REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS
134 LYSTRA COURT
TELEPHONE (707) 528-3078
REESE@REESEANDASSOC.COM
SANTA ROSA, CA 95403
FACSIMILE (707) 528-2837

Report
Soil Investigation
Melcon Residences
2485 Middle Two Rock Road
Petaluma, California

Prepared for
Joshua Melcon
53 Everett Road
Petaluma, CA 94952

By

REESE & ASSOCIATES
Consulting Geotechnical Engineers

Tyler J. Cole
Staff Geologist

Jeffrey K. Reese
Civil Engineer No. 47753

REGISTERED PROFESSIONAL ENGINEER
JEFFREY KENT REESE
No. C47753
EXP. 12-31-23
CIVIL
STATE OF CALIFORNIA

Job No. 2544.1.1
May 18, 2022

INTRODUCTION

This report presents the results of our soil investigation for your proposed new residences to be constructed at 2485 Middle Two Rock Road in Petaluma, California. The approximate site location is shown on the attached Plate 1. The sites for the proposed residences are further identified as Lots 1 and 4 of the Melcon, Hamilton, and Theusch Minor Subdivision, as indicated on the Tentative Parcel Map prepared by Curtis & Associates.

The two new residences will consist of one- and/or two-story, wood-frame construction with wood floors supported on joists above grade. The attached garages will have concrete slab-on-grade floors.

The scope of our investigation, as outlined in our proposal dated February 2, 2022, was to explore subsurface conditions and perform 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 garage floors.
5. Retaining wall design criteria.
6. Quality and compaction criteria for development of asphalt-paved driveways.
7. Soil engineering drainage.
8. Supplemental soil engineering services.

WORK PERFORMED

We reviewed selected, geologic information including:

1. "Geologic Map of the Two Rock 7.5' Quadrangle, Sonoma County, California: A Digital Database," S.P. Bezore, R. D. Koehler & R.C. Whitter, California Geological Survey, 2003.

2. "Geology for Planning in Sonoma County," Special Report 120, M. E. Huffman & C.F. Armstrong, California Division of Mines and Geologist, 1980.

3. Report "Geotechnical Investigation, Watson Residence, Petaluma, California," by Reese & Associates, Job No. 179.1.8, dated December 17, 2009.

We performed a soil investigation on the property for a previous owner regarding the construction of the existing residence on Lot 3, with particular focus on the presence or absence of landslide activity at the site. The results of that investigation were presented in our report dated December 17, 2009. In that report, it was determined that the there was no evidence of deep landsliding present at the site.

On February 16, 2022, we observed surface features and explored subsurface conditions to the extent of eight test pits at the approximate locations indicated on Plate 1. The pits were excavated to depths between about 3½ and 7 feet with a track-mounted excavator. Our geologist located the pits, observed the excavations, logged the conditions encountered, and obtained a few samples for visual classification and minor laboratory testing. In addition, we performed strength indicator tests in the walls of the pits with a penetrometer. Logs of the pits showing the soil conditions encountered are presented on Plate 2. The soils are classified in accordance with the Unified Soil Classification System explained on Plate 3.

Selected samples were tested in our laboratory to determine moisture content and classification (percent free swell and Atterberg Limits). The laboratory results are summarized on Plate 4. Detailed results of the Atterberg Limits tests are shown on Plate 5.

The pit locations shown on Plate 1 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, the pits were backfilled with the excavated materials, but without compaction.

SITE CONDITIONS

The subject property is an approximate 41.55-acre, rural residential parcel located in moderately sloping terrain about 3 miles west of Petaluma, California. The property is positioned on a moderate north-facing slope and is accessed from the north by an existing paved driveway off Middle Two Rock Road. The site is surrounded by other rural residential and agricultural properties on all sides. An existing residence is present near the north end of the property and a barn and other various outbuildings are present near the center of the site. These existing structures are located on Lot 2 of the subdivision. The upper residence will be located near, positioned on estimated eleven horizontal to one vertical (11:1) slope about 400 feet southeast of the existing residence. The lower residence site is located about 800 feet north of the residence, positioned on an estimated 8:1 slope. At the time of our exploration, the proposed building areas were unoccupied, and the ground surfaces were covered with a moderate growth of grass and weeds.

The test pits and laboratory tests indicate that the site is generally underlain by weak, compressible natural soils underlain by residual soils and weak sandstone bedrock of the Wilson Grove formation. The test pits at the upper residence site (Lot 4) encountered about 1½ to 5 feet of weak porous sandy clay topsoil underlain by about 1½ to 3 feet of very sandy clay residual soil. Residual soil is the result of the in-place weathering of bedrock. Laboratory testing indicates the upper soils and residual soils are low to possibly high in expansion potential. Such soils could undergo a medium to significant level of strength and volume changes because of seasonal changes in moisture content. The bedrock materials were encountered at depths of about 3½ to 7 feet, and consisted of moderately weathered, soft, fine-grained sandstone of the Wilson Grove Formation.

The test pits at the lower residence site (Lot 1) encountered similar porous topsoils underlain by residual soils, and also bottomed into Wilson Grove sandstone at depths of approximatively 2 to 4½ feet. In general, the bedrock materials became firm and more rock-like with depth.

No groundwater or seepage was encountered during our exploration. Our experience indicates that groundwater levels and seepage conditions vary seasonally and can rise and fall several feet annually. Determination of the precise depth to ground water, extent of seasonal water level fluctuations or existence of perched groundwater conditions is beyond the scope of this investigation.

CONCLUSIONS

Based on our field exploration, laboratory tests and engineering analyses, we conclude that, from a soil engineering standpoint, 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 weak and compressible and moderately expansive soils overlying bedrock on slopes.

Our experience indicates that compressible upper natural soils can undergo considerable strength loss and settlement when loaded in a saturated condition. Where evaporation is inhibited by footings, slabs or fill, eventual saturation of the underlying soils can occur. Accordingly, we judge that the compressible upper soils are not suitable for support of concrete slab-on-grade floors or a shallow spread footing foundation system in their present condition. Further, weak upper and plastic soils underlain by bedrock on a steep slope are subject to creep, as is common on hillsides in the Sonoma County area. Creep is a long-term, gradual downslope movement (on the order of a fraction of an inch per year) of weak and plastic soils under the force of gravity. We conclude that the moist suitable alternatives for foundation and floor support would be the use of either drilled piers and grade beams or deepened spread footings using wood floor supported on joists above grade.

We judge that garage floor slabs can be supported directly on properly prepared natural soils, provided the risk of some future minor settlement and/or heave and resultant distress are acceptable and the slabs are structurally separated from adjacent foundations. We can provide recommendations to reduce the risk of slab distress, if requested.

The test pits were backfilled with the excavated materials, but the soils were not compacted. Therefore, the test pits constitute local deep 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.

For foundations designed and installed in accordance with our subsequent recommendations, we judge that settlements will be small, less than about 1/2-inch. Post-construction settlements should be about one-half this amount.

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 Rodgers Creek fault zone located approximately 7½ miles to the northeast and the San Andreas fault zone located approximately 11 miles to the southwest.

Severe 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 severe 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 seismic ground motion values in accordance with procedures outlined in Section 1613 of the 2019 California Building Code (CBC). Mapped acceleration parameters (S_S and S_1) were obtained by inputting approximate site coordinates (latitude and longitude) into earthquake ground motion software made available for use by the Office of Statewide Health Planning and Development (OSHPD) and the Structural Engineers Association of California (SEAOC). 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 (very dense soil and soft rock), as described in Table 20.3-1 of the American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard ASCE/SEI 7-16. 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 summarizes the results of the procedures outlined above.

2019 CBC Ground Motion Parameters

Site Class	C

Mapped Spectral Response Accelerations:

S_S	1.500g
S_1	0.600g

Design Spectral Response Parameters:

S_{DS}	1.200g
S_{D1}	0.560g

RECOMMENDATIONS

Site Grading

Areas to be developed should be cleared of dense growths of grass and vegetation and should be stripped of the upper soils containing root growth and organic matter. Designated trees should be removed, and tree root systems excavated. We anticipate that the depth of stripping needed will average about 3 inches. The strippings should be removed from the site or stockpiled for reuse as topsoil in landscape areas.

Any 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.

After clearing and stripping, excavation can be performed as necessary. We anticipate that, with the exception of organic matter and rocks or hard fragments larger than 4 inches in diameter, the excavated materials will be suitable for reuse as compacted fill.

The surface exposed by excavation should be scarified to a depth of about 6 inches, moisture conditioned to within about 2 percent of optimum and compacted to at least 87 percent relative compaction.[1] Approved excavated soils then should be spread in 8-inch-thick loose lifts, similarly moisture conditioned, and compacted to at least 90 percent.

Imported fill material, if needed, should be nonexpansive and have a Plasticity Index of 15 or less. Imported material should be free of organic matter and rocks or hard fragments larger than 4 inches in diameter. Material proposed for import should be tested and approved by the soil engineer prior to delivery to the site.

It is our experience that weak upper soils can tend to trap considerable amounts of water into the late spring or early summer. In addition, based on our experience on projects in the immediate site vicinity, the weak upper soils can be susceptible to instability under the weight of conventional grading equipment, particularly when saturated. Accordingly, we believe that grading, especially if performed early in the construction season, would likely require more than normal effort to satisfactorily excavate and/or compact the materials. Local, soft saturated soil conditions should be anticipated if grading is performed in the winter, spring or early summer

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.

months. The need for overexcavations to remove unstable soils, imported granular working pads, geotextile fabrics, dewatering systems, cement-treating techniques or other measures could be needed to complete the building pad and develop subgrade. Accordingly, we suggest that the possible need for such measures be accounted for in the contract documents.

Spread Footings

Spread footings should be at least 12 inches wide and should be planned to bottom on firm bedrock at least 18 inches below the lowest adjacent grade. We anticipate that footing depths will vary up to about 3½ to 7 feet below the existing ground surface at the upper residence site (Lot 4), and 2 to 4½ feet at the lower building site (Lot 1). Because footing depths are anticipated to be relatively deep on Lot 4, the use of drilled piers may be more desirable.

Footings should be stepped to provide level (and up to 10 percent slope) bottoms. Footings should be observed by the soil engineer during excavation to determine the actual conditions encountered, recommend specific footing depths and modify our recommendations, if warranted. Because actual footing depths will vary, we suggest that the contract contain provisions to cover increased (or decreased) costs resulting from added (or reduced) footing depths.

Spread footings can be designed to impose dead plus code live load and total design load (including wind or seismic forces) bearing pressures of 2,000 and 3,000 pounds per square foot (psf), respectively. Resistance to the lateral loads can be obtained from passive pressure below the creep soil zone and soil friction using values of 300 pcf and 0.30, respectively. In addition, any footings on slopes steeper than 6:1 should be designed to resist a 3-foot-thick creep soil zone exerting an equivalent fluid pressure of 55 pounds per cubic foot (pcf). On slopes less than 6:1, and where planned cuts remove the creep soil materials, foundations need not be designed for lateral creep soil pressures.

Drilled Piers

Drilled piers can be used for foundation support and should be at least 12 inches in diameter and extend at least 5 feet into firm natural soil or bedrock, as determined in the field by the soil engineer. Piers on Lot 1 (lower residence) should be at least 8 feet deep, as measured below the ground surface. Piers on Lot 4 (upper residence) should be planned to be at least 10 feet deep. Actual pier depths could vary, as determined in the field by the soil engineer during drilling.

Vertical loads on the piers can be carried below the upper 3 feet in skin friction using a value of 600 psf. End bearing should be neglected because of the difficulty of cleaning out small diameter holes and the uncertainty of mobilizing end bearing and skin friction simultaneously. In general, piers should be spaced no closer than three diameters, center to center.

Resistance to lateral loads on piers can be obtained from a passive equivalent fluid pressure of 300 pcf applied over 2 pier diameters. The passive pressure can be assumed to commence at a depth of 1 foot but should be neglected within the upper 3 feet unless confined by pavement or slab. Passive pressure should be limited to 2,000 psf. In addition, on slopes steeper than 6:1, piers should be designed to resist a 3-foot-thick creep soil zone exerting an equivalent fluid pressure of 55 pcf, acting over two pier diameters. On slopes less than 6:1, and where planned cuts remove the creep soil materials, foundations need not be designed for lateral creep soil pressures.

Piers beneath perimeter and bearing walls should be inter-connected with grade beams designed to support the calculated structural loads. In lieu of grade beams under bearing walls, the framing must be sufficient to carry the loads, as required by the CBC. Piers should be reinforced as determined by the structural design engineer.

To retard wet concrete from settling, pier holes should not contain more than 3 inches of slough. It may be necessary to tamp the slough with a heavy timber prior to concrete placement, as determined in the field by the soil engineer.

Caving soils and groundwater were not encountered during our exploration. However, such conditions could be encountered during pier drilling operations. If caving soils or groundwater are encountered, it may be necessary to case the holes, dewater the holes or place the concrete by an approved pumping or tremie method.

Concrete Slab-on-Grade

Garage slabs should be at least 5 inches thick. All slabs should be reinforced with bars to reduce cracking and to help keep closed those cracks that do appear. Actual slab thickness and reinforcing should be determined by the architect or structural design engineer based on anticipated use and performance. The slabs should be separated from the adjacent foundations using commercial expansion join-material or other positive and low friction separators. Prior to placing the reinforcing or slab rock, the subgrade soils should be thoroughly moisture conditioned and be smooth, firm and uniform. Slabs 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 slab 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 moisture migration through slabs is detrimental, a 10-mil minimum vapor retarder should be provided between the slab rock and the concrete. Two inches of moist, clean sand could be placed over the vapor retarder to aid in curing and help provide puncture protection. However, the actual use of sand should be determined by the architect or structural 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 effects associated with placement of concrete directly on the membrane.

Retaining Walls

Retaining walls that are free to rotate slightly and support level and up to 3:1 sloping backfill should be designed to resist an active equivalent fluid pressure of 40 pcf acting in a triangular pressure distribution. Where the backfill slope is steeper than 3:1, the pressure should be increased to 55 pcf. If the wall is constrained at the top and cannot tilt, the design pressures for level and sloping backfill should be increased to 55 and 70 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. Where an imaginary 1½:1 line projected down from an adjacent foundation intersects a retaining wall, the portion of wall below the intersection should be designed for an additional horizontal surcharge of 100 psf. Retaining walls can be supported on spread footing or drilled pier foundations designed using the criteria above for building foundations.

As outlined in the 2019 CBC, it could 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 (ß)	Dynamic Component of Total Thrust (lbs/ft)*
$0 \leq \beta \leq 8:1$	$6H^2$
$8:1 < \beta \leq 4:1$	$10H^2$
$4:1 < \beta \leq 3:1$	$16H^2$

* 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.

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 (i.e., drainrock). The drainrock should extend at least 12 inches beyond the back of the wall and to within 12 inches of the ground surface, and should conform to the quality requirements for Class 2 Permeable Materials in accordance with the latest edition of the 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 covered and separated from the soil bank by a nonwoven, geotextile fabric weighing at least 4 ounces per square yard (such as Mirafi 140N, or equivalent). The upper 12 inches should be backfilled with compacted soil to inhibit surface water infiltration. Where slab-on-grade floors are placed on retaining wall base slabs (footings) the flow line of the perforated pipe in the wall backdrainage should be at least 8 inches lower than the level of the floor slab. The ground surface behind retaining walls should be sloped to drain. Where migration of moisture through retaining walls would be detrimental, the walls should be waterproofed.

Gravel-Surfaced and/or Asphalt-Paved Driveway

If the driveways are graded and paved prior to construction of the residences, or if repeatedly used by heavily loaded vehicles (such as garbage or delivery trucks), damage to the pavement can occur. Accordingly, we recommend that the driveway section be designed to handle the anticipated heavy truck and/or construction traffic. We can provide specific recommendations, if desired.

The flexible pavement materials used should conform to the quality requirements of the State of California Caltrans Standard Specifications, current edition, and the requirements of the County of Sonoma.

Prior to subgrade preparation, all underground utilities in the driveway area should be installed and properly backfilled. The upper 6 inches of subgrade soils should be uniformly moisture conditioned and compacted to at least 95 percent relative compaction and provide a firm and unyielding surface. This may require overexcavation or scarifying and recompaction to achieve uniformity. The aggregate base materials should be placed in layers no thicker than 6 inches and compaction to at least 95 percent to form a firm and unyielding base.

Geotechnical Drainage

Ponding water will tend to soften the site soils and would be detrimental to foundations. Surface drainage consisting of at least 1/2-inch per foot extending at least 4 fee tout should be provided away from all foundations. The ground surface around the houses should be sloped to provide positive lateral drainage. The roofs should be provided with gutters, and the downspouts should outlet on to paved areas or splash blocks that drain at least 30 inches away from the foundation, or be connected to nonperforated, rigid plastic pipelines that discharge by gravity to suitable outlet locations. Roof downspouts and surface drains must be maintained entirely separate from surface drains, as well as foundation and retaining wall subdrains.

Foundation subdrains should be installed along the uphill house foundations and may 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 6 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. A typical cross-section of the recommended foundation subdrain is shown on Plate 6.

We also recommend that a trenched interceptor subdrain be installed along the upslope side of the proposed building area. The approximate location of the interceptor subdrain should be determined during final design. The trench should be at least 12 inches wide and be bottomed a minimum of 5 feet below existing grade. The trench should be sloped to drain by gravity to suitable discharge points. Four-inch-diameter, perforated, rigid plastic pipe should be placed in the bottom of the trench on a bed of 2 to 3 inches of drainrock or gravel. The trench then should be backfilled to within 12 inches of the surface with similar drainrock. The upper 12 inches should consist of compacted, excavated soils to inhibit surface water infiltration. The drainrock and nonwoven fabric, if used, should conform to the recommendations above for retaining wall backdrains. A typical cross-section is shown on Plate 7.

Homeowner and/or professional landscaping should maintain good positive flow of surface water away from and around the buildings. It should be recognized that fences, walks, patio slabs, lawns, planters, etc., can impede water flow and promote surface soil saturation and seepage under slabs and foundations.

<u>Supplemental Services</u>

We should review final grading and foundation plans for conformance with the intent of our recommendations. During site grading operations, if any, the soil engineer should be notified to provide intermittent observation and testing. The soil engineer should observe the conditions encountered and modify our recommendations, if warranted. Field and laboratory tests should be performed to ascertain that the recommended moisture content and degree of compaction are being attained.

We should observe footing excavation and pier drilling operations to verify that suitable bearing materials are encountered and to modify our recommendations, if warranted. Concrete placement and reinforcing should be checked as stipulated on the project plans or as required by the Building Department. It is our understanding that approval from the Building Department must be obtained prior to the placement of concrete in foundation elements.

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. It should be understood that our services were limited to the scope of work outlined above and specifically excluded other services including, but not limited to, an evaluation or analysis of soil chemistry, corrosion potential, mold and soil/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.

If the project 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.

Supplemental services as recommended herein are performed on an as-requested basis. We can accept no responsibility for items we are not notified to check, or for use or interpretation by others of the information contained herein. Such services are in addition to this soil investigation and are charged for on an hourly basis in accordance with our Standard Schedule of Charges.

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

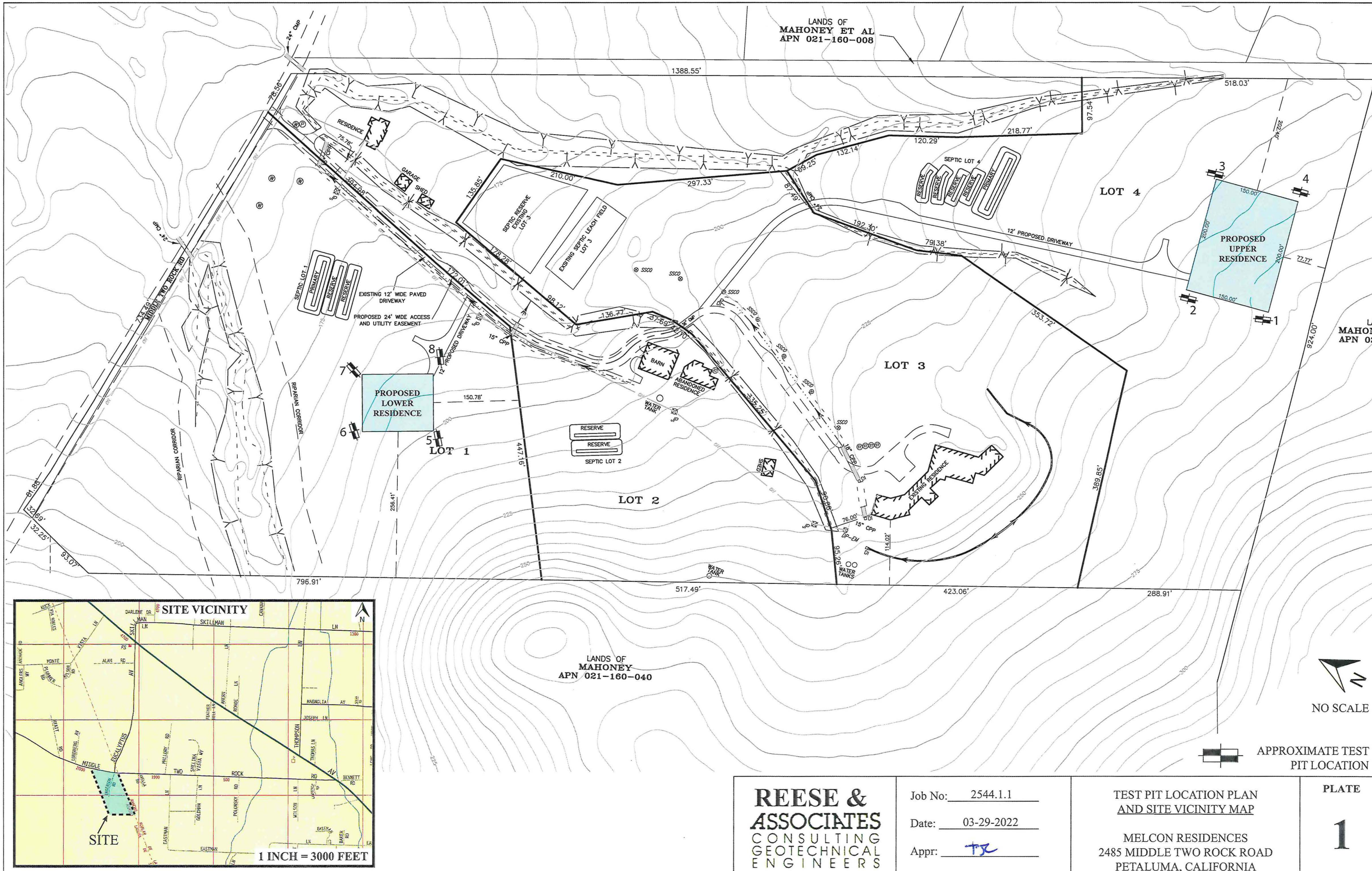
LIST OF PLATES

DISTRIBUTION

Copies Submitted: 2

Joshua Melcon
53 Everett Road
Petaluma, CA 94952
joshua.melcon@gmail.com

TJC/JKR:nay/ra/Job No. 2544.1.1



REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 2544.1.1

Date: 03-29-2022

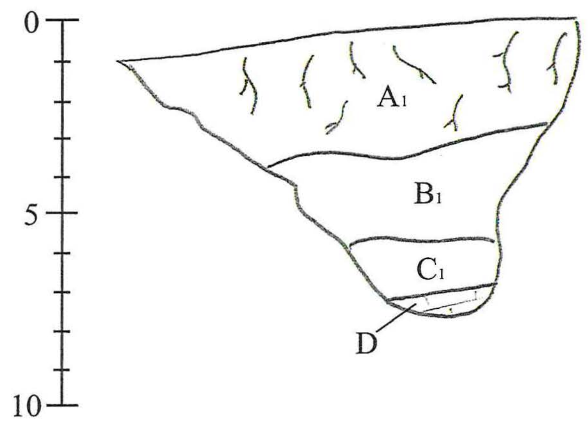
Appr: TK

TEST PIT LOCATION PLAN
AND SITE VICINITY MAP

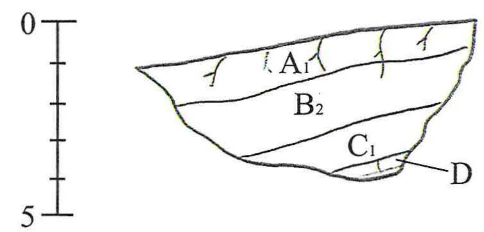
MELCON RESIDENCES
2485 MIDDLE TWO ROCK ROAD
PETALUMA, CALIFORNIA

PLATE
1

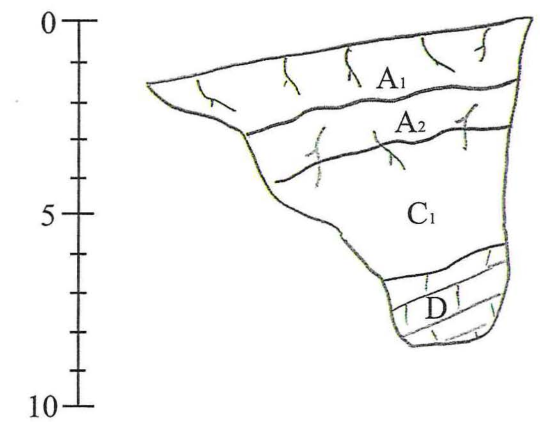
Log of Test Pit 1



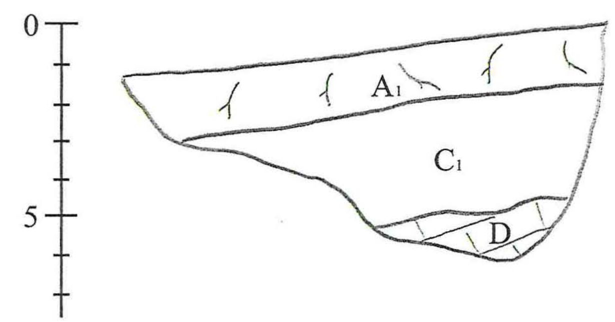
Log of Test Pit 2



Log of Test Pit 3



Log of Test Pit 4



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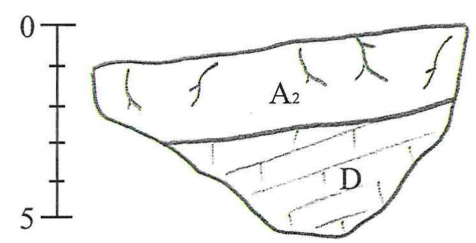

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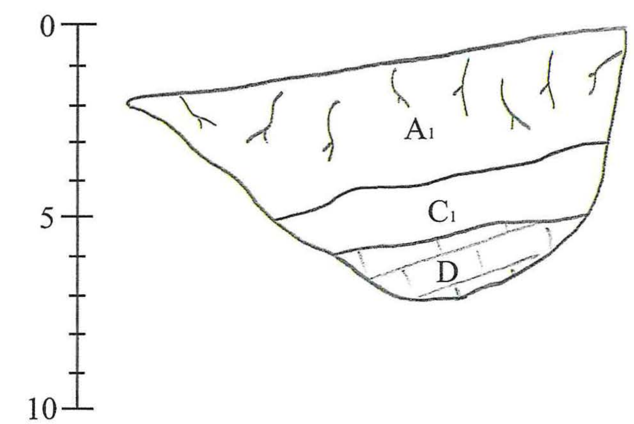
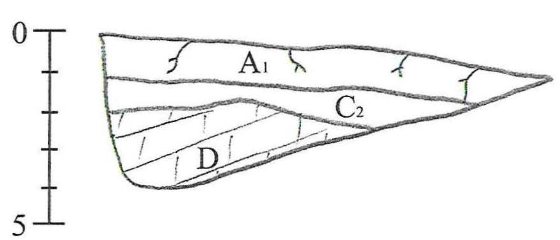


Log of Test Pit 7



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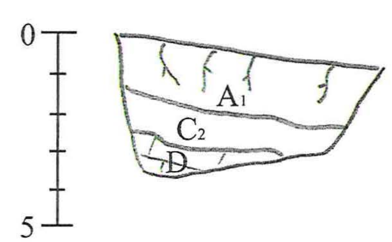


SOIL DESCRIPTIONS

A_1 - DARK BROWN SANDY CLAY (CL)
soft, wet, porous with roots (topsoil)

A_2 - BROWN CLAYEY FINE SAND (SC)
medium dense, moist,
porous with roots (topsoil)

B_1 - DARK GRAY SANDY CLAY (CL)
soft, wet to saturated, occasional small roots

B_2 - MOTTLED DARK GRAY-ORANGE
VERY SANDY CLAY (CL)
soft, saturated

C_1 - MOTTLED YELLOW-BROWN
VERY SANDY CLAY (CH)
stiff, moist, small rounded gravels up to $^1/_4$ -inch
(residual soil)

C_2 - MOTTLED GRAY-BROWN-ORANGE
VERY CLAYEY FINE SAND (SC)
medium dense, wet (residual soil)

D - MOTTLED ORANGE-GRAY CLAYEY
FINE-GRAINED SANDSTONE OF THE
WILSON GROVE FORMATION
soft, weak, moderately weathered

SCALE: 1 INCH = 5 FEET
HORIZONTAL AND VERTICAL

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 2544.1.1 Date: 03-29-2022 Appr: TR	LOG OF TEST PITS 1 THROUGH 8 MELCON RESIDENCES 2485 MIDDLE TWO ROCK ROAD PETALUMA, CALIFORNIA	PLATE 2

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN No. 200 SIEVE	GRAVEL MORE THAN HALF OF COARSE FRACTION IS LARGER THAN No. 4 SIEVE SIZE	CLEAN GRAVEL WITH LESS THAN 5% FINES	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		GRAVEL WITH OVER 12% FINES	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SAND MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN No. 4 SIEVE SIZE	CLEAN SAND WITH LESS THAN 5% FINES	SW	WELL GRADED SAND, GRAVELLY SAND
			SP	POORLY GRADED SAND, GRAVELLY SAND
		SAND WITH OVER 12% FINES	SM	SILTY SAND, GRAVEL-SAND-SILT MIXTURE
			SC	CLAYEY SAND, GRAVEL-SAND-CLAY MIXTURE
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN No. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILT, ROCK FLOUR, SANDY OR CLAYEY SILT WITH LOW PLASTICITY
			CL	INORGANIC CLAY OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAY (LEAN)
			OL	ORGANIC CLAY AND ORGANIC SILTY CLAY OF LOW PLASTICITY
	SILT AND CLAY LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOIL, ELASTIC SILT
			CH	INORGANIC CLAY OF HIGH PLASTICITY, GRAVELLY, SANDY OR SILTY CLAY (FAT)
			OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

				Shear Strength, psf	Confining Pressure, psf
EI	– Expansion Index	TxUU	– Unconsolidated Undrained Triaxial	320	(2600)
Consol	– Consolidation	TxCU	– Consolidated Undrained Triaxial	320	(2600)
LL	– Liquid Limit (in %)	DSCD	– Consolidated Drained Direct Shear	2750	(2000)
PL	– Plastic Limit (in %)	FVS	– Field Vane Shear	470	
PI	– Plasticity Index	LVS	– Laboratory Vane Shear	700	
SA	– Sieve Analysis	UC	– Unconfined Compression	2000	*
G_s	– Specific Gravity	UC(P)	– Laboratory Penetrometer	700	*
■	"Undisturbed" Sample				
□	Bulk Sample				

Notes: (1) All strength tests on 2.8" or 2.4" diameter samples unless otherwise indicated. * Compressive Strength

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 2544.1.1 Date: 2-22-22 Appr: TJR	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA MELCON RESIDENCES PETALUMA, CALIFORNIA	PLATE 3

PIT NUMBER	DEPTH	TEST TYPE*	TEST RESULTS
1	1.0	M	20.2
	1.0	UC(P)	1500
	1.0	FS	55
	2.0	UC(P)	1250
	3.0	UC(P)	500
	3.5	M	20.5
	3.5	FS	50
	4.0	UC(P)	2000
	5.0	UC(P)	2500
	5.5	M	20.1
	5.5	FS	80
	6.0	UC(P)	4250
2	0.5	UC(P)	750
	0.7	M	20.9
	0.7	FS	55
	1.0	UC(P)	750
	2.0	M	21.5
	2.0	UC(P)	1000
	2.0	FS	75
	3.0	UC(P)	1750
	3.5	M	20.8
	3.5	FS	80
	4.0	M	22.4
	4.0	UC(P)	4500+
	4.0	FS	55

*Test Type

M	Moisture Content (percent of dry weight)
MD	Moisture Content (percent of dry weight)/dry density (pounds per cubic foot)
UC(P)	Penetrometer - strength indicator (pounds per square foot)
UC	Unconfined Compression (pounds per square foot)
-200	Percent Passing No. 200 sieve by weight
FS	Percent Free Swell

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 2544.1.1 Date: 04-20-22 Appr: TJL	LABORATORY TEST DATA MELCON RESIDENCES 2485 MIDDLE TWO ROCK ROAD PETALUMA, CALIFORNIA	**PLATE** **4a**

PIT NUMBER	DEPTH	TEST TYPE*	TEST RESULTS
3	1.0	UC(P)	2250
	1.5	M	15.3
	1.5	FS	35
	2.0	UC(P)	3000
	3.0	UC(P)	2000
	4.0	UC(P)	3500
	5.0	UC(P)	4000
	6.0	UC(P)	4500+
4	1.0	UC(P)	1500
	2.0	UC(P)	2250
	3.0	UC(P)	3250
	4.0	UC(P)	4500+
5	1.0	M	14.9
	1.0	UC(P)	3250
	1.0	FS	40
	2.0	UC(P)	1750
	3.0	M	25.4
	3.0	UC(P)	4500+
	3.0	FS	30
	4.0	UC(P)	4500+
6	1.0	M	14.2
	1.0	UC(P)	1500
	1.0	FS	35
	2.0	UC(P)	2250
	3.0	UC(P)	2750
	4.0	M	26.0
	4.0	UC(P)	3250
	4.0	FS	65
	5.0	UC(P)	4500+

*Test Type

M	Moisture Content (percent of dry weight)
MD	Moisture Content (percent of dry weight)/dry density (pounds per cubic foot)
UC(P)	Penetrometer - strength indicator (pounds per square foot)
UC	Unconfined Compression (pounds per square foot)
-200	Percent Passing No. 200 sieve by weight
FS	Percent Free Swell

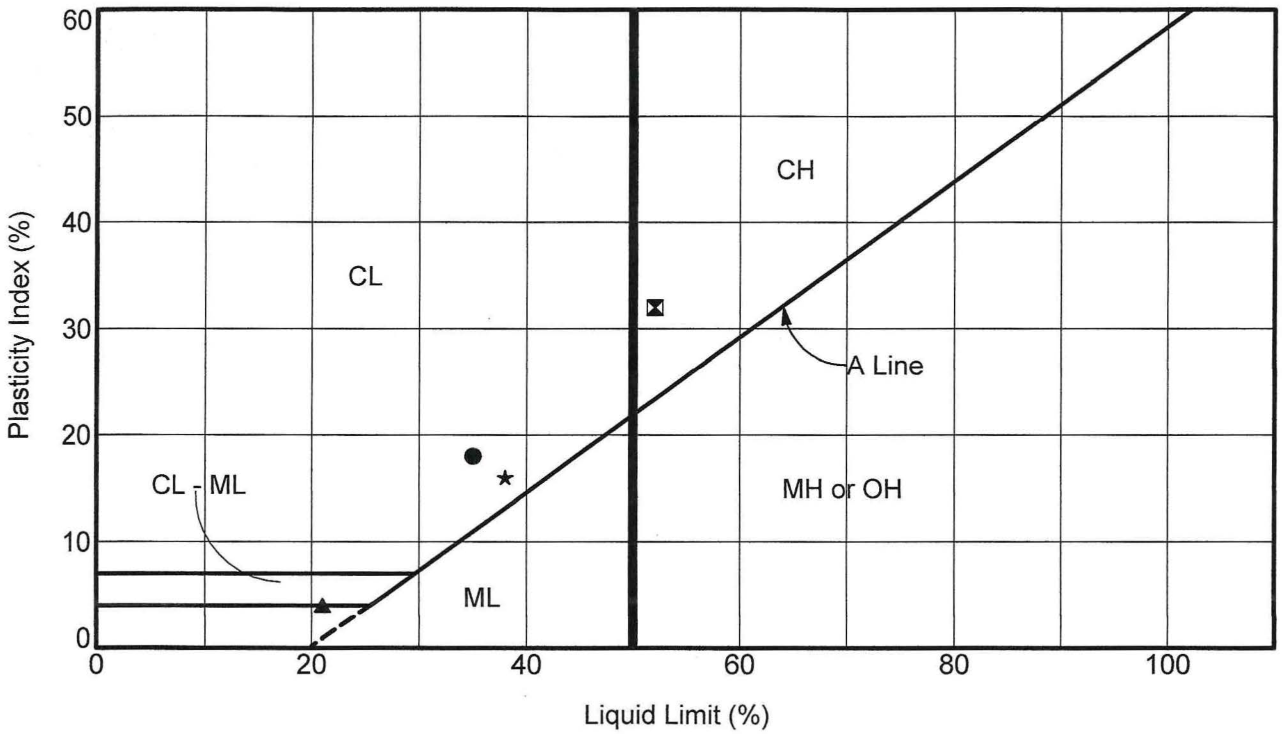
REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 2544.1.1 Date: 04-20-22 Appr: TJC	LABORATORY TEST DATA MELCON RESIDENCES 2485 MIDDLE TWO ROCK ROAD PETALUMA, CALIFORNIA	PLATE 4b

PIT NUMBER	DEPTH	TEST TYPE*	TEST RESULTS
7	1.0	UC(P)	1250
	2.0	UC(P)	2500
	2.5	UC(P)	4500+
8	1.0	UC(P)	1250
	2.0	UC(P)	> 500
	2.5	M	21.3
	2.5	FS	40
	3.0	UC(P)	3250
	4.0	UC(P)	4500+

*Test Type

M	Moisture Content (percent of dry weight)
MD	Moisture Content (percent of dry weight)/dry density (pounds per cubic foot)
UC(P)	Penetrometer - strength indicator (pounds per square foot)
UC	Unconfined Compression (pounds per square foot)
-200	Percent Passing No. 200 sieve by weight
FS	Percent Free Swell

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 2544.1.1 Date: 04-20-22 Appr: TJL	LABORATORY TEST DATA MELCON RESIDENCES 2485 MIDDLE TWO ROCK ROAD PETALUMA, CALIFORNIA	PLATE 4c



ASTM D 4318-98

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)
●	**VERY DARK GRAY SANDY CLAY (CL)** **Test Pit 1 at 3.5 feet**	**35**	**17**	**18**	**50**
⊠	**MOTTLED DARK GRAY-BROWN AND ORANGE SANDY CLAY (CH), with occasional fine gravel** **Test Pit 1 at 5.5 feet**	**52**	**20**	**32**	**80**
▲	**VERY DARK BROWN CLAYEY FINE SAND (SC)** **Test Pit 6 at 1.0 feet**	**21**	**17**	**4**	**35**
★	**MOTTLED VERY DARK GRAY AND ORANGE VERY CLAYEY FINE SAND (SC)** **Test Pit 8 at 2.5 feet**	**38**	**22**	**16**	**40**

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 2544.1.1

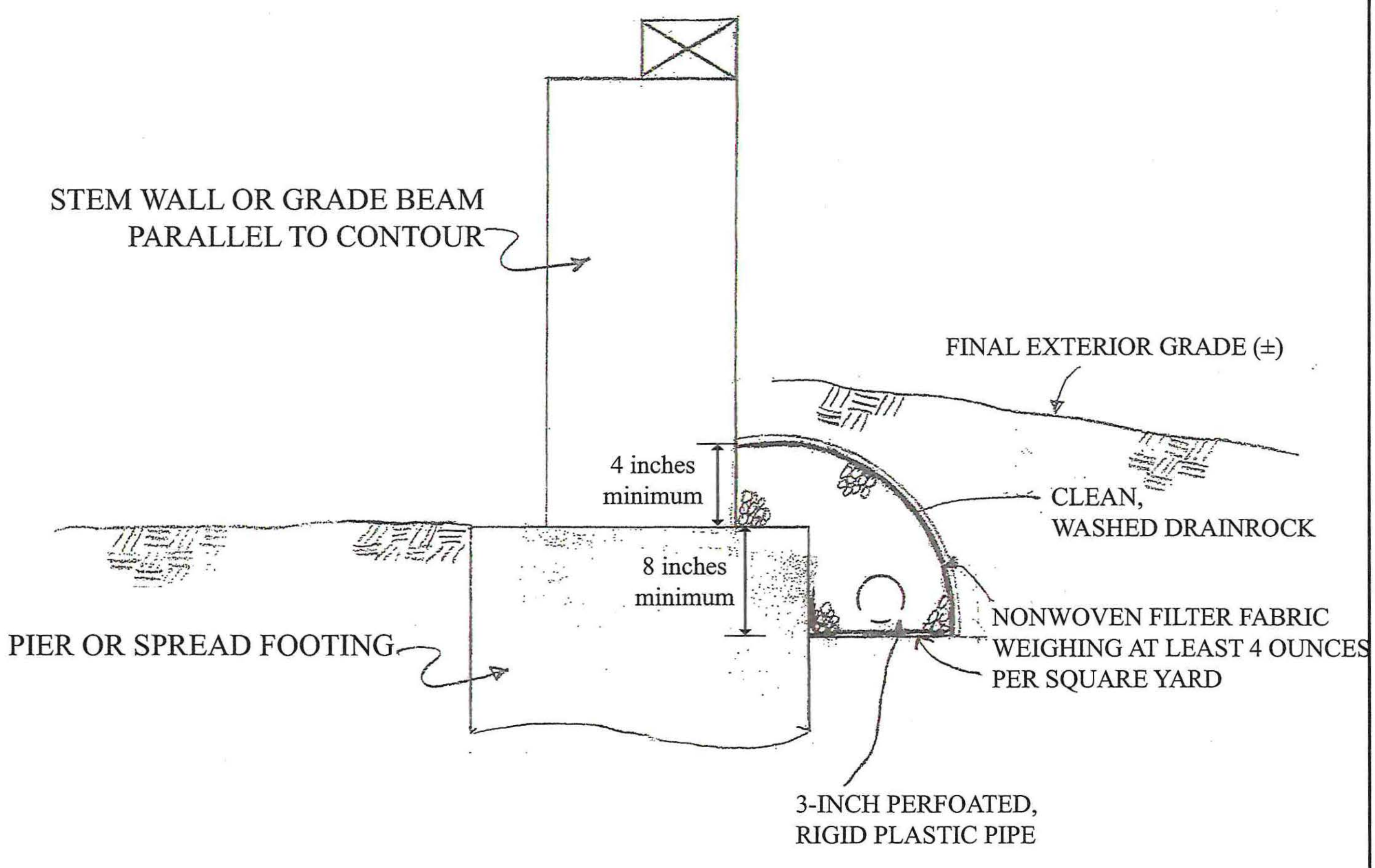
Date: 2-22-22

Appr: TR

ATTERBERG LIMITS TEST RESULTS

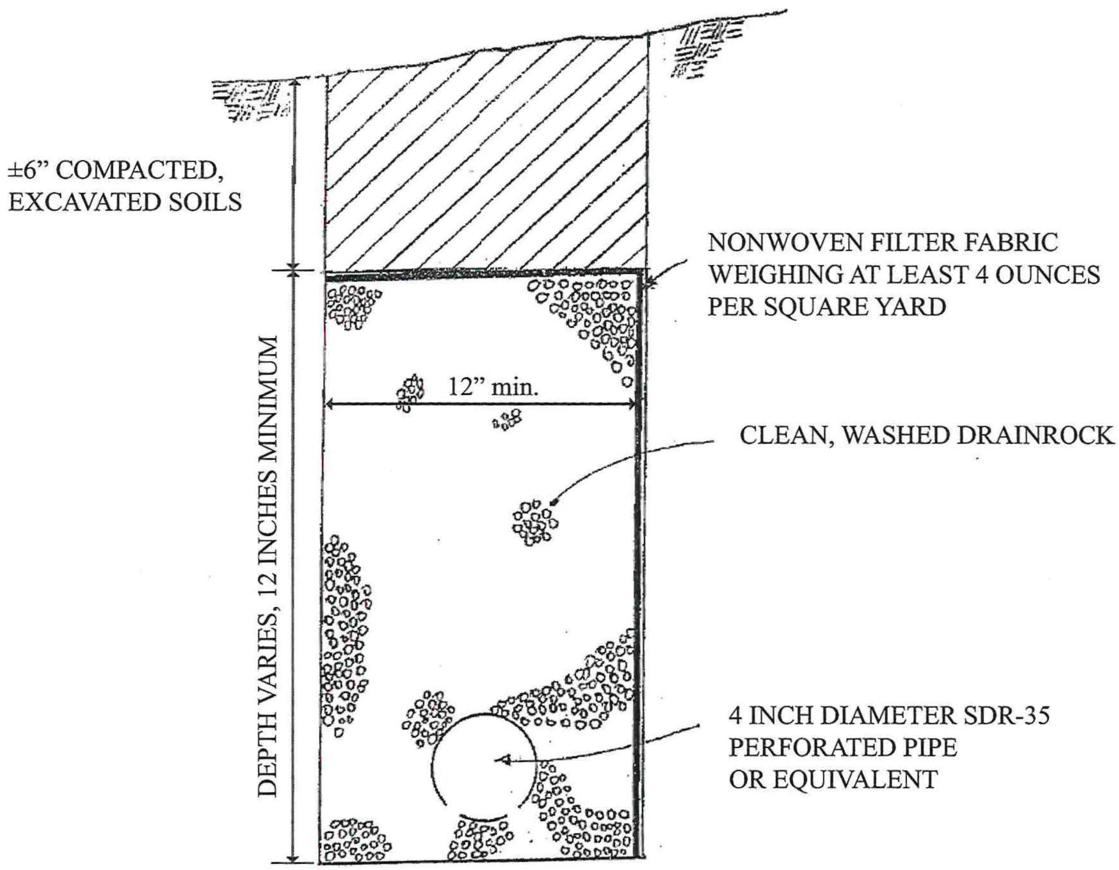
MELCON RESIDENCES
PETALUMA, CALIFORNIA

PLATE
5



FOUNDATION SUBDRAIN
(NOT TO SCALE)

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 2544.1.1 Date: 03-03-2022 Appr: TJL	TYPICAL CROSS-SECTION FOUNDATION SUBDRAIN MELCON RESIDENCES 2485 MIDDLE TWO ROCK ROAD PETALUMA, CALIFORNIA	PLATE 6



TRENCHED INTERCEPTOR SUBDRAIN
(NOT TO SCALE)

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 2544.1.1 Date: 03-03-2022 Appr: FJL	TYPICAL CROSS-SECTION TRENCHED INTERCEPTOR SUBDRAIN MELCON RESIDENCES 2485 MIDDLE TWO ROCK ROAD PETALUMA, CALIFORNIA	PLATE 7