BAUER associates, inc.
GEOTECHNICAL CONSULTANTS

September 4, 2020
Job No. 3745.0

Tanita Choudhury
1417 Wood Street
Oakland, CA 94607

Report
Geotechnical Investigation
Planned Residence
22147 Ruoff Road
Jenner, California

This report presents the results of our geotechnical investigation for the planned residence at the subject property. The planned development location is shown the Site Plan, dated June 30, 2020, prepared by Haddock Studio. The plan is partially reproduced and presented on Plate 1.

We understand that the site will be developed with a one-story, wood framed residence with raised wood floors. The residence will be comprised of separate units. Foundation loads are expected to be typical for the type of construction indicated. Associated development includes a detached carport, porch and patio, walkways, and outdoor improvements (i.e. hot tub, welcome bar, etc.). We anticipate that site grading will be relatively minor with unretained cuts and fills less than about 2 feet. Retaining walls of less than about 6 feet could be part of the design.

The scope of our investigation, as outlined in our July 15, 2020, agreement included reviewing selected published geologic information from our files, exploring subsurface conditions at the site, and performing laboratory testing on selected samples. Based upon our work, we have developed conclusions and recommendations concerning:

1. Proximity of the site to published active faults.

2. Soil/rock and ground water conditions observed.

3. Site preparation and grading.

4. Foundation type(s) and design criteria.

5. Concrete slab-on-grades.

6. Retaining walls.

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6470 Mirabel Road
Post Office Box 460
Forestville, CA 95436
707.887.2505

7. Geotechnical engineering drainage.

8. Supplemental services.

Our scope of work did not include an evaluation of any potential hazardous waste contamination or corrosion potential of the soil or groundwater at the site. Further, our scope of work did not include evaluating areas beyond the described improvement area (i.e. driveway, etc.).

WORK PERFORMED

We reviewed the following selected geotechnical data:

Blake, M.C., Jr., Graymer, R.E. Stamski, 2002, Geologic Map and Map Database of Western Sonoma, Northernmost Marin, and Southernmost Mendocino Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF- 2402, Version 1.0, Scale 1:100,000.

California Geological Survey, 2018, revision, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California: Special Publication 42,
https://www.conservation.ca.gov/cgs/Documents/Publications/SP_042.pdf

California's Office of Statewide Health Planning and Development (OSHPD), Seismic Design Maps Web Application, 2019.

California Building Code, 2019, California Building Standard Commission.

Huffman, M.E., and Armstrong, C.F., 1980; Geology for Planning in Sonoma County: California Division of Mines and Geology, Special Report 120, Scale 1:62,500.

On August 4, 2020, our geologist explored the subsurface conditions at the site to the extent of five test pits. The test pits were excavated with a Kubota KX-057 excavator equipped with a 30-inch wide bucket. The completed test pits were excavated to depths ranging to about 8-½ feet. The test pit locations, as approximately shown on Plate 1, were located by our geologist by estimating distances from features indicated on the available plan. The test pit locations should be considered accurate only to the degree implied by the method used. Our geologist logged the conditions exposed and obtained samples at selected intervals for visual identification. Logs of the test pits showing the materials encountered are presented on Plates 2 through 4. The materials are classified in accordance with the Unified Soil Classification System and Rock Classification Criteria, presented on Plates 5 and 6, respectively.

The logs show our interpretation of the subsurface conditions on the date and locations indicated, and it is not warranted that they are representative of the subsurface conditions at other locations and times. Also, the stratification lines on the logs represent the approximate boundaries between material types; the transition may be gradual. The test pits were not backfilled with compacted fill and may settle. Test pits in development areas must be properly filled during construction.

Representative samples of the soils encountered were laboratory tested to determine their classification (Atterberg limits, percent passing No. 200 sieve). The test results are presented on the logs in the manner described in the Key to Test Data, Plate 5.

SITE CONDITIONS

The property is located on a southerly sloping hill located between Ruoff Road on the south side, and Koftinow Road to the north side of the property. The Test Hole Location Plan, Plate 1, shows the project site (approximate Google Earth coordinates: 38.546738º N; -123.282580º W). The site is currently undeveloped except for a dirt driveway entering to property from Koftinow Road. The property slopes to the south from about 5:1 (horizontal to vertical) to about 8:1 with steeper slopes on the southwest portions near a small drainage gully. The property is covered with a dense redwood forest and scattered brush.

The published geologic maps reviewed indicate that the site is underlain by sandstone of the German Rancho Formation mapped as well-bedded sandstone, mudstone and conglomerate.

The results of our subsurface exploration, summarized on Plates 2 through 4 indicate that the surface soils at the site typically consist of 1-1/2 to 2-1/2 feet of loose silty sands. The surface sands are typically underlain by medium dense to dense clayey sands, and stiff sandy clays that are weak and porous to about 2 to 2-½ feet. Based on our observations and laboratory testing, these sandy soils have a low to moderate expansion potential. However, the sandy clay encountered in Test Pit 4 is considered to be of high expansion potential. The weak surface soils are prone to consolidation and/or collapse when saturated under a load. The depth of weak soils is indicated on the test pit logs.

Underlying the weak sandy and clay soils in most of our test pits is dense clayey sand. The sandy soils are underlain by sandstone that is closely fractured, moderately hard, and moderately weathered. Bedding orientation of the sandstone was not discernible in our test pits. Practical excavation refusal was encountered at the bottoms of test pits resulting in 2-inch grooves in the rock with the equipment used. The sandy soils and bedrock underlying the weak materials are considered only slightly compressible for the anticipated building loads.

Groundwater was not encountered within the test holes. Rainwater typically percolates through the surface soils and migrates downslope as seepage at the interfaces of the surface soils with bedrock, and through fractures, discontinuities, and coarse-grained layers and lenses in the bedrock. Groundwater conditions are expected to vary seasonally and at different locations. We have previously observed shallow perched water can be encountered at the ground surface, particularly during the winter and spring months.

The published geologic maps reviewed do not indicate landsliding at the site. Further, we did not observe landsliding during our exploration.

The published geologic maps do not indicate active faults at this site. The property is not within an Alquist-Priolo (AP) Earthquake Fault Zone, which could require a detailed investigation to evaluate the hazard of fault surface rupture in relation to nearby active faults. The nearest fault zones considered seismically active (experiencing surface rupture within about the last 11,000 years) is the San Andreas fault, located about ½ mile to the northeast.

DISCUSSION AND CONCLUSIONS

Based on the results of our investigation, we conclude that the planned development is feasible from a geotechnical engineering viewpoint. The primary geotechnical concern is the presence of relatively weak surface soils subject to downslope creep and expansive soils.

Upon saturation, weak/porous surface soils and variable density old fills will lose strength and/or consolidate rapidly under loads of new fill and structural elements. Saturation will occur when the natural evaporation of soil moisture is inhibited by new fill and structural elements. Expansive soils undergo significant volumetric changes with seasonal variations in moisture content. Such movements can result in unacceptable heaving and cracking of lightly-loaded structural elements, such as foundations, pavements and concrete slabs.

Suitable foundation support can be achieved by deepening foundations (i.e., deepened spread footings or drilled, cast-in-place concrete piers) to penetrate through the weak soils and the zone of seasonal moisture variation (about 3 feet) into more suitable underlying materials. Alternatively, shallow spread footings placed entirely on engineered select fill could be used.

To mitigate against the detrimental effects of slow downhill creep, as is typical on Sonoma County hillsides, it will also be necessary to extend foundation support through the weak soils and into firm soils or bedrock. Deepened foundations could consist of either deepened spread footings or cast-in-place concrete drilled piers. If deepened foundations are used, then interior slabs (if any) must be designed as structural slabs. It will be necessary to design foundations to resist lateral forces caused by downhill creep of the soils on slopes steeper than 8:1. Alternatively, weak soils could be removed and replaced as engineered fill. Shallow foundations could then be used for structure and slab support. Where fills are placed slopes are 10:1 or steeper, keyways and benches will be required.

Weak surface soils will consolidate under slab-on-grade floors. Expansive materials, when encountered, can result in heaving and cracking of slabs. Typically, living area or similar 'critical-use' slabs should be: 1) structurally supported on the foundation system and provided with a void beneath the slab to allow for uplift; or 2) weak/porous surface soils must be upgraded in building areas by removal and recompaction for their full depth; and 3) a minimum 30-inch thick confining and moisture protecting blanket of imported, non-expansive ("select") fill must be placed.

Less critical slabs (such for garage or exterior areas) may be constructed on properly prepared subgrade provided that: 1) the slabs are separated from foundations; 2) slabs are designed to minimize cracking (i.e. reinforced and provided with control joints); and 3) moderate to severe soil related cracking and movement is considered acceptable. Improved performance of slabs could be attained by removal and replacement of some, or all, of the weak and expansive soils with non-expansive engineered fill.

Control of surface run-off will significantly enhance the stability of the site. Generally, the introduction of water into soils can cause soil instability and should be avoided. The site must be graded to reduce ponding and provide positive drainage away from the building foundations. Roof gutter downspouts must be collected into non-perforated pipes and discharged into the site storm drainage, or onto concrete slabs-on-grade or asphalt pavements that drain away from the foundations. Underfloor areas should be sloped to drain and provided with outlets. Trench subdrains should be installed beneath the slab rock at slab-on-grade floors to reduce the risk of water build up in the slab rock. Foundation subdrains should be installed on the upslope side of structures.

Groundwater was not encountered in our test pits. However, groundwater conditions are expected to vary. Excavations performed in the summer or autumn months will typically result in a lower risk of encountering groundwater.

The published geologic and slope stability maps do not indicate the presence of landslides at the site and we did not observe evidence of landsliding at the planned building area during our work. The recommendations presented in this report have been prepared to mitigate the hazard of shallow landsliding. Landsliding, sloughing, and erosion must be repaired promptly before enlargement can occur.

The published geologic maps do not indicate active faults on the site, therefore the risk of fault rupture during earthquakes is considered to be low. As common throughout Northern California, the site will be subject to severe shaking as result of earthquakes along active faults in the region. We anticipate that the intensity of earthquake shaking should be similar to that of other hillside sites in the vicinity. The intensity of future shaking will depend on the distance from the site to the earthquake focus, magnitude of the earthquake, and the response of the structure to the underlying soil and/or rock. Earthquake shaking could induce landsliding and other soil movements. Mitigation of earthquake shaking typically consists of designing and constructing improvements in strict accordance with current standards for earthquake resistant construction.

RECOMMENDATIONS

Site Preparation and Grading

The following is presented for general grading. We must review and approve any grading planned, since site grading may have a negative impact on site stability.

The site should be cleared of designated brush, rubble and debris. Material generated by the clearing operations should be removed from the site. Wells, cesspools, abandoned leach fields, septic tanks excavations and other voids encountered or generated during clearing should be either backfilled with granular material or compacted soil, capped with concrete in accordance with Sonoma County Health regulations and as determined by us.

Areas to be graded should be stripped of the upper soils containing root growth and organic matter. We anticipate that the required depth of stripping will average about 3 to 6 inches. Deeper stripping will be required to remove localized heavy concentrations of root growth. The strippings should be removed from the site, stockpiled for reuse as topsoil, or mixed with at least two parts soil and used as fill in areas 10 feet beyond the proposed improvements.

For the purpose of definition, "select fill areas" referred to in the remainder of this report are buildings with shallow foundations and critical concrete slab and the zones extending for a distance of at least 5 feet beyond outside edges of slabs or other footings extending from structures. In select fill areas, weak surface soils and variable density old fills should be excavated for their full depth (2 to 2-½ feet based on our test pits). Where expansive soils are encountered, additional excavation will be necessary to provide space for a minimum of 30 inches of select fill. Where deepened foundations and either raised wood floors or structural slabs are used, overexcavation of weak soils will not be required.

Following clearing, stripping and planned excavations, weak surface soils and variable density old fills (if present) should be excavated for their full depth within select fill areas to expose firm soils. Where fills will be placed on terrain sloping steeper than about 10:1, regardless of whether or not they are within select fill areas, the surface to receive fill should be prepared by cutting level keyways and benches extending into firm materials, as determined by us during excavation. The excavation should extend to at least 5 feet beyond the edges of critical slabs. Subsurface drainage facilities should be installed at the rear of keyways as recommended by us.

If isolated deeper zones of soft, saturated, dry (shrinkage cracks), highly porous or organic soils are encountered during excavation and recompaction, the soils should be removed to expose firm soils. The depth and extent of excavations and overexcavations should be approved in the field by us.

On-site expansive soils will not be suitable as select fill, unless lime treated and provided that: 1) all rock sizes greater than 6 inches in largest dimension and perishable materials are removed; 2) the fill materials are approved by us prior to use. Expansive on-site materials, as encountered, should not be used within within 3 feet of finished slopes. Upon request, we can provide supplemental consultation regarding lime treating. Imported fill, if required, should be free of organic matter, non-expansive and should generally conform to the following requirements:

Sieve Size	Percent Passing
6-inch	100
4-inch	90-100
No. 200	15-60

Liquid Limit - 40 Maximum
Plasticity Index - 15 Maximum
(ASTM D 4318-17 Wet Test Method)

Fill should be placed in thin lifts (normally 6 to 8 inches depending on compaction equipment), uniformly moisture conditioned to 2 percent above optimum (4 percent for high expansive soils), and compacted to at least 90 percent relative compaction. Where critical slabs are underlain by cut and fill, or for differential fill thicknesses greater than 3 feet, the fills should be compacted to at least 93 percent relative compaction. All surfaces should be finished to present a smooth, unyielding subgrade. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil, as determined by ASTM D 1557-12. Optimum moisture content is the water content (percentage by dry weight) corresponding to the maximum dry density.

In general, cut and fill slopes should be constructed no steeper than 3:1. Fill slopes may be constructed up to 2:1 where non-expansive fill is used in the outer 3 feet of the slope face. Graded slopes should be planted with quick growing, dense vegetation or protected from erosion by other measures upon completion of grading. Seepage and surface water runoff should be intercepted and diverted away from the slope surfaces.

Temporary cut slopes of 1:1 may be used for planning purposes, but must be reviewed in the field by us. The tops of the cut slopes should be rounded back to 2:1 in the weak soil zones. Depending on the exposed subsurface conditions, presence of groundwater seepage and the time of year when grading is performed, temporary cut slopes may need to be excavated to 1-1/4:1 or 1-1/2:1.

At all times, temporary construction excavations should conform to the regulations of the State of California, Department of Industrial Relations, Division of Industrial Safety or other stricter governing regulations. Temporary cut slopes are the responsibility of the contractor/owner.

Foundations

As discussed previously, considering the site conditions, we anticipate that either spread footings or drilled piers are suitable foundation types for the planned structure. Combination of foundation types should not be used on individual structures.

Spread Foundations

Spread footings should be at least 12 inches wide, 12 inches deep, and penetrate at least 12 inches into firm soils or bedrock below the weak soils or engineered fill. Weak soils range to about 2-½ feet in our test pits. Footings for individual structures should be supported by only one material type to reduce the potential for differential settlement. Perimeter wall footings should be continuous. Footings bottoming in moderately to highly expansive soils should be at least 36 inches deep. Where the depth to firm material is greater than planned foundation depths specified by the structural engineer, the additional footing depth may be backfilled with lean (2-sack) concrete to within 18 inches of lowest adjacent grade. The footings should be stepped as necessary to produce level tops and bottoms. Footings should be deepened as necessary to provide at least 7 horizontal feet of confinement between the footing bottoms and the face of the nearest slope.

Spread footings bearing into firm soils can be designed using an allowable bearing pressure of 2,000 and 3,000 pounds per square foot (psf) for dead plus long-term live loads and total design loads, respectively. We should observe the footing excavations prior to the placement of reinforcing steel and concrete.

The portion of the foundations extending into firm soils may impose a passive equivalent fluid pressure of 350 pounds per cubic foot (pcf), triangular distribution, and a friction factor of 0.35 times the net vertical dead load. Passive pressures should be neglected within the upper 1 foot, unless footings are confined by other construction.

In areas where the weak soils have not been removed on slopes steeper than about 8:1, the footings should be designed and reinforced to resist creep forces exerting an active equivalent fluid pressure of 55 pcf acting on the face of foundations. The actual depth of the creep prone soils will vary and depend on the grading performed. The design must be reviewed and revised if the depth of soils exposed during construction are different than used for design. Where upslope-downslope footings are spaced wider than 15 feet, tie-beams should be provided to redistribute stresses imposed by the creeping soils.

Drilled Pier and Gradebeam Foundations

Foundation support for the structure and retaining walls can be obtained by drilled piers interconnected with gradebeams. Piers should have a minimum diameter of 14 inches, be at least 10 feet deep and extend at least 4 feet into bedrock, as determined by us. The portion of the pier extending into bedrock can impose 750 psf in skin friction. The upper portion of piers within the weak soils should be neglected in design. Further, piers should have at least 7 feet of horizontal confinement from the top of the supporting zone to the adjacent slope face. Pullout capacity of the piers should be considered as one-half the downward capacity. End bearing should be neglected because of the difficulty of cleaning out the pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously. Piers should not be located closer than three pier diameters, center to center.

All piers should be interconnected with gradebeams or slabs designed to support the design structural loads per current code requirements. Where upslope-downslope footings are spaced wider than 15 feet, tie-beams should be provided to redistribute stresses imposed by the creeping soils. Piers should be reinforced full length with reinforcing extending into the gradebeams.

The piers and gradebeams should be designed and reinforced to resist creep forces exerting an active equivalent fluid pressure of 55 pcf acting on two pier diameters and the face of foundations. The depth of creep prone soils for design is estimated at 4 feet. The actual depth of the creep prone soils will depend on the grading performed and the proximity of the structures to our test holes. If desired, a schedule of pier sizing and reinforcing could be provided by the project structural engineer on the project plans for the various anticipated depths of weak soils. The design must be reviewed and revised by us if the depth of soils exposed during construction are different than used for design.

The portion of the piers extending into firm soil and bedrock may impose a passive pressure of 350 pcf acting on two pier diameters. Passive pressure should be limited to a maximum of 3,000 psf. Passive pressure should be neglected within the upper 1 foot of pad grade unless confined by other construction.

If caving soils are encountered, it may be necessary to case the holes. If groundwater is encountered, it will be necessary to dewater the holes and/or place the concrete by the tremie method.

We encountered excavation refusal at the soil/bedrock interface. Hard rock conditions may be encountered. We recommend that the pier drilling contractor review this report to determine the appropriate drilling equipment.

The pier holes should contain no more than 3 inches of slough. The remaining slough should be tamped with a heavy timber or similar prior to concrete placement to prevent wet concrete from settling. Concrete should be placed in pier excavations promptly to avoid soil desiccation. Excess concrete, where encountered, must be trimmed to plan dimensions from the bottoms of gradebeams and tops of piers to reduce uplift pressures.

We should observe the start of pier drilling operations to note the conditions exposed and provide recommendations to the contractor. We should observe the completed pier excavations prior to the placement of reinforcing steel and concrete. We should be contacted to provide special inspection of reinforcing steel and concrete for the piers, gradebeams, and retaining walls.

Seismic Design Criteria

Using Google Earth site latitude and longitude coordinates of 38.546738º N; -123.282580º W, respectively, the following seismic design criteria is based on 2019 CBC guidelines, ASCE 7-16 and the USGS Earthquake Ground Motion Parameters:

Spectral Response Acceleration	g
S_S (0.2 sec.)	2.386
S_1 (1.0 sec.)	1
S_{DS} (0.2 sec.)	1.909
S_{D1} (1.0 sec.)	0.933
Peak Ground Acceleration (PGA)	1.02
Seismic Design Category	E

Title 24, Part 2, Section 1613.2.2, of the 2019 CBC indicates that site categorization for seismic design should be based on the average soil values within the upper 100 feet of the site. Although the scope of our investigation was limited to relatively shallow test pits (ranging to about 8-½ feet deep), we estimate that a Site Classification "C" will be appropriate for design. Upon request, we could perform supplemental calculations or exploration to determine the site specific subsurface conditions ranging to 100 feet.

Retaining Walls

Foundation support for retaining walls can be obtained from spread footings or drilled piers in accordance with the previous section recommendations.

Retaining walls free to rotate (yield more than 0.1 percent of the wall height at the top of the backfill) and with backfill flatter than 3:1 should be designed to resist an active lateral earth pressure (triangular distribution) of 40 pcf. An active lateral earth pressure of 50 pcf should be used for slopes inclined from 3:1 to 2:1. Rigid walls which cannot yield should be designed for an "at-rest" lateral earth pressure of 60 pcf. These pressures do not consider additional loads resulting from adjacent foundations, traffic loads, or other downward loads. If additional surcharge loadings are anticipated, we can assist in evaluating their effects.

A minimum factor of safety of 1.5 against overturning and sliding should be used in the design of retaining walls. Where applicable, seismic wall stability may be evaluated based on a uniform lateral earth pressure of 17xH psf (where H is the height of the wall in feet). This pressure is in addition to the active equivalent fluid pressures presented in this report. This force should be considered to act at a height of 0.33H on the wall. For restrained walls, seismic pressures may be assumed to act in combination with active rather than at-rest earth pressures. The factor of safety against instability under seismic loading should be at least 1.1.

Retaining walls should be provided with permanent backdrains to prevent the build-up of hydrostatic pressure. The drains and backfill should be constructed as shown on Plate 7. The top of the perforated drainage pipe should be located at least 8 inches below any adjacent interior slabs to reduce the risk of seepage through walls into interior building areas.

Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed as specified by the Project Architect or Structural Engineer.
Backfill materials should be compacted in a manner to prevent over-stressing the wall. Further, wall bracing should be considered. Retaining walls will yield slightly during backfilling. Therefore, retaining walls should be backfilled prior to building on or adjacent the walls. On-site expansive soils, if encountered, may be used as backfill only outside of the zone defined by a 1:1

projection from the top of the footing. The use of imported granular material will generally require less backfilling effort. We should be contacted to observe the backfill of retaining walls.

We typically recommend that foundations not be located within retaining wall backfills to avoid the potential for differential settlement. Mitigation may include designing foundations to span from retaining walls to beyond the backfill area. We should be contacted to provide supplemental consultation if foundations will extend across retaining wall backfills.

Concrete Slab-On-Grade

Non-critical slabs (such as for exterior and storage areas) may be constructed on properly prepared subgrade provided that: 1) the slabs are separated from foundations; 2) slabs are designed to minimize cracking (i.e. reinforced and provided with control joints); and 3) some soil related cracking and movement is considered acceptable. Improved performance of slabs could be attained by removal and replacement of some, or all, of the weak soils with non-expansive engineered fill.

Slabs should be underlain with a capillary moisture break and cushion layer consisting of at least four inches of clean, free-draining crushed rock. The crushed rock should be at least 1/4-inch, and no larger than 3/4-inch, in size.

Moisture will condense on the underside of slabs. Where moisture migration through slabs is detrimental, waterproofing methods and specifications should be determined by others for incorporation into the project plans. Slabs should be at least 4 inches thick and reinforced to reduce cracking. Exterior and utility area slabs should be carefully separated from foundations with felt paper, mastic, or other positive and low friction separation.

Some cracking of slabs must be anticipated considering concrete shrinkage. Reinforcing must be carefully installed in accordance with the structural engineer's recommendations to minimize the potential of cracking. We typically recommend the use of steel rebar reinforcement (rather than welded wire mesh) as directed by the structural engineer. We have previously observed that wire mesh is often not properly located in the slabs.

Geotechnical Engineering Drainage

Ponding water will be detrimental to foundations, therefore the site should be graded to provide positive drainage away from improvements and slopes.

Where crawl spaces are used, the crawl space areas should be sloped to drain and provided with outlets through foundations. Outlets should be provided in the slab rock if concrete slab-on-grade floors are constructed, to reduce the risk of water build up in the slab rock. Increased mitigation should be provided by the installation of trench subdrains beneath the slab rock. The subdrains should consist of 12-inch deep by 12-inch wide trenches that cross the slab area, as directed by us. The slab rock should be connected to the subdrain rock. The pipe, rock, and fabric should conform to those described below. The pipe should consist of PVC Schedule 40 or ABS with a SDR of 35 or better. The trench should be backfilled with clean, free-draining, 3/4 or 1-1/2-inch crushed drain rock, separated from adjacent soil/rock by a non-woven filter fabric. As an alternative, Class II permeable material complying with Caltrans Section 68, may be used without filter fabric.

Area drainage should be connected to the site storm drain system discharging in erosion resistant areas well away from the structures and slopes. Surface drains must be maintained entirely separate from subsurface drainage. As requested, we can assist in providing suitable drainage discharge locations.

Supplemental Services

We should be contacted during design to discuss our recommendations and the design approach. We should review the final plans for conformance with the intent of our recommendations.

During grading and foundation construction, we should provide intermittent geotechnical engineering observations, along with necessary field and laboratory testing, during: 1) removal of weak soils; 2) fill placement and compaction; 3) preparation and compaction of subgrade; and 4) excavation of foundations. These observations and tests would allow us to check that the contractor's work conforms with the intent of our recommendations and the project plans and specifications. These observations also permit us to check that conditions encountered are as anticipated, and modify our recommendations, as necessary. Upon completion of the project, we should perform a final observation prior to occupancy. We should summarize the results of this work in a final report.

These supplemental services are performed on an as-requested basis, and we can accept absolutely no responsibility for items that we are not notified to observe. These supplemental services are in addition to this investigation and are charged for on an hourly basis in accordance with our Schedule of Charges. We must be provided with at least 48 hours notice for scheduling our initial site visit, and 24 hours thereafter.

LIMITATIONS

We performed the investigation and prepared this report in accordance with generally accepted standards of the geotechnical engineering profession. No other warranty, either express or implied, is given.

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. Site conditions and standards of practice change. Therefore, we should be notified to update this report if construction is not performed within 18 months of the submittal date.

We trust this provides the information you require at this time. If you have questions or wish to discuss this further, please call.

Very truly yours,

BAUER ASSOCIATES, INC

Steven J. Klick
Engineering Geologist

CERTIFIED ENGINEERING GEOLOGIST
Steven J Klick
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Arthur H. Graff
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
ARTHUR H. GRAFF
2319
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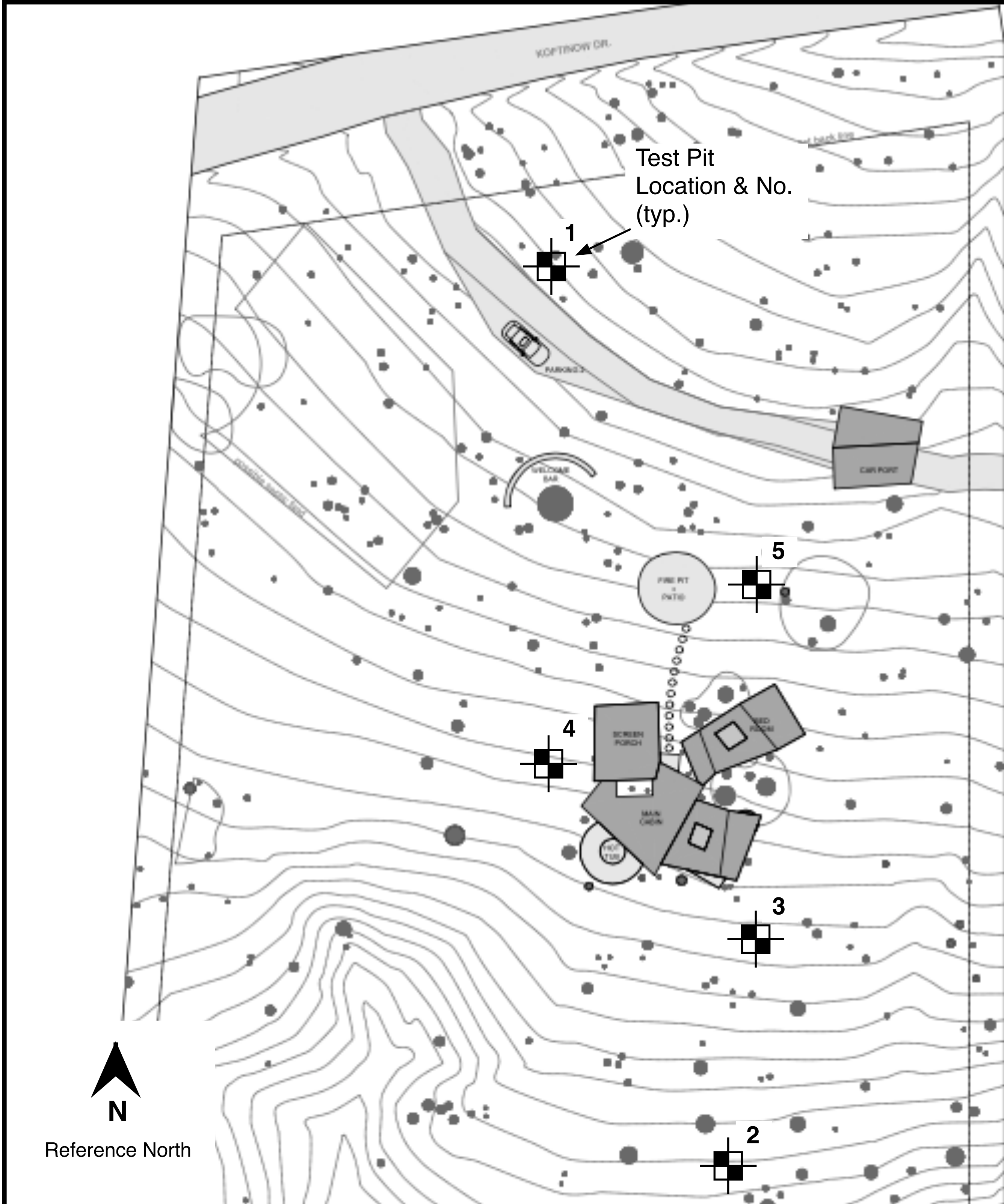
GEOTECHNICAL
STATE OF CALIFORNIA

SJK/AHG (gi/ruoff rd)
Attachments: Plates 1 through 7
Email only



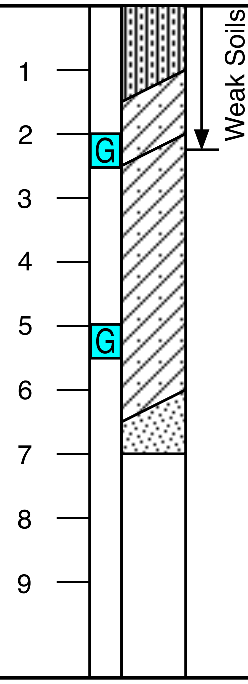
Reference: Site Plan dated September 1, 2020, prepared by Haddock Studio
Note: The locations of all features are approximate and may vary.

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 3745.0 Date: 9/20 By: SJK	**TEST HOLE LOCATION PLAN** 22147 RUOFF ROAD Jenner, California	PLATE **1**

LOG OF TEST PIT 1

Equipment:	Kubota KX-057
Date:	August 4, 2020
Elevation:	Not Available

Laboratory Tests | Moisture Content (%) | Dry Density (pcf) | Depth



GRAY SILTY SAND (SM)
loose, moist, roots

ORANGE BROWN CLAYEY SAND (SC)
medium dense, moist, roots

ORANGE BROWN CLAYEY SAND (SC)
dense, moist

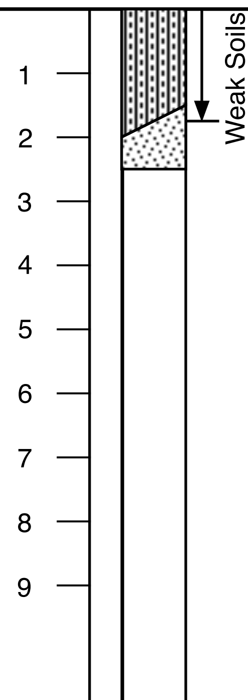
BROWN SANDSTONE - German Rancho Fm.
close fractures, moderately hard, moderate weathering

Practical refusal at 7 feet
No free water encountered

LOG OF TEST PIT 2

Equipment:	Kubota KX-057
Date:	August 4, 2020
Elevation:	Not Available

Laboratory Tests | Moisture Content (%) | Dry Density (pcf) | Depth



GRAY SILTY SAND (SM)
loose, moist, roots

BROWN SANDSTONE - German Rancho Fm.
close fractures, moderately hard, moderate weathering

Practical refusal at 2-1/2 feet
No free water encountered

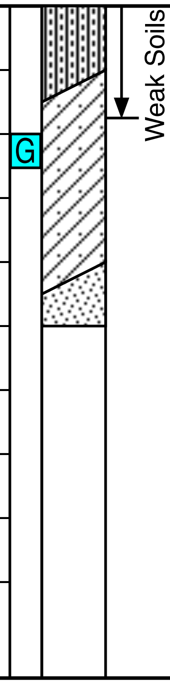
BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 3745.0 Date: 8/20 By: SJK	LOG OF TEST PITS 1 & 2 — 22147 RUOFF ROAD Jenner, California	PLATE 2

LOG OF TEST PIT 3

Equipment: Kubota KX-057
Date: August 4, 2020
Elevation: Not Available

Laboratory Tests | Moisture Content (%) | Dry Density (pcf) | Depth

Depth: 1, 2, 3, 4, 5, 6, 7, 8, 9



GRAY SILTY SAND (SM)
loose, moist, roots

ORANGE YELLOW CLAYEY SAND (SC)
dense, moist, roots and porous in upper 1 foot

BROWN SANDSTONE - German Rancho Fm.
close fractures, moderately hard, moderate weathering

Practical refusal at 5 feet
No free water encountered

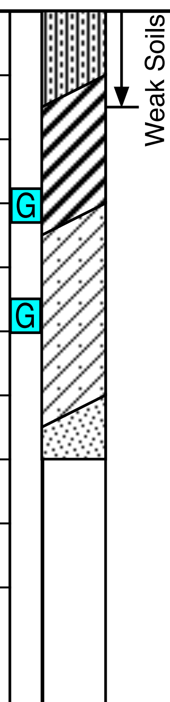
LOG OF TEST PIT 4

Equipment: Kubota KX-057
Date: August 4, 2020
Elevation: Not Available

Laboratory Tests | Moisture Content (%) | Dry Density (pcf) | Depth

Atterberg Limits
LL = 51
PL = 18
PI = 33
Minus No. 200
= 58 percent

Depth: 1, 2, 3, 4, 5, 6, 7, 8, 9



GRAY SILTY SAND (SM)
loose, moist, roots

ORANGE YELLOW SANDY CLAY (CH)
stiff, moist

ORANGE YELLOW CLAYEY SAND (SC)
dense, moist

BROWN SANDSTONE - German Rancho Fm.
close fractures, moderately hard, moderate weathering

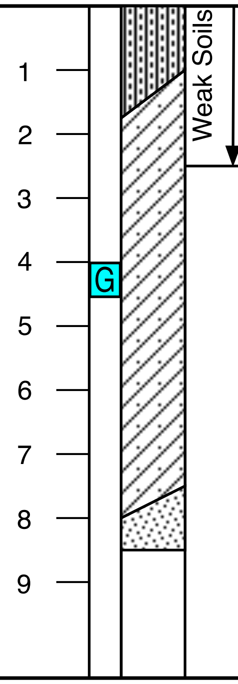
Practical refusal at 7 feet
No free water encountered

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 3745.0 Date: 8/20 By: SJK	**LOG OF TEST PITS 3 & 4** 22147 RUOFF ROAD Jenner, California	PLATE **3**

LOG OF TEST PIT 5

Equipment: Kubota KX-057
Date: August 4, 2020
Elevation: Not Available

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Depth



GRAY SILTY SAND (SM)
loose, moist, roots

YELLOW ORANGE CLAYEY SAND (SC)
dense, moist, porous with abundant roots in upper 1-1/2 foot

BROWN SANDSTONE - German Rancho Fm.
close fractures, moderately hard, moderate weathering

Practical refusal at 8-1/2 feet
No free water encountered

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 3745.0 Date: 8/20 By: SJK	**LOG OF TEST PIT 5** 22147 RUOFF ROAD Jenner, California	PLATE **4**

MAJOR DIVISIONS					TYPICAL NAMES
COURSE GRAINED SOILS	**GRAVELS** more than half coarse fraction is larger than no. 4 sieve size	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW		WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM		SILTY GRAVELS, POORLY GRADED GRAVEL-SAND MIXTURES
			GC		CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND MIXTURES
	SANDS more than half coarse fraction is smaller than no. 4 sieve size	CLEAN SANDS WITH LITTLE OR NO FINES	SW		WELL GRADED SANDS, GRAVELLY SANDS
			SP		POORLY GRADED SANDS, GRAVEL-SAND MIXTURES
		SANDS WITH OVER 12% FINES	SM		SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC		CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS	**SILTS AND CLAYS** LIQUID LIMIT LESS THAN 50		ML		INORGANIC SILTS, SILTY OR CLAYEY FINE SANDS, VERY FINE SANDS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL		INORGANIC SCLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS OR LEAN CLAYS
			OL		ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH		ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS

KEY TO TEST DATA

LL = Liquid Limit (in %)
PL = Plastic Limit (in %)
PI = Plasticity Index (in %)
-200 = % Passing
-No. 4 = % Passing

	Shear Strength, psf	Confining Pressure, psf	
Tx	320	(2600)	Unconsolidated Undrained Triaxial
Tx CU	320	(2600)	Consolidated Undrained Triaxial
DS	2750	(2600)	Consolidated Drained Direct Shear
UC	2000		Unconfined Compression

Note: All strength tests on 2.4 in. inside diameter sample unless otherwise indicated

SAMPLER GRAPHIC SYMBOLS

Standard California Sampler (2.4 in. ID)

Standard Penetration Test (SPT) (1.4 in. ID)

No Sample Recovery

G Grab Sample

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 3745.0 Date: 8/20 By: SJK	SOIL CLASSIFICATION CHART & KEY TO TEST DATA 22147 RUOFF ROAD Jenner, California	PLATE 5

I. CONSOLIDATION OF SEDIMENTARY ROCKS; usually determined from unweathered samples. Largely dependent on cementation.

U = Unconsolidated
P = Poorly consolidated
M = Moderately consolidated
W = Well consolidated

II. BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness in Feet	Stratification
Massive	greater than 4.0	very thick bedded
Blocky	2.0 to 4.0	thick bedded
Slabby	0.2 to 2.0	thin bedded
Flaggy	0.05 to 0.2	very thin bedded
Shaly or Platy	0.01 to 0.05	laminated
Papery	less than 0.01	thinly laminated

III. FRACTURING

Intensity	Size of Pieces in Feet
Crushed	less than 0.05
Intensely fractured	0.05 to 0.1
Closely fractured	0.1 to 0.5
Moderately fractured	0.5 to 1.0
Occasionally fractured	1.0 to 4.0
Very little fractured	greater than 4.0

IV. HARDNESS

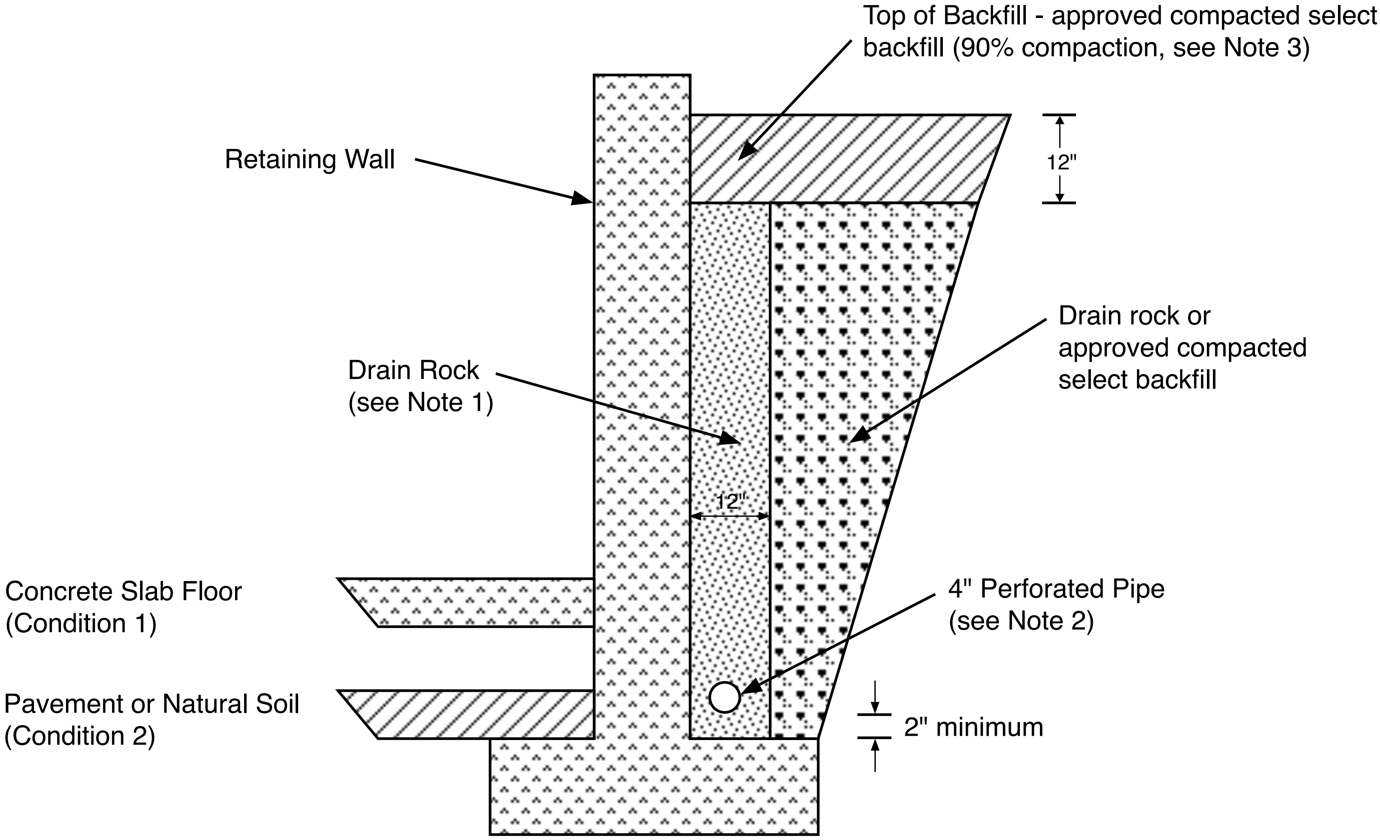
Soft – Reserved for plastic material alone
Low Hardness – Can be gouged deeply or carved easily with a knife blade.
Moderately Hard – Can be readily scratched with a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
Hard – Can be scratched with difficulty; scratch produces little powder and is often faintly visible.
Very Hard – Cannot be scratched with a knife blade; knife leaves a metallic streak.

V. STRENGTH OF UNFRACTURED SPECIMEN

Plastic – Capable of being molded by hand.
Friable – Crumbles by rubbing specimen with fingers.
Weak – Crumbles under light hammer blows.
Moderately Strong – Withstands a few heavy hammer blows before fracturing.
Strong – Withstands a few heavy ringing hammer blows and usually yields large fragments.
Very Strong – Resists heavy ringing hammer blows and yields with difficulty only dust and small flying fragments.

VI. WEATHERING; The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

Deep – Moderate to complete decomposition of minerals, extensive disintegration, deep and thorough discoloration, fractures all extensively coated with oxides, carbonates and/or silt and clay.
Moderate – Slight change or partial decomposition of minerals, little disintegration, little to no effect on cementation, moderate to occasionally intense discoloration, fractures moderately coated with oxides, carbonates and/or silt and clay.
Little – No megascopic decomposition of minerals, little to no effect on cementation, slight and intermittent or localized discoloration, fractures coated with few oxides
Fresh – Unaffected by weathering agents, no disintegration or discoloration.



WALL DRAINAGE DETAIL
(Not to scale)

NOTES:

(1) Drain rock should be either: 1) clean, free-draining, and meet the requirements for Class II Permeable material, Section 68, State of California "Caltrans" Standard Specifications, latest edition; or 2) 3/4 or 1-1/2 inch crushed drain rock separated from the adjacent soil/rock by non-woven filter fabric.

Prefabricated synthetic drainage structure, such as Miradrain 6000 or equivalent, may be used in lieu of drainrock along the back of the retaining wall.

(2) Pipe should consist of PVC Schedule 40 or ABS with an SDR of 35 or better, installed perforations down. Pipes for subsurface walls should be sloped at a minimum gradient of 1% to drain to outlets by gravity or sump with automatic pump. The pipe invert should be a minimum of 8 inches below adjacent interior slabs-on-grade. Surface drainage should not be connected to subsurface drain pipes.

(3) The upper 12 inches of the drain should be backfilled with compacted clayey soils to exclude surface water. Retaining walls should be backfilled with materials approved by us and per the recommendations in the report. Backfilling methods should be appropriate to avoid over-stressing the wall structures. Wall bracing should be considered prior to backfilling.

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 3745.0 Date: 8/20 By: SJK	**WALL DRAINAGE DETAIL** 22147 RUOFF ROAD Jenner, California	PLATE **7**