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Report
Soil Investigation
Fremont Road Residence
54 Fremont Road
San Rafael, California

Prepared for
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INTRODUCTION

This report presents the results of the soil investigation we performed for a proposed new residence to be constructed at 54 Fremont Street in San Rafael, California. The residence will be positioned on a steeply sloping, northeast-facing hillside, as shown on Plate 1.

Preliminary plans prepared by Arterberry Design are dated July 3, 2018 indicate that the proposed residence will be a one-, two- and three-story structure with the garage on the lower level. We understand that either wood floors supported on joists above grade or concrete slab-on-grade floors are being considered. Retaining walls are planned at the garage and lower level areas of the residence and will retain cuts that will vary in depth from about 21 and 10 feet, respectively. The residence will be accessed by a new driveway off Marquard Avenue.

The object of our investigation, as outlined in our confirming proposal dated December 18, 2018, was to review selected, geologic references in our files, explore subsurface conditions, measure depth to groundwater, if encountered, and determine physical properties of the soils encountered. We then performed engineering analyses to develop conclusions and recommendations concerning:

1. Proximity of the site to active faults.
2. Site preparation and grading.
3. Foundation support and design criteria.
4. Support of concrete slab-on-grade floors.
5. Retaining wall design criteria.

6. Soil engineering drainage.

7. Supplemental soil engineering services.

WORK PERFORMED

We reviewed selected, published geologic information in our files and online, and viewed historic imagery from Google Earth of the site and vicinity. Imagery and literature reviewed are listed in the Reference section of this report.

On September 5, 2018, we observed surface features and explored subsurface conditions to the extent of five (5) test pits at the approximate locations indicated on Plate 1. The pits were excavated to depths of about 7 to 12 feet with track-mounted excavator equipment. Our project geologist located the pits, observed the excavations, logged the conditions encountered and obtained a few samples for minor laboratory classification testing. In addition, we performed strength indicator tests in the walls of the pits with a penetrometer. Logs of the pits showing soil conditions encountered are presented on Plate 2. The soils are classified in accordance with the Unified Soil Classification System explained on Plate 3. Rock physical characteristics are described on Plate 4.

Selected samples were tested in our laboratory to determine classification (Atterberg Limits and percent free swell). The laboratory results and penetrometer data are summarized on Plates 5a and 5b. Detailed results of the Atterberg Limits tests are presented on Plate 6.

The pit locations shown on Plate 1 are approximate and were determined by visually estimating from existing surface features. The locations should be considered no more accurate than implied by the methods used to establish the data. At the completion of the exploration all the pits were backfilled with soil and rock obtained from the pit excavations.

SURFACE AND SUBSURFACE CONDITIONS

The project site is located in west San Rafael, within steep hillside terrain in an area termed Moore Hill. The building site is situated on a steep northeast-facing hillslope, bounded by Marquard Avenue on the northeast and Fremont Road on the southwest. Existing residential homes are located east and west of the property. Directly upslope, an existing older home is present. We understand the home was constructed in the 1920s.

Based on the topographic maps, elevations of the property range from approximately 90 to 145 feet above mean sea level with an average slope inclination of about two-and-one-half horizontal to one vertical (2½:1).

Located along the upslope side of Fremont Road, and upslope from the project site, a drainage swale is present. This swale drains a large portion of the northwest/southeast trending ridgeline that includes Moore Hill. In the central and east portions of the subject lot a subtle lobate feature is present.

The test pits indicate that, in general, the proposed residence area is underlain by discontinuous accumulations of sandy silts, silty sands and clays and clayey sands with gravels overlying highly weathered bedrock of the Franciscan Complex. The total soil thickness in the pits ranged from about 5 to 10 feet thick. Accumulations of soil observed in test pits 2, 3 and 5 ranged from about 7 to 10 feet. In test pits 1 and 4, soil thickness varied from about 5 to 6½.

The upper 1 to 1½ feet of soil in Test Pit 2 consisted of relatively weak, clayey gravel fills. The upper materials encountered below the existing fills in Test Pit 2 and exposed at the surface in Test Pit 3 consisted of sandy silts, typical of topsoils. Underlying the topsoils, angular to subangular gravels within a sandy silt matrix were observed. These materials are judged to be characteristic of debris flow deposits. Laboratory tests indicate that these materials exhibit a low expansion potential. That is, the soil would tend to undergo low strength and volume changes with seasonal moisture variations. The materials were also observed to be porous and contain small roots. In Test Pits 2 and 3, buried topsoil (paleosol) was observed underlying about 5 to 7 feet of debris flow material. The paleosol was in turn underlain by additional debris flow deposits to depths of about 10 feet. In Test Pit 3, underlying the topsoils we observed about 6 feet of highly expansive clay.

All of the test pits bottomed into highly weathered sandstone of the Franciscan Complex. Published maps indicate bedrock of the Franciscan Complex occurs at the project site.

Neither groundwater nor seepage were observed in any of our test pits during the exploration. Our experience indicates that groundwater levels vary seasonally and could rise and fall several feet annually. Precise groundwater location, or the presence of a perched water condition, is beyond the scope of this investigation.

<u>Landslides and Slope Stability</u>

Published geologic maps indicate that landsliding is a major slope process affecting the hillsides and mountains of the Marin County region. However, no mapped landslides are shown on the published geologic maps that affect the proposed residence site. Conversations with neighbors in the surrounding area indicated that during a particularly heavy winter storm in the past, a combination of mud and water was deposited up against the older home, upslope of the subject property. It was indicted that the mud extended up the bottom of the roofline.

Based on our observations, a possible debris flow path (denoted with directional arrows on Plate 1) is present upslope and extends into the subject property. The extent of the debris flow within the project area is depicted approximately on the attached Plate 1. The test pit data indicates that the depth of debris flow deposits varies up to about 10 feet.

Soil creep is a phenomenon where weak soil moves slowly downslope under the force of gravity at a fraction of an inch per year. The topsoil, expansive clays and debris-flow deposits encountered in the test pits are considered susceptible to creep.

DISCUSSION AND CONCLUSIONS

Based on our field exploration, laboratory tests, soil engineering analyses and experience with similar subsurface conditions at nearby sites, we conclude that the site can be used for the proposed residential construction. The most significant soil engineering factors that must be considered during design and construction are the presence of:

1. Existing fills, relatively deep weak, compressible soils, and plastic clayey soils of high expansion potential in localized areas overlying firm rock materials on steeply sloping terrain.

2. Debris outwash area.

3. Potential excavation instability related to deep weak soils.

4. Potential for strong seismic shaking.

We believe that the weak, porous compressible topsoils would be subject to significant settlements when saturated under load. Where evaporation is inhibited by slabs, footings, or fill, eventual saturation and settlement could occur. Therefore, we judge that the topsoils are not suitable for support of foundations, floor slabs or new fills in their present condition.

Expansive clays such as those encountered at the site can undergo shrink and swell with seasonal variation in moisture content and can heave and distress lightly loaded footings and slabs. Foundations must be bottomed below the zone of significant moisture change. We have observed that soils exposed to evaporation can undergo significant seasonal moisture changes to depths of about 2 to 3 feet, or more during drought conditions.

Our experience indicates that where weak, porous and/or plastic, clayey soils occur on slopes, these soils are subject to creep as is common on hillsides in the Marin County area. We therefore conclude that it will be necessary to extend foundations below any weak, upper porous soils and, on slopes well below the creep soil zone and extend into firm, underlying bedrock materials.

We judge that satisfactory support for the residence can best be obtained from a system of drilled piers and grade beams that extend well below the zone of significant seasonal moisture variation and are designed to resist lateral creep forces. Furthermore, we judge that retaining walls, if needed, to support planned fills should also be supported on drilled piers that are similarly designed to resist lateral creep soil pressures. However, where planned cuts remove the zone of indicated creep soil and expose firm, bedrock materials, spread footings can be used for support of retaining walls.

Because of presence of creep-affected soils and the associated risk of slab distress, we do not recommend the use of concrete slab-on-grade floors in the living areas unless special grading techniques are utilized. We judge that the most suitable alternative would be the use of wood floors supported on joists above grade. Therefore, the balance of this report is oriented toward that alternative. If other floor systems are to be considered, we should be consulted to provide specific criteria.

We believe that a concrete floor slab can be used in the garage. However, because of anticipated differential supporting conditions consisting of bedrock in cut areas transitioning to overlying creep-affected soils and highly expansive clay, slab settlement and/or heave and more than normal cracking should be anticipated. Provided the risk of future distress would be acceptable, the garage slab could be supported on a compacted soil subgrade and would need to be separated from adjacent foundations using felt paper or expansion joint material.

Based on our observations and conversations with neighbors regarding a potential past debris flow event, we judge that there is a potential for future debris flows to affect the subject lot. Debris flows are relatively rapidly moving mixtures of saturated soil and rock debris. Debris flows pose a hazard to personnel and structures at the time they occur, as well as depositing loose accumulations of soil subject to compressibility under future loading conditions.

Debris fences and/or deflection walls installed upslope of proposed improvements or catchment walls incorporated into the residence design are judged to be warranted as part of the site development. As recommended in subsequent section of this report, such devices used to catch or deflect debris may need routine maintenance and/or debris removal. In addition, surface drainage measures will be needed to help collect runoff water into pipelines that discharge into existing or planned new drainage facilities. We believe that the presence of the existing residence upslope of the subject site would provide some protection to the planned residence in the event of a future debris flow. Therefore, as a minimum, we judge that a catchment/debris walls should be provided for the area of the proposed residence that extends beyond the existing uphill structure in the area shown on Plate 1. However, it should be understood that if the existing residence upslope is demolished or removed, catchment and/or debris walls would be needed.

Because of the presence of thick weak and/or plastic clayey soils, the depth of proposed cuts, and the relative positioning of existing residential structures within close vicinity to proposed excavations, we judge that there is a higher than normal risk of potential instability of temporary cut slope excavations. Accordingly, the possible need for shoring of temporary excavations should be recognized. All temporary slopes, shoring and the stability of improvements during construction should be contractually established as solely the responsibility of the contractor.

For foundation designed and installed in accordance with our recommendations, we judge that settlements will be small, less than about 1 inch. Post-construction settlements should be about one-half this amount.

The pits were backfilled with the excavated soils but were not compacted. Therefore, the test pit backfills constitute local zones of highly compressible materials. Where encountered in planned improvement areas, the pit backfills should be removed for their entire depth and the soils replaced as properly compacted fill or foundation elements deepened accordingly.

SEISMIC DESIGN

The geologic maps reviewed did not indicate the presence of active faults at the site and the site is not located within a presently designated Alquist-Priolo Earthquake Fault Zone. Therefore, we judge that there is little risk of fault-related ground rupture during earthquakes. In a seismically active region such as Northern California, there is always some possibility for future faulting at any site. However, historical occurrences of surface faulting have generally closely followed the trace of the more recently active faults.

The closest faults generally considered active are the San Andreas located approximately 8½ miles to the southwest and the Hayward fault zone located approximately 7 miles to the northeast.

Very strong ground shaking will occur during earthquakes. The intensity at the site will depend on the distance to the earthquake epicenter, depth and magnitude of the shock, and the response characteristics of the materials beneath the site. Because of the proximity of active faults in the region and the potential for very strong ground shaking, it will be necessary to design and construct the project in strict accordance with current standards for earthquake-resistant construction.

We have determined the seismic ground motion values summarized below in accordance with procedures outlined in Section 1613 of the 2016 California Building Code (CBC). Mapped acceleration parameters (S_S and S_1) were obtained by inputting approximate site coordinates (latitude and longitude) site location into seismic design mapping tools provided by the Structural Engineers Association of California/California's Office of Statewide Health Planning and Development (SEAOC/OSHPD) . Based on our review of available geologic maps and our knowledge of the subsurface conditions, we judge that the site can be classified as Site Class C, as described in Table 20.3-1 of the American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard ASCE/SEI 7-10. Using corresponding values of site coefficients for Site Class C and procedures outlined in the CBC, the mapped acceleration parameters were adjusted to yield the design spectral response acceleration parameters S_{DS} and S_{D1}. The following earthquake design data summarize the results of the procedures outlined above.

2016 CBC Ground Motion Parameters

Site Class	C

Mapped Spectral Response Accelerations:

S_S	1.500g
S_1	0.600g

Design Spectral Response Accelerations:

S_{DS}	1.000g
S_{D1}	0.520g

RECOMMENDATIONS

Site Preparation and Grading

Cuts and fills generally decrease site stability. Therefore, grading should be kept to a minimum. Fills should be avoided and any planned cuts or fills should be retained with walls.

Areas to be developed should be cleared of dense growths of grass and should be stripped of the upper soils containing root growth and organic matter. We anticipate that the depth of stripping needed will average about 3 inches. The grass and strippings should be removed from the site or reused in landscaping areas.

Wells, septic tanks or other underground obstructions encountered during grading should be removed or abandoned in place. The resultant voids should be backfilled with soil or granular material that is properly compacted, as subsequently discussed, or capped with concrete. The method of removal/abandonment and void backfilling should be determined by the appropriate governing agency and/or the soil engineer.

In general, areas to receive fill should be scarified at least 6 inches deep, moisture conditioned to slightly above optimum (at least 4 percentage points above optimum for expansive clayey soils) and compacted to at least 90 percent relative compaction[1]. As indicated above, we recommend planned fills on slopes be retained by walls.

Approved on-site and/or imported fill materials should be placed in layers, moisture conditioned and similarly compacted to at least 90 percent relative compaction. On-site clayey soils should be moisture conditioned to and maintained at levels at least 4 percentage points above optimum.

Imported fills, if used, should be of low expansion potential and have a Plasticity Index of 15 or less. The imported fill material should be free of organic matter and rocks or hard fragments larger than 4 inches in diameter. The material proposed for use as nonexpansive fill should be tested and approved by the soil engineer before delivering to the site.

Foundation Support

Because of slope steepness and the presence of a creep soil zone, we recommend that foundation support for the house be obtained from a drilled pier and grade beam system. Piers on or close to slopes should be designed and reinforced to withstand lateral creep soil forces

1 Relative compaction refers to the in-place dry density of fill expressed as a percentage of maximum dry density of the same material determined in accordance with the American Society for Testing and Materials (ASTM) Standard ASTM D1557 laboratory compaction test procedure. Optimum moisture content refers to the moisture content at maximum dry density.

imposed by the tendency of the weak, upper and/or expansive soils to creep downward on the slope.

Our recommended creep depth zones (A and B) are shown on Plate 1. Foundation piers should be designed using the criteria tabulated below.

Zone	**Minimum Pier Diameter**	**Design Creep Soil Depth**	**Lateral Creep Soil Pressure***	**Minimum Penetration Into Supporting Soil**	**Minimum Pier Depth**
A	18 inches	7 feet	55 pcf	9 feet	16 feet
B	18 inches	10 feet	55 pcf	11 feet	21 feet

* Creep soil pressure in pounds per cubic foot (pcf), equivalent fluid. Creep soil pressures should be applied over two pier diameters.

Where planned cuts remove all or a portion of the creep-affected soils, the recommended depth of the design creep soil zone could be reduced accordingly. Therefore, it may be desirable to include a schedule on the project plans to account for such variations.

Grade beams and/or tie beams cast below-grade that are parallel to contour should also be designed to resist lateral creep soil forces. We judge that the need to design grade beams and tie beams to resist lateral forces could be omitted if the elements are formed on top of the ground surface.

Vertical loads on the piers can be carried below the upper 2 feet or the creep soil zone, whichever is deeper, in skin friction using a value of 700 pounds per square foot (psf). No isolated foundations or piers should be used and all piers should be tied together with grade or tie beams. Tie beams, if used, should be at least 12 inches square and contain at least two No. 5 (or three No. 4) reinforcing bars. Piers should be reinforced for their full depth with cages. For piers, resistance to lateral loads can be obtained from a passive earth pressure of 300 pcf, assumed to act over two pier diameters. Passive pressure can be calculated from a depth of 2 feet, but should be neglected within the creep soil zone or within 8 horizontal feet of the face of a descending slope, whichever is deeper. Passive pressure should be limited to 3,000 psf. Piers should be spaced no closer than three diameters, center to center. Piers beneath perimeter and bearing walls should be interconnected with grade beams designed to support the calculated structure loads. In lieu of grade beams under bearing walls, the framing must be sufficient to carry loads, as required by the CBC. To help tie the foundations together on slopes, upslope/downslope tie beams or grade beams should be no farther apart than about 12 feet.

Although no caving soils or groundwater were encountered within any of our test pits during our exploration, such conditions could be encountered during drilled pier installations. If caving soils or groundwater are encountered, it may be necessary to case the holes, dewater the holes or place concrete by an approved pumping or tremmie method.

To retard the wet concrete from settling, the pier holes should not contain more than 3 inches of slough. The slough may need to be tamped prior to concrete placement, as determined by the soil engineer.

Retaining and Catchment Walls

Retaining walls that are free to rotate slightly and support level (and up to 3:1 slope) backfill should be designed to resist an active equivalent fluid pressure of 45 pcf acting in a triangular pressure distribution. Where the backfill slope is steeper than 3:1, the pressure should be increased to 60 pcf. If the wall is constrained at the top and cannot tilt, the design pressures for level and sloping backfill should be increased to 60 and 75 pcf, respectively. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an added surcharge pressure equivalent to 1½ feet of additional backfill.

Because of the potential for future debris flows and where at least 15 feet of level buffer space is not provided between the proposed residence and the uphill slope, a catchment wall should be provided. The catchment wall can be an extension of the building wall or a separate wall of concrete, masonry block or timber construction. The catchment wall or building wall extension should extend at least 5 feet above final grade on the uphill side. Where the catchment wall is separate from a building or retaining wall, creep soil pressures can be neglected for design. Catchment walls should be designed to resist an active equivalent fluid pressure of 45 pcf.

In planned fill areas, retaining wall foundations should be pier supported. Retaining wall foundation piers can be designed in accordance with the recommendations above for house foundations, except the creep soil pressure for landscape walls can be reduced to 35 pcf. However, because of the surcharge weight of the fill, the portion of the piers in the creep soil zone should also be designed to resist an added uniform pressure equal to 40 times the height of the fill retained minus 1 foot [40(H-1)psf]. The uniform pressure should similarly be applied to two pier diameters, and the creep soil force can be assumed to commence at the bottom of fill.

As outlined in the 2016 CBC, it may be necessary to design retaining walls to resist additional lateral soil loads imposed during seismic shaking. Accordingly, based on the

Mononobe-Okabe Method, we have computed the following dynamic component of total thrust induced on the wall for varying backslope inclinations.

Backslope Inclination (β)	Dynamic Component* of Total Thrust (lbs/ft)
$0 \leq \beta \leq 8:1$	$7H^2$
$8:1 < \beta \leq 4:1$	$11H^2$
$4:1 < \beta$	$22H^2$

* The dynamic component of total thrust should be applied as a line load at a height of 0.6H above the base of the retaining wall; where H is height of the retaining wall.

In general, walls retaining planned cuts should also be supported on a drilled pier foundation. However, where planned cuts remove the creep-affected soils and expose firm bedrock, it will be suitable to use spread footings. Such footings, if used, should extend 12 inches below lowest adjacent grade and can be designed for dead plus long-term live load and total design load (including wind or seismic forces) bearing pressures of 2,000 and 3,000 psf, respectively. Passive pressure should be neglected within the upper 12 inches unless confined by asphalt pavements or a concrete slab, and within 8 horizontal feet from the face of the nearest slope. Resistance to lateral loads can be obtained using a passive equivalent fluid pressure and a friction factor of 300 pcf and 0.30, respectively.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch-diameter, perforated rigid plastic pipe (SDR 35 or equivalent) sloped to drain to outlets by gravity and free-draining crushed rock or gravel drainrock. The crushed rock or gravel should extend to within 12 inches of the surface. The drainrock should conform to the quality requirements for Class 2 Permeable Material per Caltrans Standard Specifications. As an alternative, any clean, washed durable rock product containing less than 1 percent soil fines, by weight, could be used if the rock is separated from the soil bank and covered with a nonwoven geotextile fabric (such as Mirafi 140N or equivalent) weighing at least 4 ounces per square yard. The upper 12 inches should be backfilled with compacted soil to inhibit surface water infiltration. The ground surface behind retaining walls should be sloped to drain. Where migration of moisture through walls would be detrimental, the walls should be waterproofed.

Slab-on-Grade

The garage floor slab should be underlain with a capillary moisture break and cushion layer consisting of at least 4 inches of free-draining gravel or crushed rock (slab rock). The gravel or crushed rock should be at least 1/4-inch and no larger than 3/4-inch in size. Moisture vapor will condense on the underside of slabs. Where migration of moisture vapor through slabs is detrimental, a 10-mil minimum vapor retarder should be provided between the supporting base material and the slabs. Two inches of moist, clean sand could be placed on top of the membrane to aid in curing and to help provide puncture protection. However, the actual use of sand should be determined by the architect or design engineer. The use of a less permeable and stronger membrane should be considered if sand is not to be placed for puncture protection, or where the flooring manufacturer requires a vapor barrier. Concrete design and curing specifications should recognize the potential adverse affects associated with placement of concrete directly on the membrane.

Slabs should be at least 4 inches thick and be reinforced with at least No. 3 bars to reduce cracking, and the garage slab should be structurally separated from the foundations. Actual slab thickness and reinforcing should be determined by the structural design engineer based on anticipated use and performance. Prior to placing the reinforcing or slab rock, the subgrade soils should be thoroughly moistened and be smooth, firm and uniform. Slab subgrade should not be allowed to dry prior to concrete placement.

To help provide an outlet for water that could accumulate in the underslab rock and reduce the risk of future moisture migration up through the garage floor slab, we recommend that at least two perforated plastic pipes, at least 10 feet long be embedded in the grade below the underslab rock. The underslab subdrain system should be connected to a non-perforated outlet pipe that extends through or beneath the perimeter foundation to a suitable discharge point. A typical cross-section of our recommended underslab subdrain is shown on the attached Plate 7. We should provide additional consultation concerning the actual configuration and location of the underslab subdrain system during final design, once foundation plans have been prepared.

<u>Soil Engineering Drainage</u>

Ponding water will cause softening of the site soils and would be detrimental to foundations. It is important that the building site be sloped to drain away from foundations. The roofs should be provided with gutters, and the downspouts should be connected to nonperforated pipelines that discharge to the street, and away from leach field areas, if applicable.

Foundation subdrains may be needed along the uphill house foundations and could be needed at intermediate grade beam levels. Foundation subdrains should consist of trenches about 12 inches wide by about 18 inches deep that are filled with free-draining gravel or crushed rock. The trench should extend at least 8 inches below the bottom of the adjacent grade beam. A 3-inch-diameter, perforated plastic pipe should be installed in the trench on a bed of drainrock. The drainrock (and fabric) should conform to the recommendations above for retaining wall backdrains. The rock should extend to within 6 inches of the surface and at least 4 inches above the bottom of the grade beam. The upper 6 inches should consist of compacted, excavated soil to inhibit surface water infiltration. The perforated pipe should extend to a suitable gravity discharge point, as discussed above. A typical cross-section of a foundation subdrain is shown on Plate 8.

With a drilled pier and grade beam foundation, there is a potential for outside water to seep under grade beams and collect in underfloor areas. Careful attention to fine or finish grading around the house should be provided. Loose or poorly compacted materials should not be allowed adjacent to grade beams, and underfloor drainage inlets, pipelines, swales and/or subdrains should be installed. We can provide specific recommendations, if desired.

Roof downspouts and surface drains must be maintained entirely separate from retaining wall backdrains, foundation and underslab subdrains.

Supplemental Services

We should review final grading and foundation plans for conformance with the intent of our recommendations. During construction, we should observe site grading work, pier drilling operations, footing excavations, and subdrain installations to verify that the conditions encountered are as anticipated and to modify our recommendations, if warranted. Field and laboratory tests should be performed to ascertain that the specified moisture content and degree of compaction are being attained.

MAINTENANCE

Periodic land maintenance will be required by the homeowner. Drains should be checked regularly and cleaned and maintained as necessary. A dense growth of deep-rooted, fast-growing ground cover should be established and maintained on all graded slopes. Sloughing, erosion or sliding are common on newly graded slopes, especially during the first few winters. Supplemental erosion inhibitors such as jute mesh or other commercially available materials may be prudent to apply. Any such sloughing, erosion or sliding that does occur should be repaired promptly before it can enlarge.

LIMITATIONS

We have performed the investigation and prepared this report in accordance with generally accepted standards of the soil engineering profession. No warranty, either express or implied, is given. This scope of work is limited to evaluating the physical properties of earth materials considered typical of geotechnical engineering practice and does not include other concerns such as soil chemistry, corrosion potential, mold, and soil and/or groundwater contamination.

Subsurface conditions are complex and may differ from those indicated by surface features or encountered at test pit locations. Therefore, variations in subsurface conditions not indicated on the logs could be encountered. Supplemental services as recommended herein are in addition to this investigation and are charged for on an hourly basis in accordance with our Standard Schedule of Charges. Such supplemental services are performed on an as-requested basis. We can accept no responsibility for items we are not notified to check, nor for the use or interpretation by others of the information contained herein.

If the project or location is revised, or if conditions different from those described in this report are encountered during construction, we should be notified immediately so that we can take timely action to modify our recommendations, if warranted.

Site conditions and standards of practice change. Therefore, we should be notified to update this report if construction is not performed within 24 months.

LIST OF PLATES

DISTRIBUTION

Copies submitted: 5

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