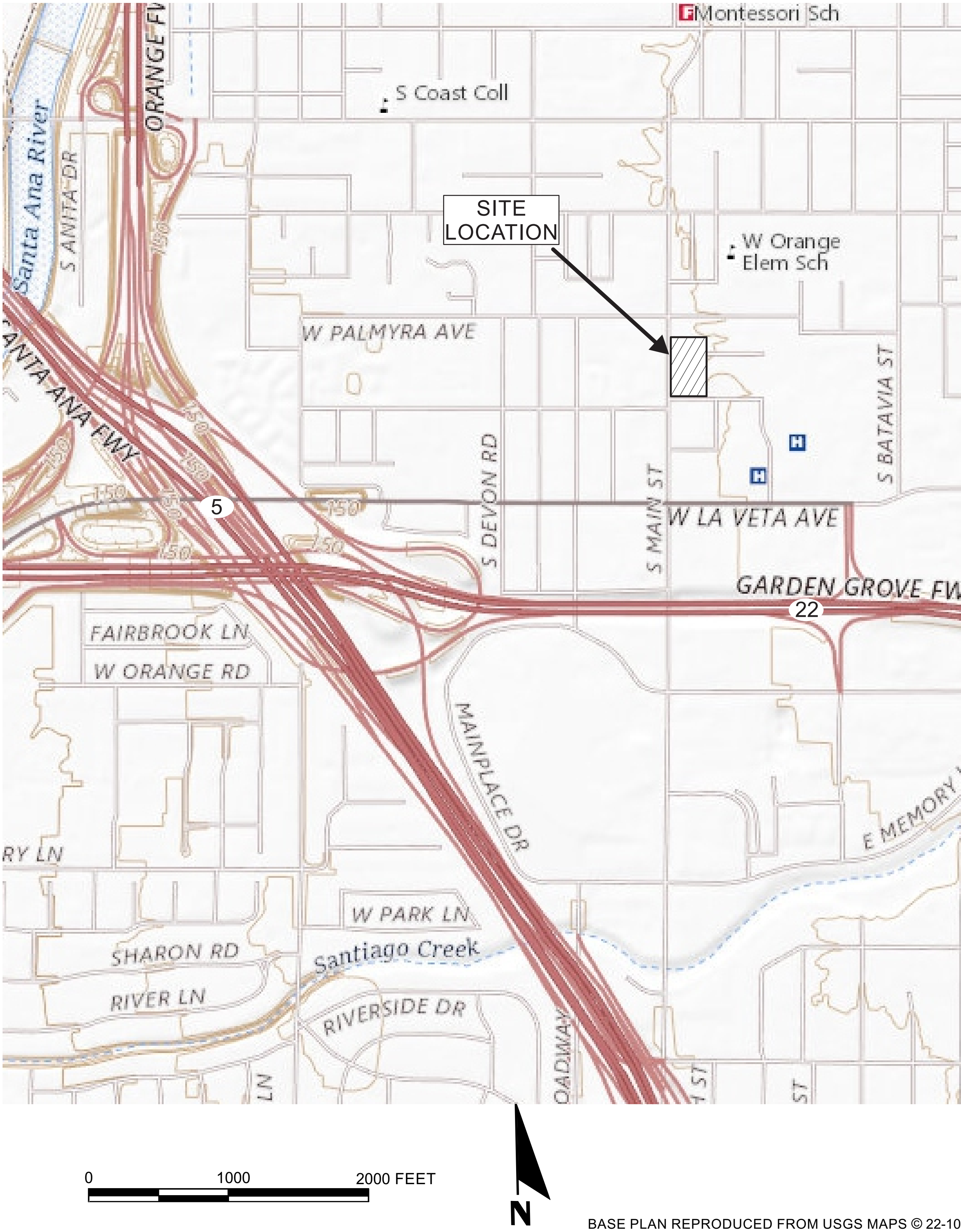
2. Structural Engineers Association of California/Office of Statewide Health Planning and Development, U.S Seismic Design Maps, https://seismicmaps.org/.

3. United States Geological Survey, 2008 National Seismic Hazard Maps, Source Parameters, http://geohazards.usgs.gov/cfusion/hazfaults_search/hf_search_main.cfm.

4. American Society of Civil Engineers (2017), “Minimum Design Loads and Associated Criteria for Buildings and Other Structures,” ASCE/SEI 7-16.

5. California Geological Survey, 1998, Seismic Hazard Zones Map of the Orange 7.5-Minute Quadrangle, Orange County, California,” Official Map, released April 15, 1998.

6. County of Orange, Department of Public Works, “Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs)”, Exhibit 7.III, December 20, 2014



BASE PLAN REPRODUCED FROM USGS MAPS © 22-10-18

GEOTECHNICAL PROFESSIONALS, INC.

PMB ST. JOSEPH MOB

GPI PROJECT NO.: 2981.I

SCALE: 1" = 1000'

SITE LOCATION

FIGURE 1

EXPLANATION

B-1 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

PHASE 1 MEDICAL OFFICE BUILDING

0 100 200 FEET

N

BASE PLAN REPRODUCED FROM GOOGLE MAPS DATED: 2020

GEOTECHNICAL PROFESSIONALS, INC.

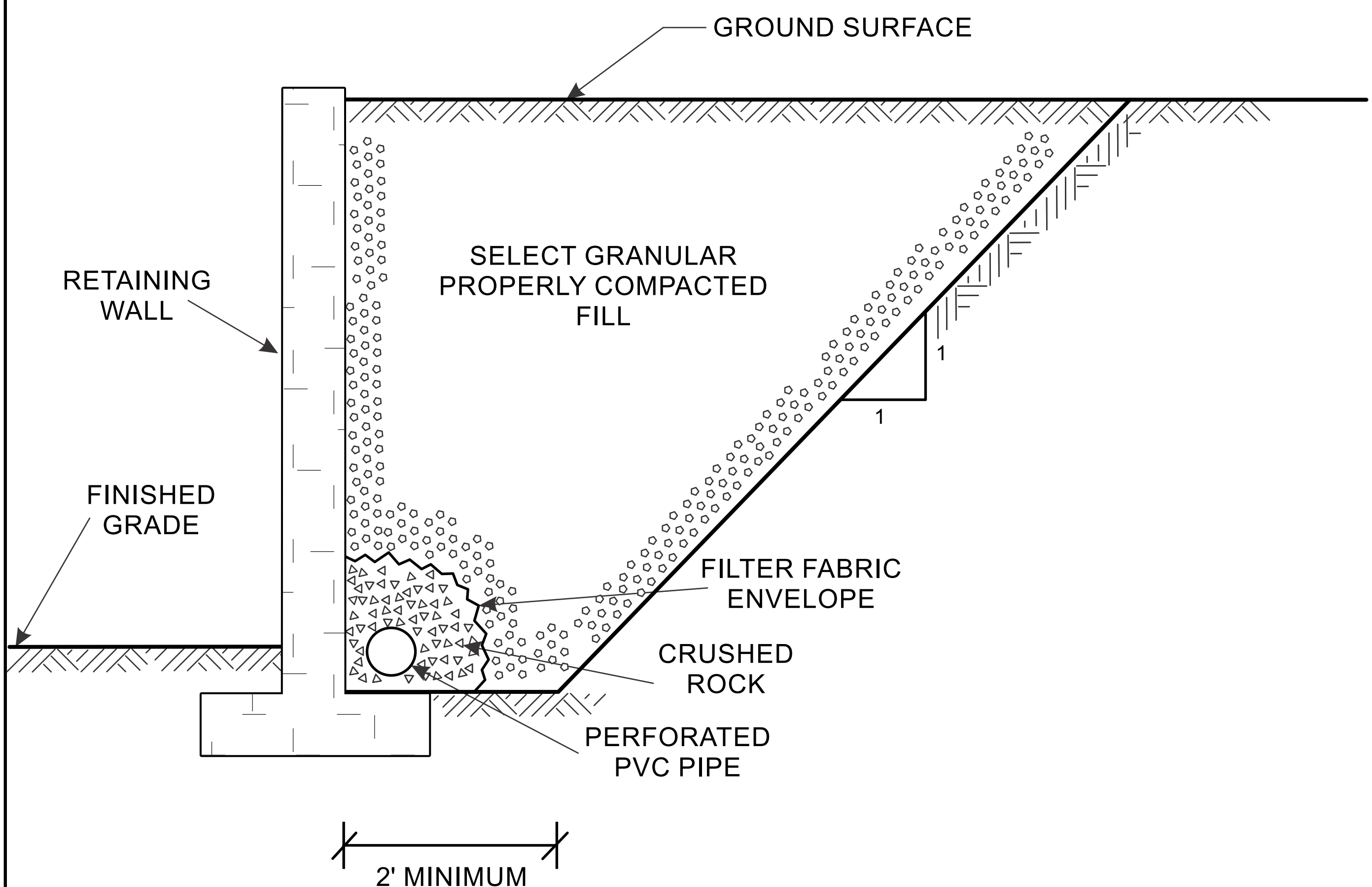
PMB ST. JOSEPH MOB

GPI PROJECT NO.: 2981.I

SCALE: 1" = 100'

SITE PLAN

FIGURE 2



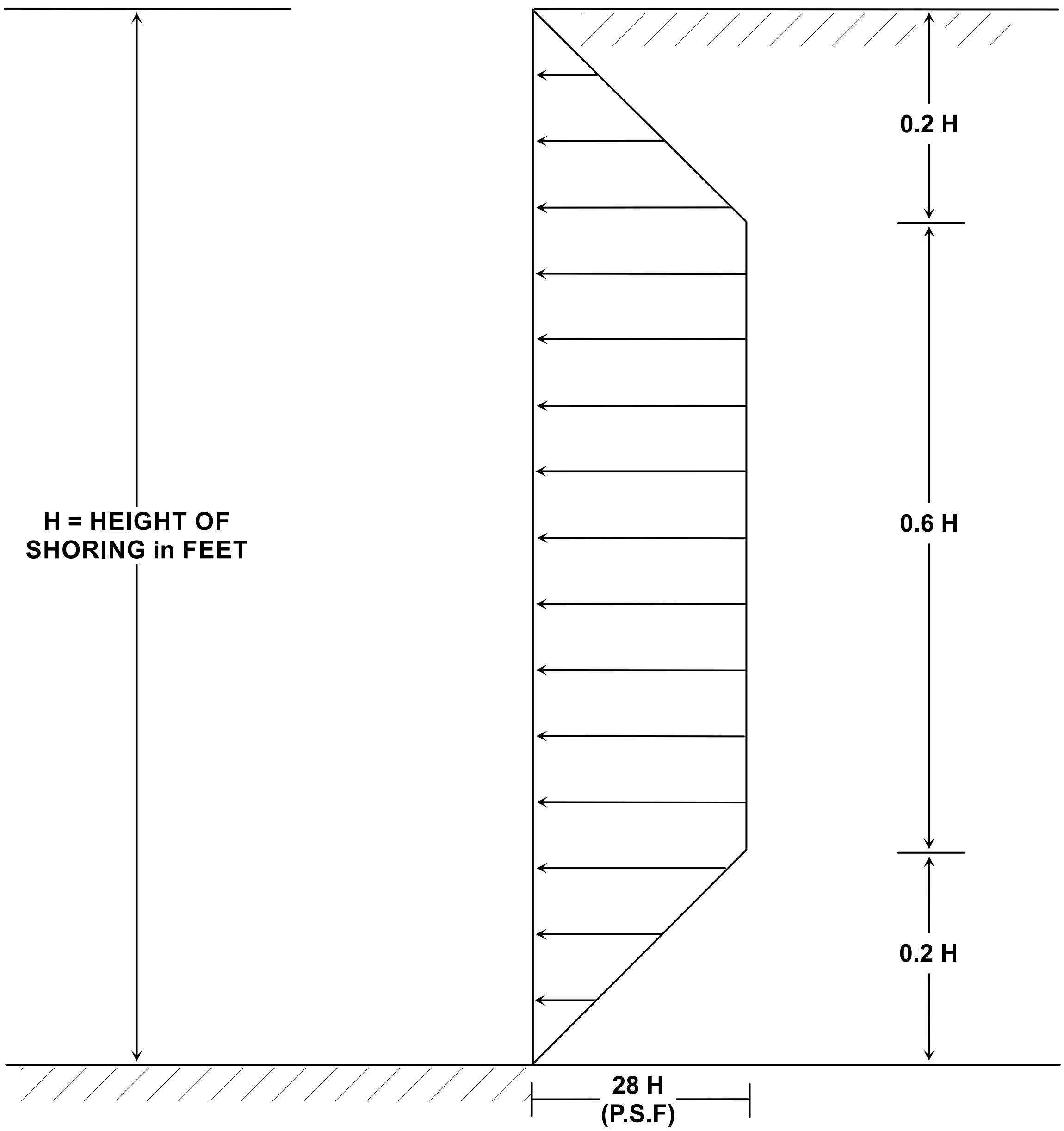
GEOTECHNICAL PROFESSIONALS, INC.

PMB ST. JOSEPH MOB

GPI PROJECT NO.:2981.C	NOT TO SCALE

RETAINING WALL BACKFILL DETAIL

FIGURE 3



GEOTECHNICAL PROFESSIONALS, INC.

PMB ST. JOSEPH MOB

GPI PROJECT NO.: 2981.I

NO SCALE

LATERAL EARTH PRESSURES FOR TIE-BACK SHORING

FIGURE 4

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions for the site were investigated by drilling and sampling five exploratory borings. The borings were advanced to depths of 81 to 101 feet below the existing ground surface. The locations of the explorations are shown on the Site Plan, Figure 2.

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D 3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D 6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches, employing the “free-fall” hammer described above. After an initial seating drive of 6 inches, the number of blows needed to drive the sampler into the soil a depth of 12 inches was recorded as the penetration resistance. These values are the raw uncorrected blowcounts.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 through A-5 in this appendix.

Upon completion of the sampling of hollow-stem auger borings, the holes were backfilled with the excavated soils and patched with cold patch asphalt.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from Google Earth and should be considered very approximate.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	2.5-Inch AC over 14-Inch BASE	
						Fill: **SANDY CLAY (CL)** brown, moist	155
					5	Natural: **CLAYEY SILT (ML)** brown, moist, firm, with sand	
							150
	22.1	92	11	D	10	@ 10 feet, wet	
							145
					15		
							140
					20		
							135
					25		
							130
					30		
							125
					35		
							120

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I
ST. JOSEPH MOB

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
14.5	113	21	D	40	**SILTY CLAY (CL)** orange brown, moist, stiff	
						115
14.6		12	S	45	@ 45 feet, firm	
						110
13.5	111	21	D	50	**SILTY SAND (SM)** orange brown, moist, medium dense	
						105
3.0		32	S	55	**SAND (SP)** light brown, dry, dense	
						100
2.1	106	90/11	S	60	@ 60 feet, very dense, coarse	
						95
				65		
						90
24.9	102	95/11	D	70	**CLAY (CL)** brown, very moist, hard	
						85
				75		
						80

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	16.6	112	52	D	80		**CLAY (CL)** brown, moist, hard	
							Total Depth 81 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	3-Inch over 6-Inch BASE	
						Fill: **SILTY SAND (SM)** brown, moist	155
					5	Natural: **SILTY SAND (SM)/SANDY SILT (ML)** orange brown, slightly moist	150
				B	10		145
					15		140
	1.4	107	48	D	20	**SAND (SP)** light brown, dry, dense, with gravel @ 21 feet, lens of sandy clay	135
					25		130
					30		125
					35		120

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I
ST. JOSEPH MOB

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
13.2	119	61	D	40	**SANDY CLAY (CL)** orange brown, moist, hard	
						115
12.4		19	S	45		
						110
		41	D	50	**SILTY SAND (SM)** orange brown, slightly moist, medium dense, trace clay	
						105
				55		
						100
2.4	104	68	D	60	**SAND (SP)** light brown, dry, dense, trace gravel	
						95
		35	S	65		
					@ 66 feet, lens of silty sand	
						90
20.9	105	74	D	70	**SILTY SAND (SM)** orange brown,wet, dense	
						85
				75		
5.3					@ 75 feet, slightly moist	
						80

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I
ST. JOSEPH MOB

LOG OF BORING NO. B-2

FIGURE A-2

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	22.0	104	68	D	80	**SANDY CLAY (CL)** brown, slightly moist, hard	
						Total Depth 81 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	5-Inch AC	155
					Fill: **CLAY (CL)** brown, moist to very moist	
				5	Natural:**CLAY (CL)** brown, moist	150
				10		145
				15		140
			B	20	**SAND WITH SILT (SP-SM)** brown, moist	135
				25		130
7.8	103	56	D	30	**SAND (SP)** brown, moist, dense, gravel and cobbles	125
				35		120

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					40	@ 40 feet, no recovery	115
	4.9		50/6"	S	45	**SANDY CLAY (CL)** brown, dry to slightly moist, hard	110
			50/3"	D	50	**SAND (SP)** light brown, dry, very dense, with gravel and cobbles	105
					55		100
	2.4	104	50/6"	D	60		95
					65		90
	20.9	105	51	D	70	**CLAY (CL)** orange brown, very moist, hard	85
	5.3		40	S	75	**SILTY SAND (SM)** brown, slightly moist, dense, trace gravel	80

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	22.0	104	54	D	80		**SILTY CLAY (CL)** orange brown, very moist, hard	75
							Total Depth 81 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-5-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	2-Inch AC over 1.5-Inch BASE Fill: **SILTY CLAY (CL)** brown, moist, trace gravel	155
	15.5	100	12	D	5	Natural: **SILTY CLAY (CL)** brown, slightly moist to moist, firm	150
					10		145
					15		140
					20		135
	17.3	111	34	D	25	**CLAY (CL)** orange brown, moist, very stiff	130
					30		125
					35		120

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-6-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
98

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PROJECT NO.: 2981.I

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LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
16.2	114	42	D	40	**CLAY (CL)** orange brown, moist, very stiff	
						115
18.6		12	S	45	**SANDY CLAY (CL)** orange brown, very moist, stiff	
						110
15.7	99	23	D	50	**SILTY SAND (SM)** orange brown, moist, medium dense	
						105
2.7		32	S	55	**SAND (SP)** light brown, dry, dense	
						100
2.4	103	68	D	60		
						95
				65		
						90
21.3	92	92/9"	D	70	**CLAYEY SILT (ML)** brown, wet, hard	
					SILTY SAND (SM) brown, moist, very dense	
						85
				75		
						80

SAMPLE TYPES
C Rock Core
S Standard Split Spoon
D Drive Sample
B Bulk Sample
T Tube Sample

DATE DRILLED:
2-6-20
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
98

GPI

PROJECT NO.: 2981.I
ST. JOSEPH MOB

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
22.6	101	52	D	80	**SILTY CLAY (CL)** orange brown, very moist, hard	
						75
				85		
						70
18.4	103	95/10"	D	90	**SAND WITH SILT (SP-SM)** brown, wet, very dense	
						65
				95		
						60
30.0	92	48	D	100	**CLAY (CL)** orange brown, very moist, hard	
					Total Depth 101 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-6-20
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
98

GPI

PROJECT NO.: 2981.I
ST. JOSEPH MOB

LOG OF BORING NO. B-4

FIGURE A-4

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	2-Inch AC	155
						Fill??: **SANDY CLAY (CL)** brown, moist	
					5	Natural: **SANDY CLAY (CL)** brown, moist	150
				B	10		145
	3.1	101	33	D	15	**SAND (SP)** light brown, dry, medium dense	140
					20		135
					25		130
					30		125
	4.2		95/8"	D	35	**CLAYEY SAND (SC)** brown, dry, very dense, with gravel and cobble	120

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-6-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-5

FIGURE A-5

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
		50/6"	D	40	@ 40 feet, no recovery	115
3.9		50/4"	D	45	**SILTY SAND (SM)** brown,dry very dense, with gravel and cobbles	110
3.1	115	50/4"	D	50	**SAND (SP)** brown, dry to slightly moist, very dense, with gravel and cobbles	105
				55		100
3.5	103	89	D	60		95
2.6		42	S	65		90
2.4	114	50/6"	D	70		85
				75		80

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 2-6-20

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-5

FIGURE A-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	6.3	91	50/6"	D	80		**SILTY SAND (SM)** brown, slightly moist, very hard, with gravel and cobbles	75
							Total Depth 81 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
2-6-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2981.I

ST. JOSEPH MOB

LOG OF BORING NO. B-5

FIGURE A-5

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring and SPT samples from the borings. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D 2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix B.

PERCENTAGE PASSING NO. 200 SIEVE

Selected soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. A summary of the percentages passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-3	20	Sand with Silt (SP-SM)	7
B-5	15	Sand (SP)	3
B-5	45	Silty Sand (SM)	13

GRAIN SIZE DISTRIBUTION

Samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. The retained material was run through a standard set of sieves in accordance with ASTM D 422. The weight of soil retained on each sieve was recorded and the total dry weight was calculated. The grain size distribution data from the full sieve analyses is presented in Figure B-1. A summary of the percentages passing the No. 200 sieve (ASTM D1140) is presented above.

ATTERBERG LIMITS

Liquid and plastic limits were determined for select cohesive soil samples in accordance with ASTM D 4318. The results of the Atterberg Limits tests are presented in Figure B-2.

DIRECT SHEAR

Direct shear tests were performed on undisturbed samples in accordance with ASTM D 3080. The sample was placed in the shear machine, and pre-selected normal loads were applied. The samples were inundated, allowed to consolidate, and then were sheared to failure at a strain rate of 0.001 to 0.0007 inches per minute. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear test are presented in Figures B-3 to B-9.

CONSOLIDATION

One-dimensional consolidation tests were performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the sample was placed in the consolidometer and loaded to up to 0.4 ksf. Thereafter, the sample was incrementally loaded to a maximum load of up to 25.6 ksf. The sample was inundated at 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to 0.4 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures B-10 to B-13.

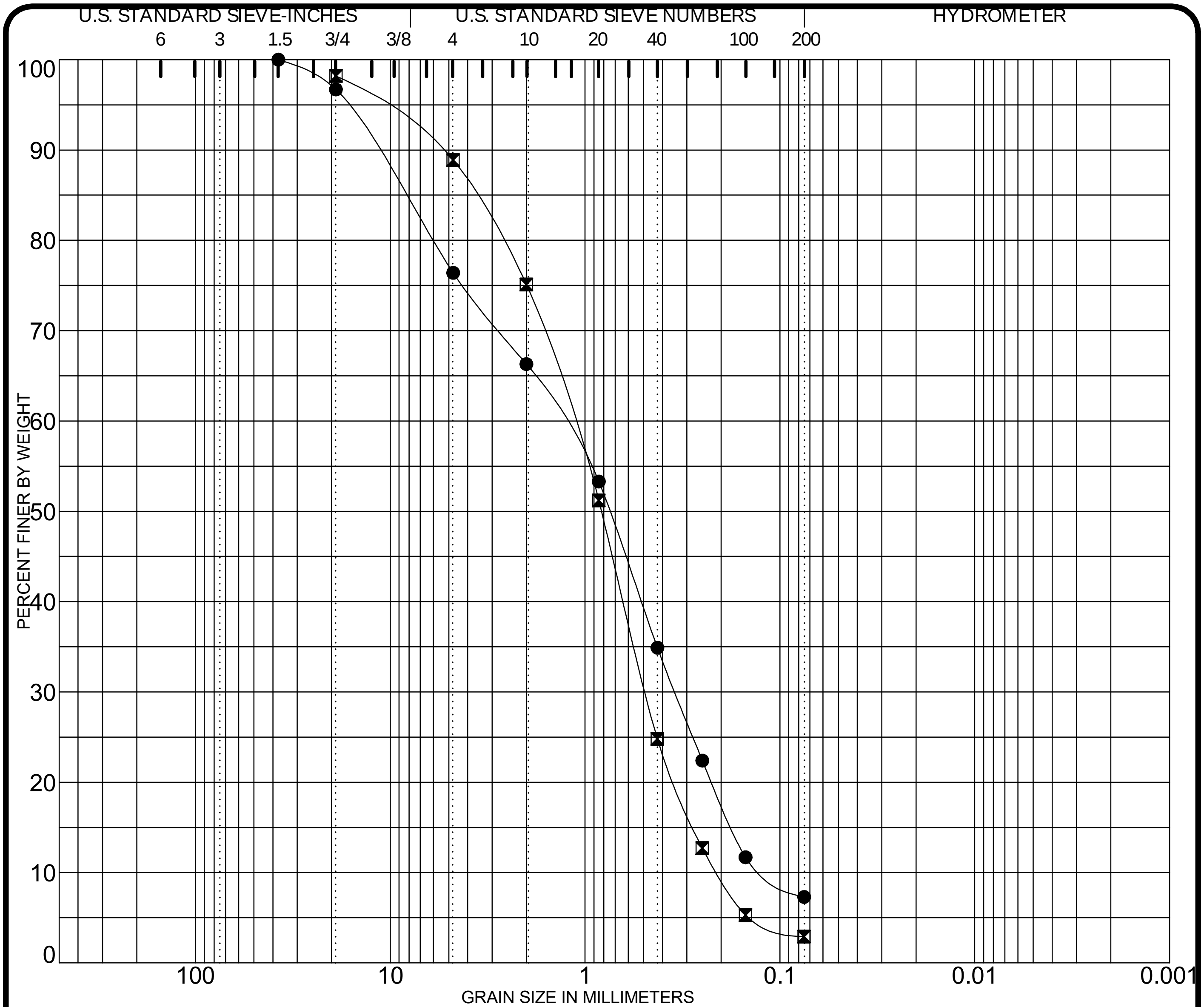
EXPANSION INDEX

An expansion index test was performed on a bulk sample. The test was performed in accordance with ASTM 4289 to assess the expansion potential of on-site soils. The results of the test are summarized below:

BORING/ TEST PIT NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-1	0-4	Sandy Clay (CL)	10
B-4	0-4	Silty Clay (CL)	26

CORROSIVITY

Soil corrosivity testing was performed by HDR on a soil sample provided by GPI. The test results are summarized in Table 1 of this appendix.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-3	20.0	SAND WITH SILT (SP-SM)					0.79	11.5
⧗	B-5	15.0	SAND (SP)					0.98	5.6

	Sample Location		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-3	20.0	37.50	1.32	0.345	0.1148	23.6	69.1	7.3	
⧗	B-5	15.0	19.00	1.16	0.487	0.2075	9.3	86.0	2.9	

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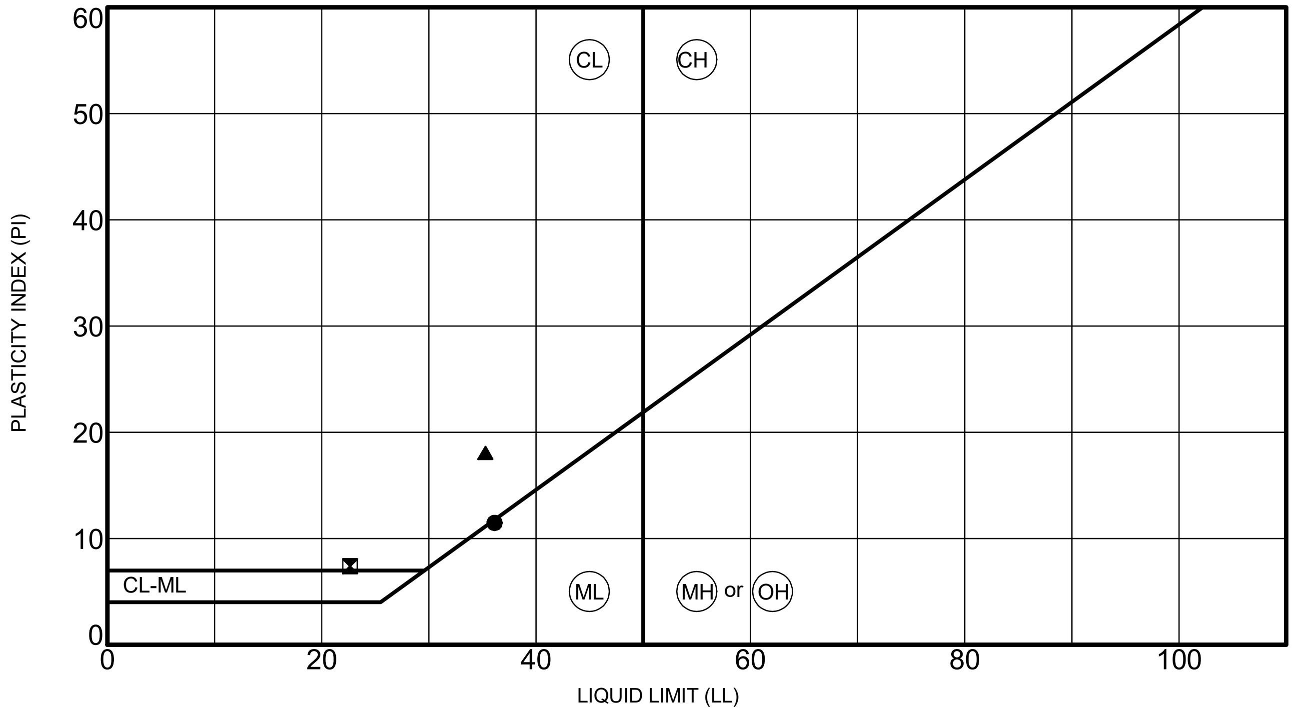
GRAIN SIZE DISTRIBUTION

FIGURE B-1



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	10.0	36	25	11		SANDY SILT (ML)
⧗	B-1	45.0	23	15	7		CLAYEY SILT (ML)
▲	B-4	25.0	35	17	18		CLAY (CL)

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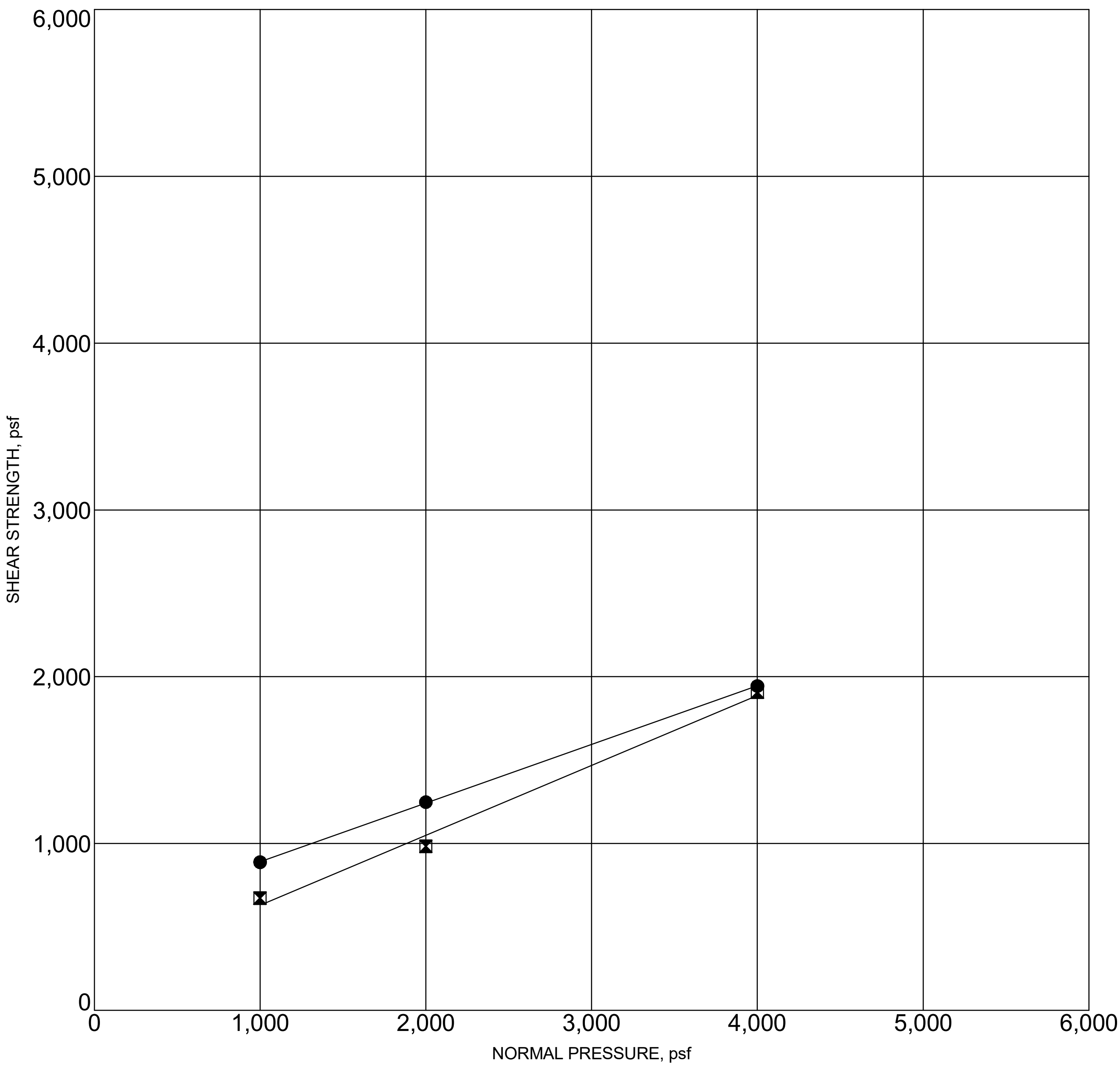
ATTERBERG LIMITS TEST RESULTS

FIGURE B-2



● **PEAK STRENGTH**
Friction Angle= 19 degrees
Cohesion= 540 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 23 degrees
Cohesion= 210 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	10.0	SANDY SILT (ML)	92	22.1

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PROJECT NO.: 2981.I

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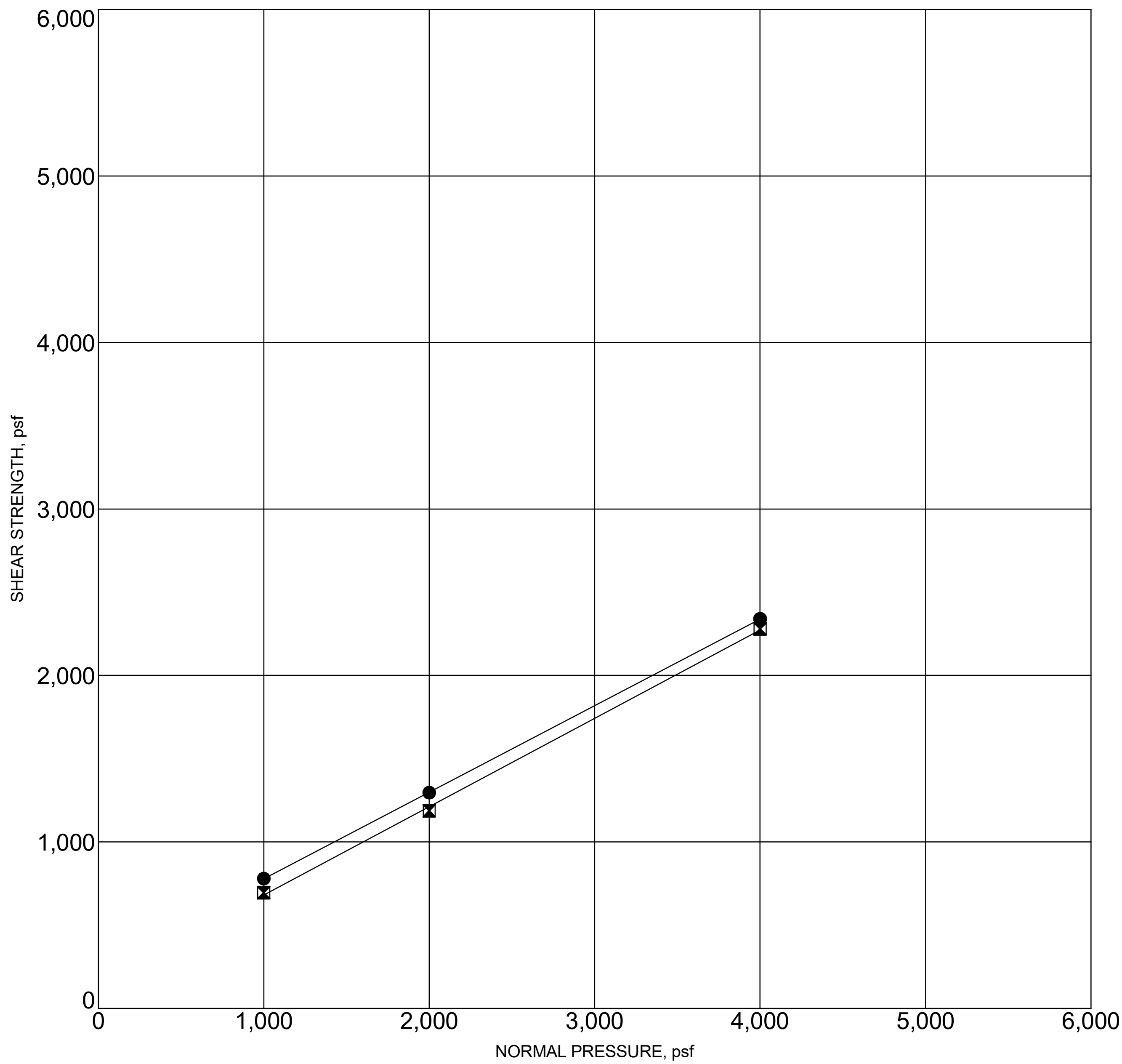
DIRECT SHEAR TEST RESULTS

FIGURE B-3



● **PEAK STRENGTH**
Friction Angle= 27 degrees
Cohesion= 258 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 28 degrees
Cohesion= 150 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	40.0	SILTY CLAY (CL)	113	14.5

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PROJECT NO.: 2981.I

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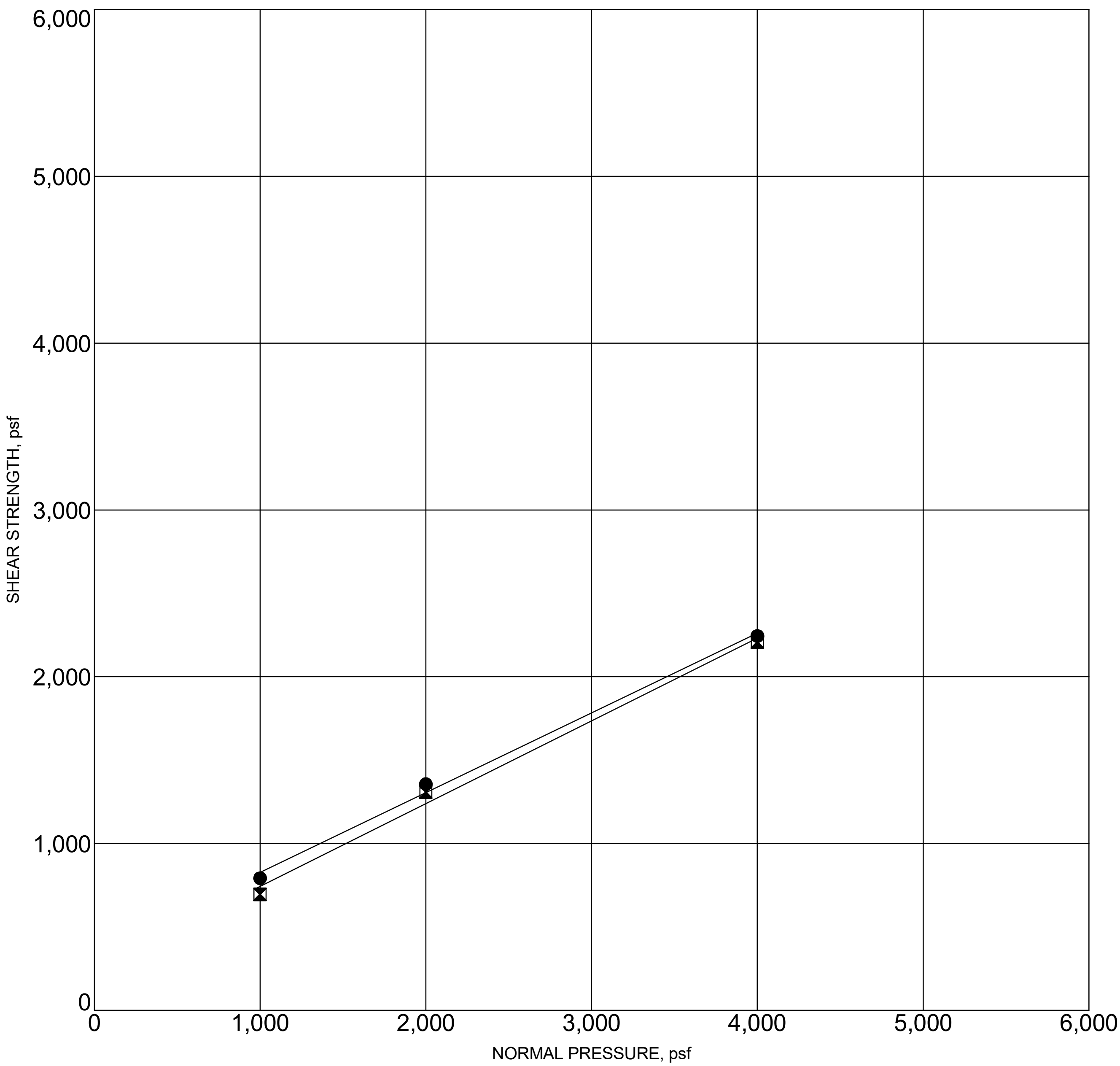
DIRECT SHEAR TEST RESULTS

FIGURE B-4



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 348 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 26 degrees
Cohesion= 246 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	50.0	SANDY SILT (ML)	111	13.5

PROJECT: ST. JOSEPH MOB

PROJECT NO.: 2981.I

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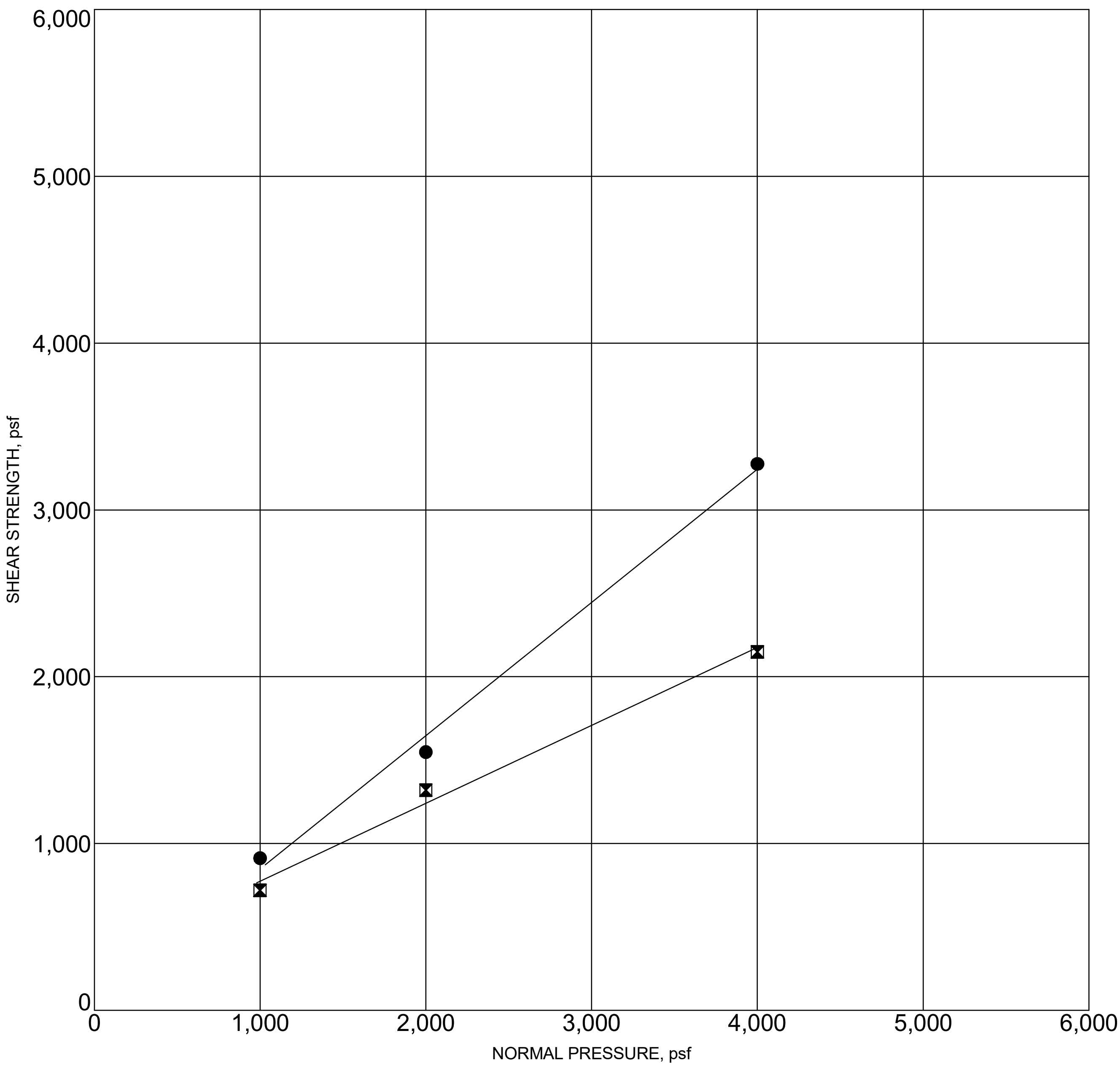
DIRECT SHEAR TEST RESULTS

FIGURE B-5



● **PEAK STRENGTH**
Friction Angle= 39 degrees
Cohesion= 48 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 25 degrees
Cohesion= 306 psf

Sample Location		Classification	DD,pcf	MC,%
B-2	40.0	CLAYEY SAND (SC)	119	13.2

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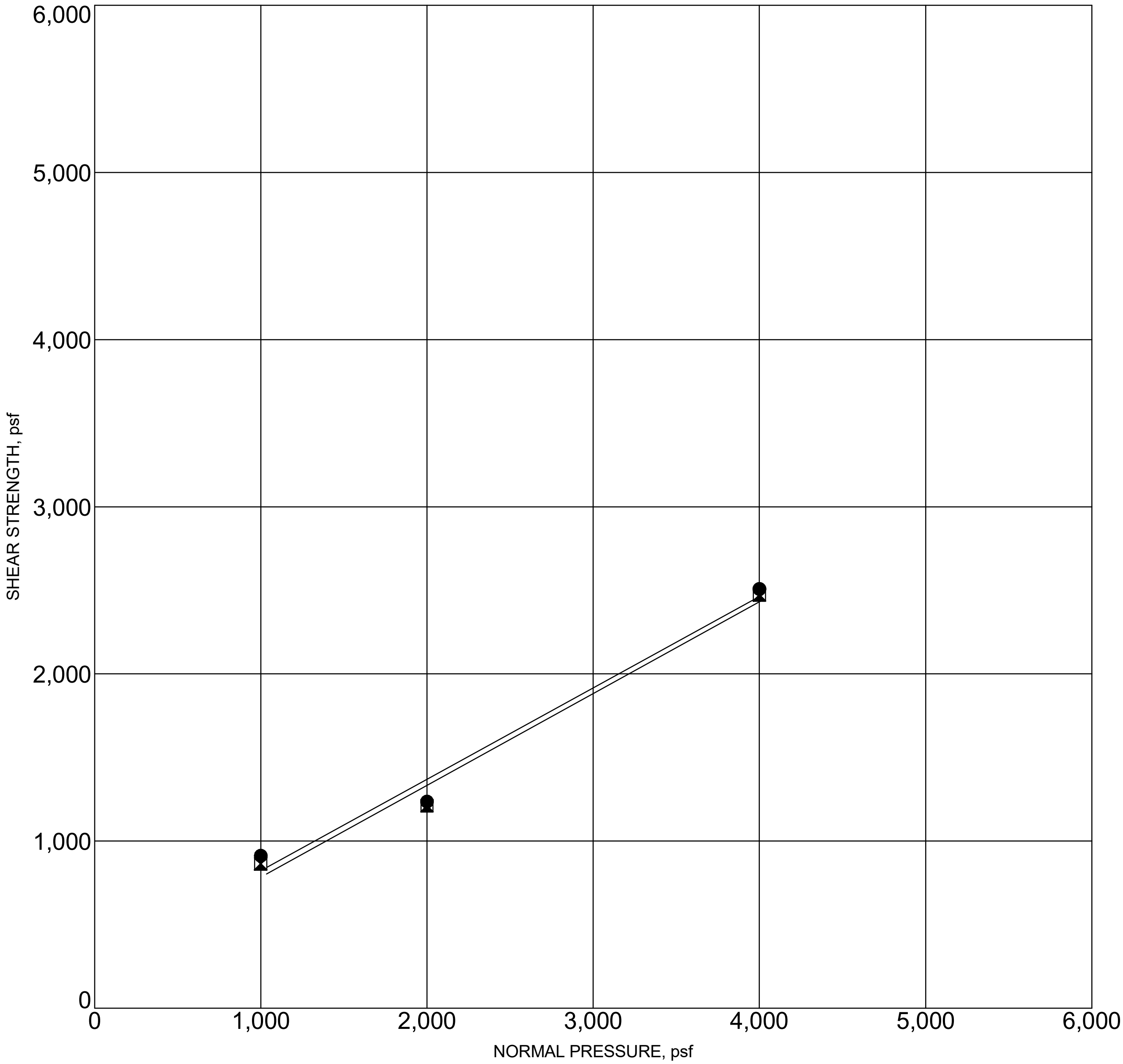
DIRECT SHEAR TEST RESULTS

FIGURE B-6



● **PEAK STRENGTH**
Friction Angle= 29 degrees
Cohesion= 276 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 29 degrees
Cohesion= 234 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	30.0	SILTY SAND (SM)	103	7.8

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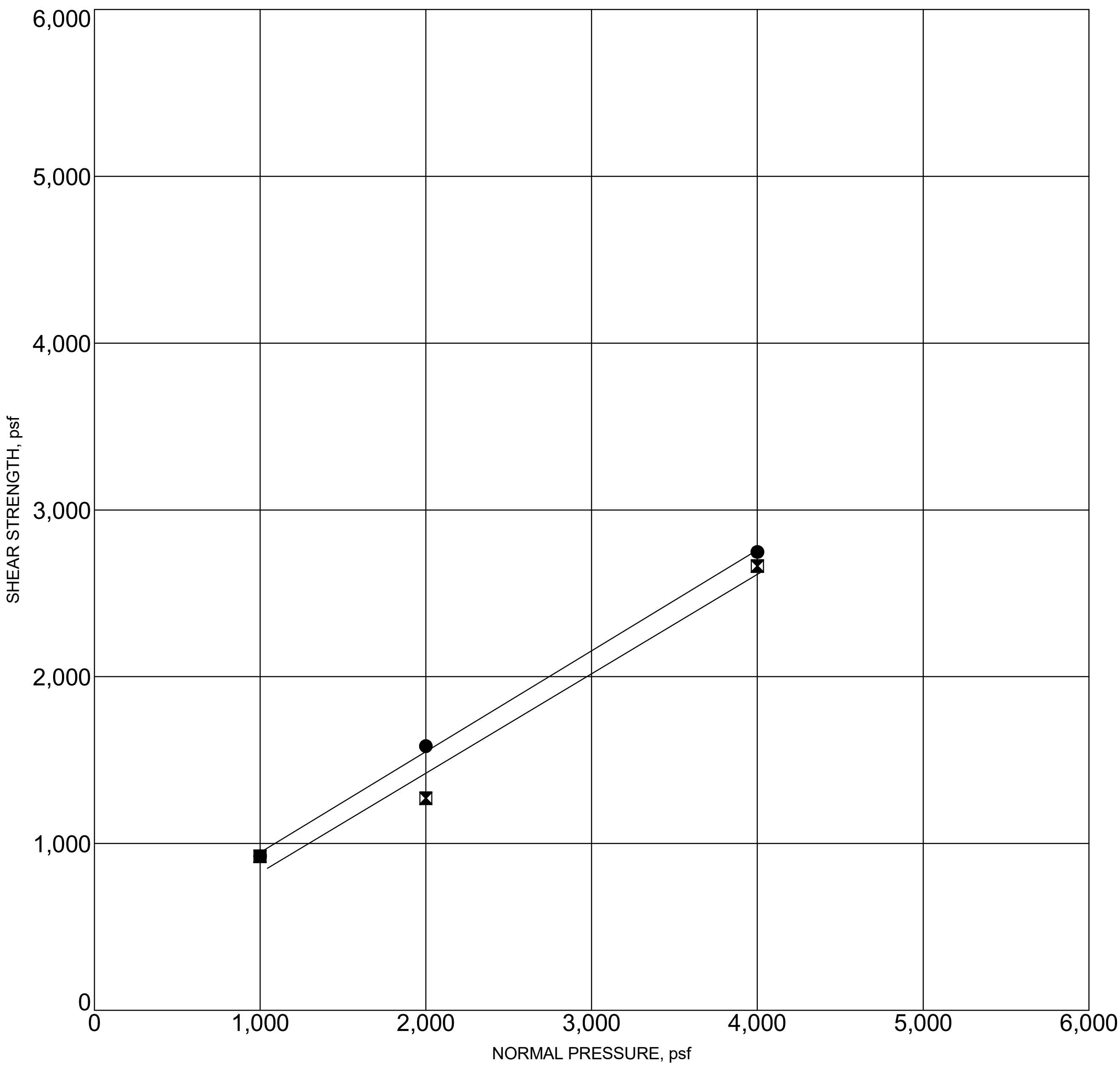
DIRECT SHEAR TEST RESULTS

FIGURE B-7



● **PEAK STRENGTH**
Friction Angle= 31 degrees
Cohesion= 342 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 31 degrees
Cohesion= 228 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	60.0	SAND (SM)	104	2.4

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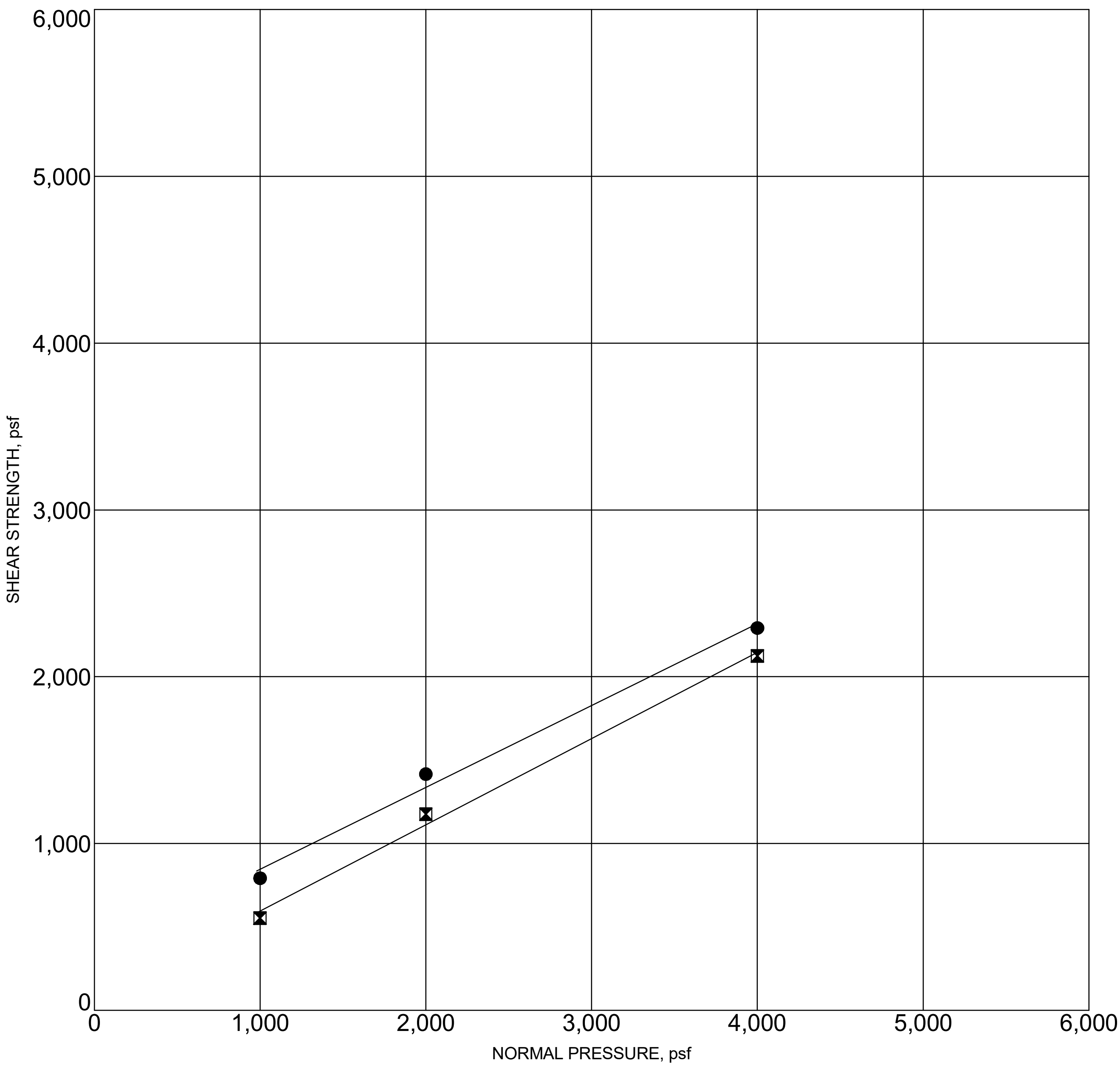
DIRECT SHEAR TEST RESULTS

FIGURE B-8



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 354 psf

ULTIMATE STRENGTH
Friction Angle= 27 degrees
Cohesion= 78 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	25.0	CLAY (CL)	111	17.3

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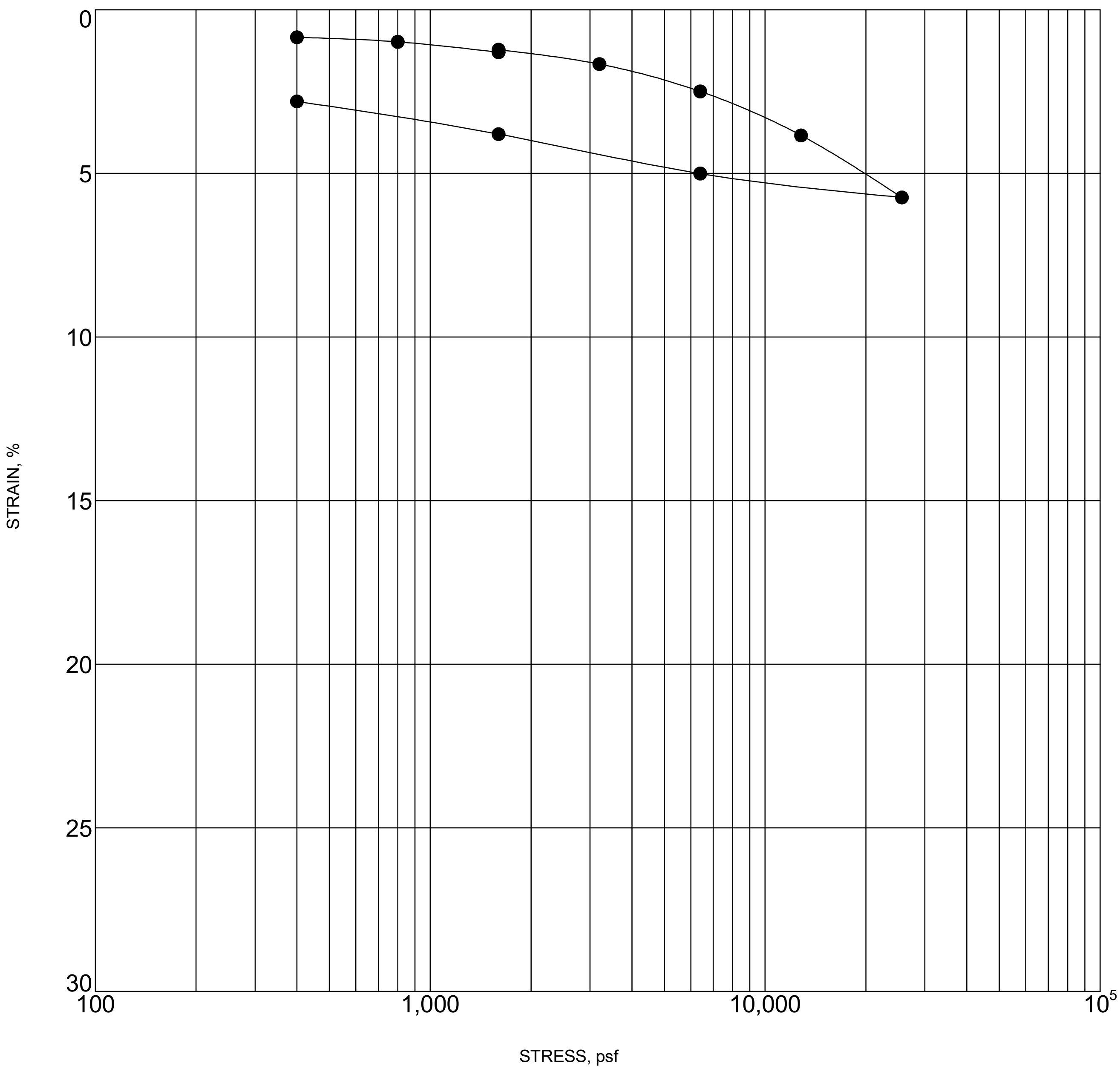
DIRECT SHEAR TEST RESULTS

FIGURE B-9



Sample inundated at 1600 psf

	Sample Location		**Classification**	**DD,pcf**	**MC,%**
●	B-4	40.0	CLAY (CL)	114	16.2

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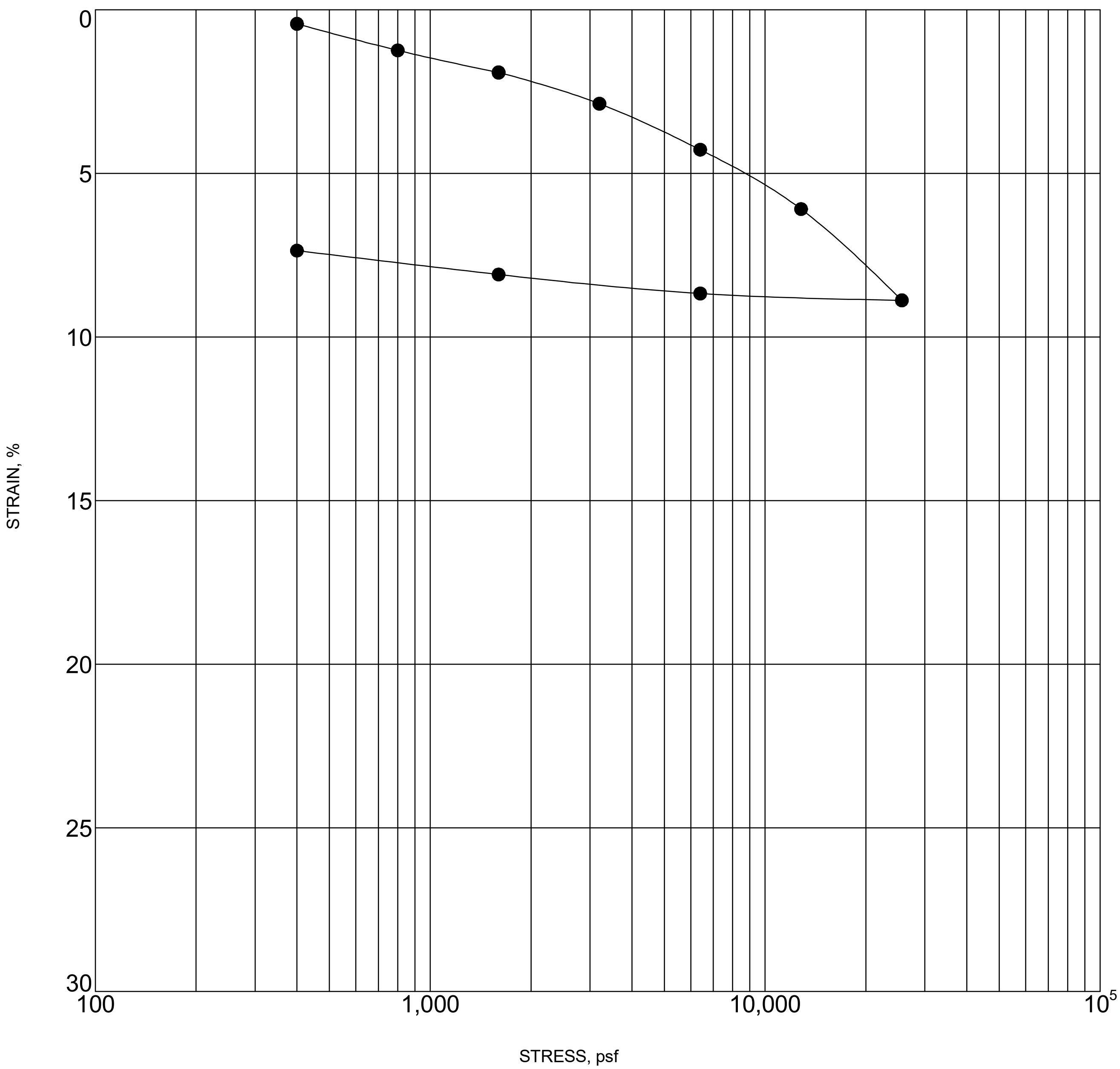
CONSOLIDATION TEST RESULTS

FIGURE B-10



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-4	80.0	SILTY CLAY (CL)	101	22.6

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CONSOLIDATION TEST RESULTS

FIGURE B-11

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
PMB St. Joseph
Your #2981.I, HDR Lab #20-0082LAB
25-Feb-20

Sample ID

			B-1 @ 0-4'	B-2 @ 20'
Resistivity		**Units**		
as-received		ohm-cm	11,600	1,400,000
saturated		ohm-cm	3,280	26,800
pH			8.2	9.1
Electrical Conductivity		mS/cm	0.20	0.03
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	154	30
magnesium	Mg^{2+}	mg/kg	16	4.4
sodium	Na^{1+}	mg/kg	47	19
potassium	K^{1+}	mg/kg	43	2.6
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	302	110
fluoride	F^{1-}	mg/kg	15	3.9
chloride	Cl^{1-}	mg/kg	2.5	0.9
sulfate	SO_4^{2-}	mg/kg	142	4.5
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	0.9	ND
nitrate	NO_3^{1-}	mg/kg	4.6	8.8
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

APPENDIX C

TERRA GEOSCIENCES

SEISMIC SHEAR-WAVE SURVEY

PMB ST. JOSEPH PROJECT

SEC OF COLUMBIA AND MAIN STREETS

CITY OF ORANGE, CALIFORNIA

Project No. 203369-1

February 3, 2020

Prepared for:

Geotechnical Professionals, Inc.
5736 Corporate Avenue
Cypress, CA 90630

Geotechnical Professionals, Inc.
5736 Corporate Avenue
Cypress, CA 90630

Attention: Mr. Don Cords, Principal

Regarding: Seismic Shear-Wave Survey
PMB St. Joseph Project
SEC of Columbia and Main Streets
City of Orange, California
GPI Project No. 2981.I

INTRODUCTION

As requested, this firm has performed a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods for the above-referenced site. The purpose of this survey was to assess the one-dimensional average shear-wave velocity structure, at various depth intervals, beneath the subject survey area, to a depth of at least 100 feet. Geologic mapping by Morton and Miller (2006), indicates the local survey area to be mantled by Holocene to late Pleistocene age young alluvial fan deposits. These deposits are generally described as being comprised of unconsolidated to moderately consolidated silt, sand, pebbly cobbly sand, and bouldery earth materials, in turn underlain at depth by progressively older alluvial deposits.

The location of the seismic traverse has been approximated on a captured Google™ Earth image (Google™ Earth, 2020), which is presented as the Seismic Line Location Map, Plate 1, for reference. Additionally, photographic views of the survey line are presented on Plate 2, for visual and reference purposes. As authorized by you, the following services were performed during this study:

- ***Review of available pertinent published and unpublished geologic and geophysical data in our files pertaining to the site.***

- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.***

- ***Preparation of this report, presenting the results of our findings with respect to the shear-wave velocities of the subsurface earth materials.***

Accompanying Map, Illustrations, and Appendices

Plate 1 - Seismic Line Location Map
Plate 2 - Survey Line Photographs
Appendix A - Shear-Wave Model and Data
Appendix B - References

SUMMARY OF SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One seismic shear-wave survey traverse (Seismic Line SW-1) was performed across a portion of the site, as selected by you, as approximated on the Seismic Line Location Map, Plate 1. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph (Geometrics, 2004). This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM surveys used the same linear geometry array that consisted of a 161-foot long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular seven-foot intervals. Since the survey area was covered by asphalt, each geophone was anchored by the use of drilled holes. For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using a 25-foot offset from the beginning and end of the survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple hammer impacts (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of any artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 20 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Reduction

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2016). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies. Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within Appendix A.

Summary of Data Analysis

Data acquisition went very smoothly and the quality was considered to be very good. The seismic model data (Shear-Wave Model SW-1, see Appendix A) indicates that beneath the survey traverse there are several layers where the seismic velocity generally increases with depth, with a minor velocity reversal between 92.1 to 128.0 feet in depth, and then increasing with depth again.

We understand that the site will consist of construction of a multi-story structure where the foundations will be placed at depth. Therefore, as requested, we have provided a table indicating the average shear-wave velocity ("weighted average") for varying 100-foot block intervals, so that the proper Site Class (ASCE, 2017; Table 20.3-1) can be selected based upon the proposed construction.

TABLE 1 – CALCULATED V100 SHEAR-WAVE VALUES

Depth Interval (feet)	Shear-Wave Velocity ("weighted average")
0 to -100	**1,180.0 ft/sec**
-10 to -110	**1,217.7 ft/sec**
-20 to -120	**1,249.0 ft/sec**
-30 to -130	**1,271.5 ft/sec**
-40 to -140	**1,287.9 ft/sec**

The "weighted average" velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100). This formula is as follows:

$$\mathbf{V100' = 100/[(T1/V1) + (T2/V2) + ...+ (TN/VN)]}$$

Where t1, t2, t3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. As noted above, Table 1 uses this formula and adjusts the V100 interval to account for Site Class design selection at varying 100-foot depth block intervals below the surface.

The shear-wave model displays these calculated layers and associated velocities (feet/second) to the maximum obtained depth of 215 feet, where locally sampled (dark gray shaded area on shear-wave model represents the constrained data). The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within Appendix A for visual and reference purposes.

CLOSURE

The field survey was performed by the undersigned on February 1, 2020, using "state of the art" geophysical equipment and techniques along the selected portion of the subject study area as directed by you. It is important to note that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single “true” earth model. Therefore, the interpretation of any seismic data set uses “best-fit” approximations along with the geologic models that appear to be most reasonable for the local area being surveyed.

It should be noted that when compared with traditional borehole shear-wave surveys, which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent. Client should understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and, in the interpretation, and that the results of this survey may not represent actual subsurface conditions.

These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied. If the client does not understand the limitations of this geophysical survey, additional input should be sought from the consultant.

Respectfully submitted,
TERRA GEOSCIENCES

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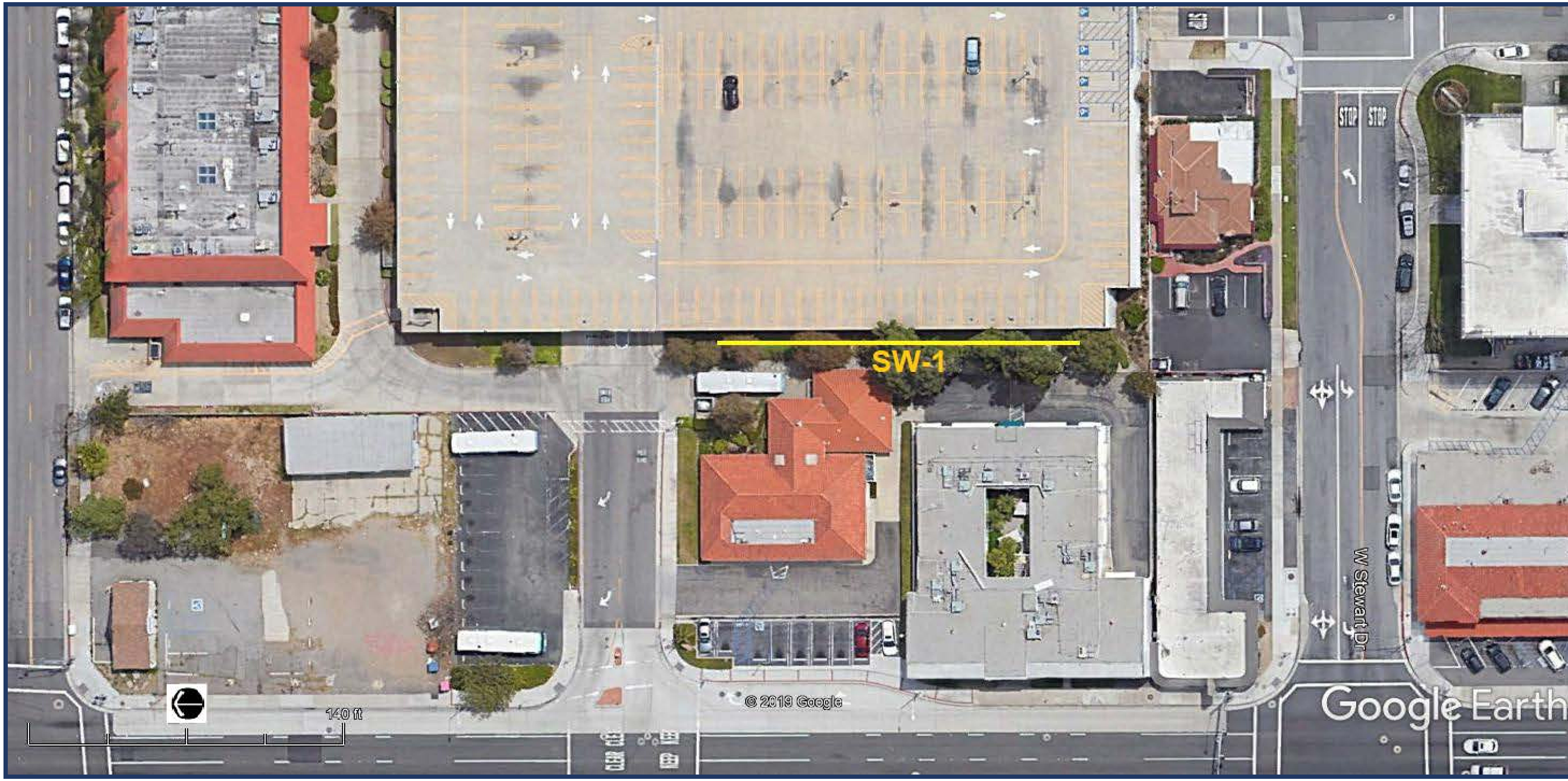
PGP 1002

SEISMIC LINE LOCATION MAP



Base map from Google™ Earth imagery (2020); Seismic shear-wave traverse SW-1 shown as yellow line.

SURVEY LINE PHOTOGRAPHS

View looking north along Seismic Line SW-1.

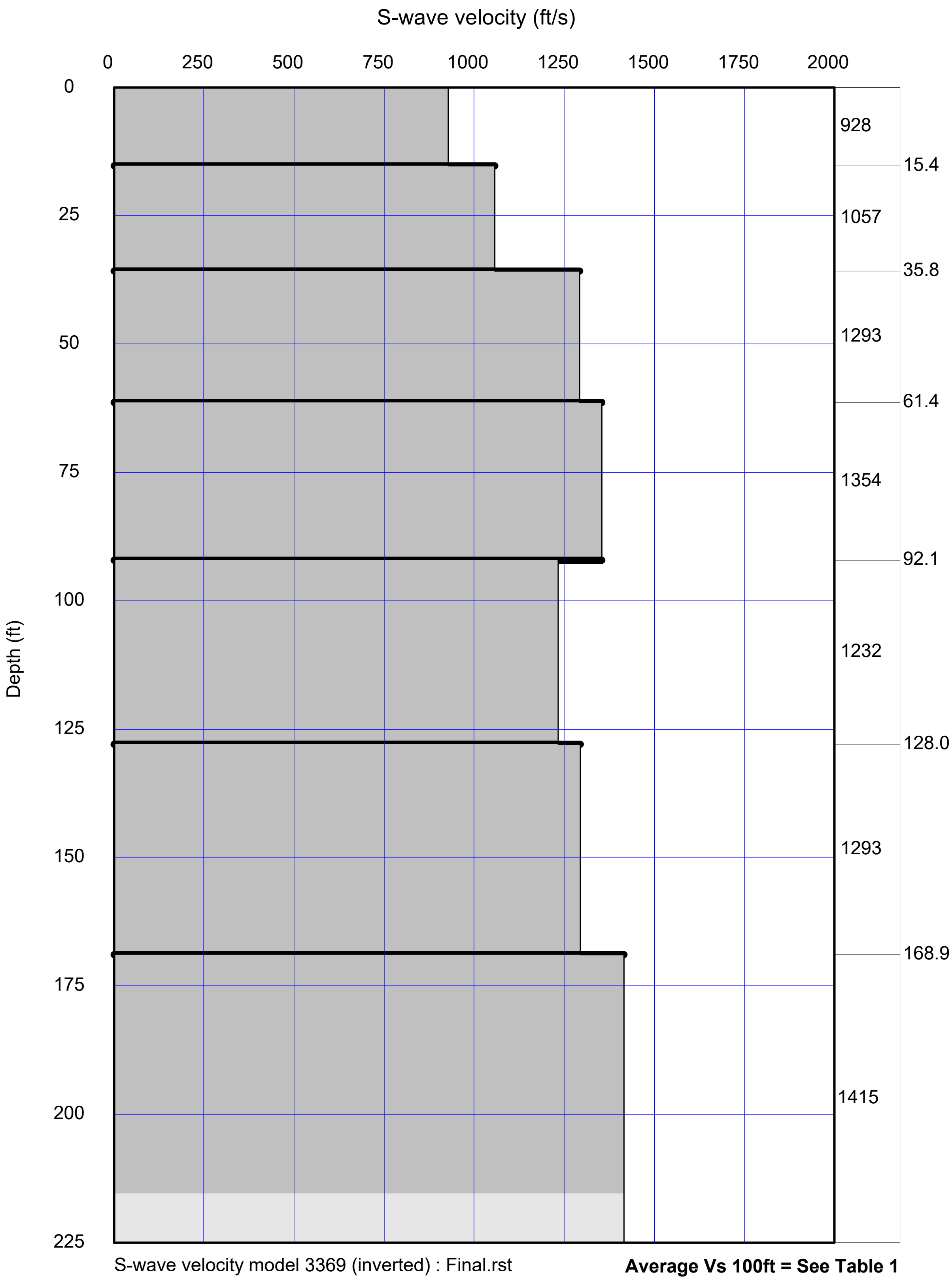
View looking south along Seismic Line SW-1.

APPENDIX A

SHEAR-WAVE MODEL AND DATA

SEISMIC LINE SW-1

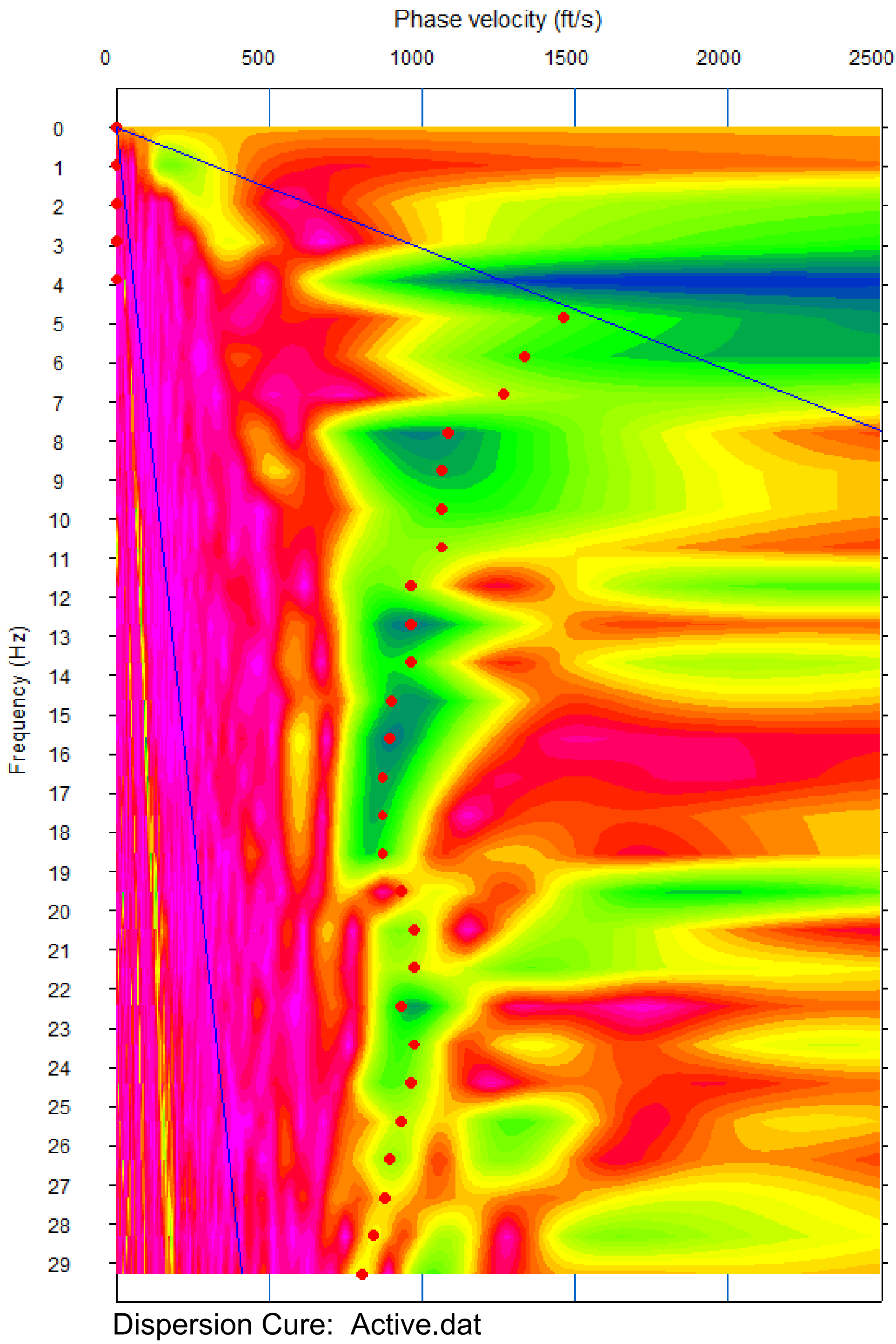
SHEAR-WAVE MODEL



SHEAR-WAVE MODEL SW-1

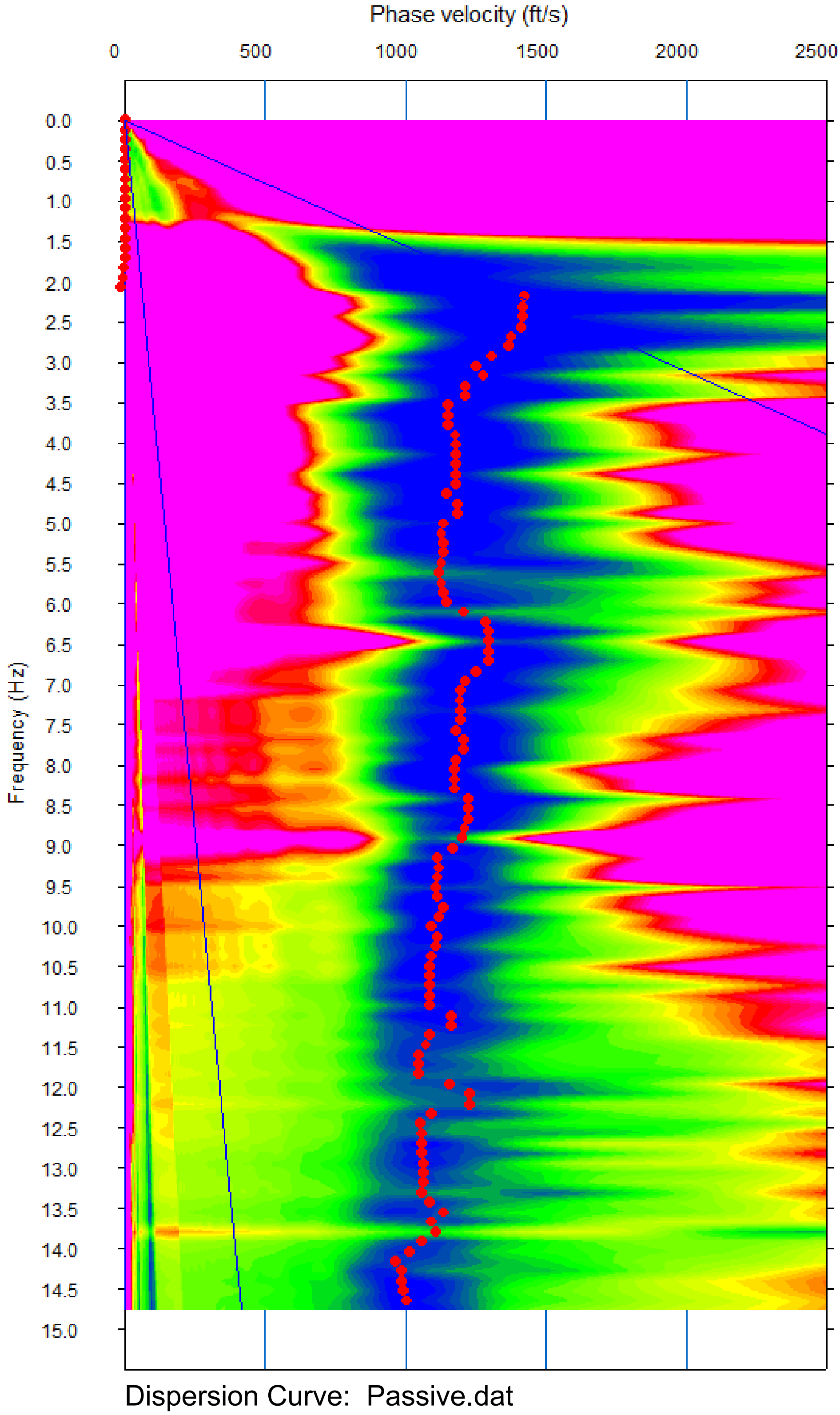
COMBINED DISPERSION CURVE

SEISMIC LINE SW-1



ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



PASSIVE DISPERSION CURVE

APPENDIX B

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REFERENCES

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