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Geotechnical Investigation

BAUER
associates, inc.
GEOTECHNICAL CONSULTANTS

February 6, 2025
Job No. 4441.0

Bloom Holdings, LLC
Attention: Bob Mueller
731 South Highway 101, Suite 12
Solana Beach, CA 92075

Report
Geotechnical Investigation
Proposed Residence Development

This report presents the results of our geotechnical investigation for the subject project. The project is shown on Sheet A1.1, *Enlarged Partial Site Plan*, dated February 4, 2025, prepared by Cohn + Associates. The site plan is partially reproduced and presented on Plate 1.

We understand that you are currently in escrow to purchase the undeveloped 14.4-acre Adjusted Parcel B. Development plans include constructing a two-story residence, a one-story guest house and a detached garage with standing seam metal roofs. The residence and guest house will have structurally supported wood floors. The detached garage will have a concrete slab-on-grade floor. The foundation loads are expected to be typical for the type of construction indicated. It is also planned to construct a swimming pool with one or two infinity edges and a surrounding pool terrace up to 10 feet above existing grades. Tiered planters are planned on the downhill side of the pool terrace. Access to the garage will be provided by a new driveway. Cuts and fills on the order of 5 to 10 feet high, respectively, are planned. Retaining walls will be needed for the level breaks across the site.

The scope of our investigation, as outlined in our agreement dated November 5, 2024, included reviewing selected geologic and geotechnical references from our files, performing a site reconnaissance, and exploring subsurface conditions at the site. Based upon our work, we have developed conclusions and recommendations concerning:

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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, including seismic design criteria.

5. Concrete slabs-on-grade.

6. Retaining walls.

7. Geotechnical engineering drainage.

8. Supplemental services.

Our scope of work does not include evaluation of any potential hazardous waste contamination or corrosion potential of the soil or groundwater at the site. Further, our scope of services does not include evaluation of areas beyond the proposed improvements discussed.

WORK PERFORMED

We reviewed selected geologic and geotechnical references in our files pertinent to the site. The references reviewed are presented in the *List of References* at the end of this report.

On November 4, 2024, our geologist visited the site to: 1) observe existing surface conditions; 2) confirm our exploration approach; and 3) mark exploration locations. On November 7, 2024, our geologist explored the subsurface conditions at the site to the extent of five test pits. On January 27, 2025, our geologist performed supplemental exploration at the planned guest house and pool areas to the extent of two test pits. The test pits were excavated with a Kubota KX057-5 mini excavator equipped with a 30-inch-wide bucket. The completed test pits ranged in depth from about 3-1/2 to 8 feet. All of the test pits were excavated into bedrock.

The test pits were located by pacing or estimating distances from features indicated on the site plan provided to us. The test pit locations, shown on Plate 1, should be considered accurate only to the degree implied by the method used.

Test pits were backfilled with the on-site excavated materials and tamped with the mini-excavator bucket. The test pits were not backfilled with compacted fill and will settle.

Our geologist logged the conditions exposed and obtained representative disturbed grab samples of the materials encountered for visual identification and possible laboratory testing. Logs of the test pits showing the materials encountered, groundwater seepage conditions, and grab sample depths are presented on Plates 2 through 5. The materials encountered are classified in accordance with the Unified Soil Classification System and the Rock Classification Criteria, presented on Plates 6 and 7.

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.

SITE CONDITIONS

The approximately 14.4-acre Adjusted Parcel B is situated on the northwestern foot slopes of the Yountville Hills. The parcel is located on the westerly-facing flank of a northwesterly trending spur ridge. A southwesterly ravine is within the central portion of the parcel. Much of the parcel is oak-forested with grassy slopes on the north and west borders of the parcel. The proposed development area extends over a westerly, grass-covered slope with scattered oak trees just downhill of the oak forest. Slope gradients within the development area range between 3:1 (horizontal to vertical) and 5:1. Wire fencing roughly parallels the transition area from oak forest to grass cover. A gravel road from Yount Mill Road borders the western parcel boundary and accesses the site. The primary development area is situated just south of the ravine. A man-made swale crosses the mouth of the ravine along the wire fencing and drains into a vertical 12-inch corrugated pipe. The pipe then drains into a buried culvert and follows the inboard side of the gravel road northward towards Yount Mill Road.

The geologic map by Clahan et al. (2005) indicates various volcanic rocks associated with the Sonoma Volcanics underlie Yountville Hills. The volcanic unit underlying the site is described by the authors as andesite lava flows of Stags Leap with undifferentiated rhyolite lava flows and flows breccias, and dacite of Mt. George in nearby areas. Our site reconnaissance and subsurface exploration encountered dacite lava flows.

Our subsurface exploration, summarized on Plates 2 through 5, encountered dacite bedrock ranging between about 3 and 7 feet below the ground surface in all of the test pits. The dacite bedrock is gray to dark gray, moderately hard to hard, weak to strong, and little to moderately weathered. Fracturing is generally close to moderate. The bedrock is considered to be dense and incompressible under the anticipated range of loading. Practical excavation refusal in the bedrock was encountered in several of the test pits with the mini excavator equipment used.

The dacite bedrock is generally covered by weak and porous colluvial soils and variable density fill materials ranging between about 3 and 7 feet deep. These soils typically consist of loose to medium dense sandy gravels and gravelly sands, and medium stiff to stiff gravelly clays. Weak and porous colluvium and old fills are susceptible to consolidation and collapse when saturated and under load. In addition, colluvial soils are subject to downhill creep on terrain sloping steeper than about 6:1. Creep is the gradual, downhill movement of soil and loose rock materials overlying competent bedrock. Creep forces are typically influenced by slope gradient and seasonal moisture changes. Frequent volcanic cobbles and volcanic boulders were present on the surface and within the colluvium and fill soils. The fill depths over much of the proposed development area range between about 1 and 3 feet deep (Test Pits 1, 2, 5, and 6), and fills and colluvium of between about 6 and 7 feet deep (Test Pits 3, 4, and 7) to the north of the planned development. The estimated depths of weak surface soils are shown on the test pit logs. Our visual classification suggests the surface soils have generally low plasticity.

Groundwater seepage was observed within only Test Pit 7 during our subsurface exploration. The seepage was prominent within the colluvial soils between about 5 and 7 feet deep. We anticipate that rainwater infiltrates into the surface soils and flows downslope and on the bedrock surface. Groundwater conditions are expected to vary seasonally and at different locations. Temporarily perched groundwater can be encountered at or near the ground surface or relatively shallow depths, particularly during the winter and spring months.

Landslide mapping indicate similar interpretations by Dwyer et al. (1976) and Clahan et al. (2005). Dwyer et al. (1976) maps soil creep within the ravine area. Clahan et al. (2005) maps a larger area in the ravine location, approximately 600 feet long and 450 feet wide, and encompassing the entire proposed development area. In our test pits, we did not encounter landslide debris. In Test Pit 7 excavated at the mouth of the ravine, we encountered about 7 feet of colluvium overlying bedrock. Our review if Lidar imagery shows a bowl-shaped area further uphill that narrows downhill at the fence line.

Interactive geologic maps of the area prepared by the California Geological Survey (CGS, 2024 revision) do not show active faults crossing the site, and the site is not shown to be within current Alquist-Priolo Earthquake Fault Zone boundaries of required investigation for seismically active faults (experiencing surface rupture within about the last 11,000 years). The nearest fault within earthquake zones of required investigation is the West Napa, located about 6 miles to the south-southeast. Older fault traces related to the West Napa, not currently considered 'Holoceneactive', are located 1 mile and more to the southeast. Our authorized scope of work did not include subsurface investigation to evaluate the presence or active faults crossing the site.

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 concerns are: 1) the presence of weak colluvial soils and variable density fill materials encountered in our test pits; 2) the creep potential of the surface soils (colluvium and variable density fills) overlying the bedrock; 3) anticipated hard excavation into the bedrock; and 4) the anticipated high volume of groundwater seepage from uphill sources. Preliminary development plans showed structures within the mouth of the ravine; however, based on our discussions with you regarding the soil conditions, the development was moved further south and away from the ravine.

Weak and porous colluvium and variable density fill soils are subject to uneven supporting characteristics and differential movements and are prone to consolidation and/or collapse when saturated and under loads of new fills and/or structural elements. Saturation will occur when the natural evaporation of soil moisture is inhibited by new fill and structural elements. In addition, the weak surface materials are prone to downhill creep. Therefore, we judge that the existing surface soils overlying the bedrock will not be suitable for support of new foundations, slabs, and other hardscape surfaces in their present condition.

However, suitable foundation support can be obtained from foundations excavated though the weak materials and into firm bedrock, or into engineered fill of relatively even thickness. Foundations excavated though weak soils and into bedrock will need to be designed to resist creep forces. Drilled pier foundations may not be an appropriate foundation type across the development due to the hardness and strength of the bedrock; however, drilled piers may be appropriate for support of the infinity edge swimming pool. Therefore, drilled pier recommendations are included in this report.

Excavations will encounter hard, resistant bedrock. Deep excavations, such as at retaining walls supporting cuts or other deep excavations to achieve level areas, will require heaving ripping and/or jack hammering equipment. In addition, the excavated materials may include a large portion of oversize rocks which will not be re-useable as engineered fill. Contractors bidding this job should be provided with this report to become familiar with the site conditions as they pertain to their operations and the appropriate equipment needed to perform their tasks. If more detailed information regarding excavatability of the rock is required or desired, additional excavations using the type and size of equipment planned for construction should be performed.

We did not encounter moderately or highly expansive materials in our test pits. If expansive materials are encountered during construction, we must be contacted to provide supplemental recommendations as appropriate.

It has been our experience that swimming pools, particularly infinity edge design, are not tolerant to differential movements resulting from expansion and contraction of the underlying materials, or from weak and porous soils. Small differential movements can cause visible distortions of infinity edge pools and pool decks, including uneven-appearing water lines within the pool, recycled water flowing unevenly over the infinity edge, etc. Therefore, swimming pool support should be obtained entirely into firm bedrock. Footings or drilled piers extending into bedrock will be needed.

On sloping terrain to receive fill, keyways and benches with subdrainage improvements with cleanouts will be required. Concrete slabs will be prone to differential settlement in areas underlain by uneven fill thicknesses. Where slabs extend across areas of uneven fill thicknesses, typically on the order of 3 feet or more, there will be potential for differential settlement. To mitigate the potential for differential settlement, it may be necessary to over-excavate portions of the bedrock to provide a relatively uniform fill thickness and/or provide additional compaction effort (typically 93 percent relative compaction). We recommend that we be contacted to provide consultation when preliminary plans are available in order to assist the designers in determining suitable grading and foundation support approaches, prior to final design.

Less critical use slabs-on-grade, such as walkways and small equipment slabs, may be constructed on properly prepared subgrade (i.e., vegetation grubbed, removal of rubble, debris, and obstructions, level pad) provided: 1) the slabs are separated from foundations with felt paper, mastic, or other positive and low friction separation; 2) slabs are designed and reinforced to minimize cracking (i.e., reinforced and provided with control joints); and 3) some soil-related cracking and differential movements are considered acceptable. Improved performance of less critical use exterior slabs could be attained by removal of all, or at least some, of the weak soils and replacement as engineered fill. The depth of overexcavation is dependent upon the level of performance desired by the owner.

Groundwater seepage was encountered only in Test Pit 7, and did not develop in the rest of our test pits during excavation. However, groundwater conditions are expected to vary in depth and extent across the site, especially at the mouth of the ravine. Perched groundwater conditions will vary seasonally and by location across the site, particularly after periods of prolonged rainfall or during the winter and spring months. Excavations performed in the summer or autumn months will typically result in a lower risk of encountering groundwater or wet soil conditions than in winter or spring months and may require less mitigation during site work should groundwater seepage or saturated soils be encountered.

Control of surface run off will significantly enhance the stability of the site. The site must be graded to provide positive drainage away from the building foundations, slab edges, and finished cut and fill slopes. Roof gutter-downspouts must be collected into non-perforated pipes and discharged into the site storm drainage system or onto erosion resistant areas well away from the structures and slopes. Roof downspouts and surface drains must be maintained entirely separate from subsurface drainage. Foundation drains should be considered adjacent foundations extending through weak soils and bearing into bedrock.

Crawl space areas should be sloped to drain and provided with outlets through foundations. If living area slabs-on-grade are used, underslab drains should be provided to reduce the risk of water build up in the slab rock. In non-living area slabs-on-grade (e.g. garage), outlets should be provided in the slab rock to reduce the risk of water build up in the slab rock. If desired by others, underslab drains could also be used for garage and other non-critical slab areas. All collected water must be discharged onto erosion resistant areas, away from the development.

The published maps indicate landsliding at the site. As discussed previously, Dwyer et al. (1976) maps the ravine area as soil creep. Clahan et al. (2005) maps an approximately 600 feet long and 450 feet wide landslide that encompasses the entire proposed development area. Colluvial soils were encountered in Test Pit 7 excavated in the ravine area, and our review of Lidar imagery indicates a bowl-shaped area further uphill and at the head of the ravine that narrows downhill at the fence line. We anticipate that past landsliding and erosion has occurred further uphill and the materials transported downhill as intermittent debris flows.

We observed creep-prone soils uphill of the planned development area. We typically recommend that the hazard of materials flowing to the structure be mitigated by the construction of a catchment wall or a minimum 8 feet wide level buffer zone between structures and upslope areas. Catchment walls, where used, can either be constructed adjacent the structure or incorporated into the structure and should be at least 2 feet above planned slopes. As requested, we can provide additional comments regarding catchment when the plans have been more fully developed.

The results of our literature review did not reveal active faults passing through the site. Since future fault rupture is generally considered more likely to follow the trace of the most recent fault rupture, we estimate the risk of future surface rupture during earthquakes to be low.

Like the entire Napa County and Northern California areas, severe ground shaking during earthquakes generated by active faults in the region represents a significant geologic hazard to developments in the region. The intensity of ground shaking will be dependent on several factors such as distance from the site to the earthquake focus, magnitude of the earthquake, depth of the earthquake, duration of ground shaking, and response of the underlying soil and rock. It will be necessary to design and construct the structure in accordance with current standards for earthquake-resistant construction.

RECOMMENDATIONS

Site Preparation and Grading

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

Areas to be graded should be cleared of rubble, debris, old fills (where encountered) and loose rocks. Material generated by the clearing operations should be removed from the site. Wells, cesspools, and other voids encountered or generated during clearing should be either backfilled with granular material or compacted soil or capped with concrete as determined by us and in accordance Napa County requirements.

Areas to be graded should be stripped of the upper soils containing root growth and organic matter. 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 beyond structures, walks and paved areas.

Test pits were backfilled with the on-site excavated materials and tamped with the mini-excavator bucket. Test pits in development areas must be properly filled during construction to reduce the risk of settlement.

Following clearing and stripping, planned excavations should be performed. For the purpose of definition, "select fill areas" referred to in this report are buildings with shallow foundations or critical-use concrete slab areas. Select fill areas also include the zones extending for a distance of at least 5 feet beyond outside edges of slabs and perimeter footings or other footings extending from buildings. Within the select fill areas, existing weak surface and old fill soils should be removed for their full depth. The depth and extent of overexcavation should be approved in the field by us. Excavation of weak soil materials and placement of select fill will not be required where: 1) deepened foundations into bedrock are used for support of the structure; 2) no living area concrete slabs are planned; and 3) structurally supported slabs spanning between foundations are used.

Areas to receive fill on terrain sloping steeper than about 5:1 should be prepared by excavating level keyways and benches extending into firm bedrock. Subsurface drains with cleanouts should be installed at the rears of keyways. The depths of keyways and locations of subsurface drainage facilities should be determined and approved by us in the field during grading. A typical fill and subdrain detail is presented on Plate 8.

Within the select fill areas, the exposed bottoms should be scarified to a minimum depth of 6 inches (where possible), moisture conditioned to at least 2 percent above optimum moisture content and compacted to at least 90 percent relative compaction. Scarification and recompaction may be waived, as determined by us, where firm, undisturbed bedrock is encountered. 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 (2021). Optimum moisture content is the water content (percentage by dry weight) corresponding to the maximum dry density.

If grading is performed during the winter or spring seasons, we anticipate that higher groundwater conditions may be encountered. Severe groundwater conditions may result in the need for dewatering, placement of stabilization fabrics, and/or placement of ballast rock to achieve stable excavation bottoms.

The on-site soils, excluding large rocks generated by site excavations, should be suitable for reuse as select fill. Depending on the volume of large rocks generated, a portable crusher may be considered to make select fill. 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 4318-17e1 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, and compacted to at least 90 percent relative compaction. Where fill thickness differential under foundation, slab or hardscape surfaces is 3 feet or greater, the fill should be compacted to at least 93 percent relative compaction. In vehicle traffic areas, the upper 6 inches of subgrade and aggregate base rock materials should be compacted to at least 95 percent relative compaction. It may be necessary to excavate portions of bedrock in order to provide fill areas with a relatively even thickness. All surfaces should be finished to present a smooth, unyielding subgrade.

Permanent cut slopes exposing competent bedrock should be constructed no steeper than 2:1. If requested, we can provide supplemental consultation for cut slopes that encounter bedrock where slopes steeper than 2:1 are desired. Permanent fill slopes should be constructed no steeper than 2:1. Fill and cut slopes should be planted with erosion-resistant vegetation or protected from erosion by other measures upon completion of grading. Ground cover should be maintained on the slopes.

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. The performance of temporary cut slopes is the responsibility of the contractor/owner.

Foundations

Suitable foundation support for structures can be obtained from foundations excavated though the weak materials entirely into firm bedrock, or entirely into engineered fill of relatively even thickness. Drilled piers may be an appropriate foundation type for support of the infinity edge swimming pool. Combinations of foundation types in fill and bedrock should be avoided, but combinations of piers and footings in bedrock would be suitable.

Spread Foundations

Spread footings bearing entirely into bedrock or engineered fill may be used for foundation support. Footings should be at least 12 inches wide and penetrate through the weak surface soils to bear at least 12 inches into undisturbed bedrock. The footings should be stepped as necessary to produce level tops and bottoms. Footings should be deepened as necessary to provide at least 5 horizontal feet of confinement between the footing bottoms and the face of the nearest slope where the footing bears into bedrock. Dowelling into bedrock may be considered during foundation excavation. We should observe the quality and fracturing of the bedrock to determine if dowelling is acceptable and to provide supplemental recommendations, as needed. Where a graded pad of even thickness is constructed, footings should be at least 12 inches wide and bear at least 12 inches into firm engineered fill.

Spread footings supported in bedrock may be designed using allowable bearing pressures of 3,000 and 4,000 pounds per square foot (psf) for dead plus long-term live loads and total design loads, respectively. Footings in engineered fill should be designed using allowable bearing pressures of 2,000 and 3,000 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, undisturbed bedrock may impose a passive pressure of 400 pounds per cubic foot (pcf) triangular distribution and a friction factor of 0.4 times the net vertical dead load. Footings in engineered fill may impose a passive pressure of 350 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.

Drilled Piers (Swimming Pools)

If piers are used for support of the pool, piers would need to extend through the engineered terrace fill to bear into firm bedrock. Piers should be at least 16 inches in diameter and extend a minimum depth of 8 feet deep with at least 4 feet into firm bedrock. The portion of the pier extending into engineered fill and bedrock can impose 750 psf in skin friction. Pullout capacities of the piers can be one-half of the downward capacity. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously. Piers should not be located closer than 3 pier diameters, center to center.

The portion of the piers extending into engineered terrace fill and firm bedrock may impose a passive pressure of 350 pcf acting on two pier diameters. Passive pressure should be neglected within the weak soils unless foundations are confined by other construction. Passive pressure should be limited to a maximum of 3,000 psf.

Piers should be interconnected with grade beams designed to support the design structural loads per current code requirements. Piers should be reinforced for their full length with steel reinforcing that extends into the grade beams.

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 the wet concrete from settling. Concrete should be placed in pier excavations promptly to avoid soil desiccation. Excess concrete must be removed to planned dimensions from the tops of piers and bottoms of grade beams to reduce uplift pressures.

Caving materials may be encountered where wet, weak soils are present. If caving soils are encountered, it may be necessary to case the holes. If groundwater is encountered during drilling, it will be necessary to place the concrete using the tremie method or dewater the holes prior to concrete placement. The drilling subcontractor should review this report and visit the site to draw their own conclusions regarding drilling characteristics, suitable drill rigs and the need for casing and dewatering prior to bidding.

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.

Swimming Pools

Swimming pools are lightly loaded structures that are generally not tolerant to differential movements resulting from: 1) expansion and contraction of the underlying materials; 2) settlement of weak and porous soils; and 3) construction defects (e.g., broken water pipes). Small differential movements can cause visible distortions of swimming pools and pool decks, uneven-appearing water lines within the pool, concrete cracking, recycled water flowing unevenly over infinity edges, etc. These types of movements have resulted in owner dissatisfaction and significant repair costs in the past. Therefore, uniform support for the swimming pool should be obtained from firm bedrock as determined by us during excavation. Where the pool bottom does not bear into firm bedrock, spread footings or drilled piers should extend through the engineered terrace fill to bear into the underlying bedrock and be designed in accordance with the previous section recommendations. Further, where bedrock is not exposed on the pool bottom, the pool bottom should be designed to span between foundations that extend into bedrock. Even with this design, some movements must be anticipated. If it is desired to further reduce the potential for differential movements, the structural engineer should be contacted to incorporate additional factors of safety. Additionally, we understand from the pool contractor that precautions can be incorporated into the installation to allow for easier post construction mitigation, if needed.

Pool walls should be designed for lateral earth pressures as discussed in the *Retaining Walls* section above. A drainage relief valve should be provided in the pool bottom to relieve hydrostatic pressures.

Seismic Design Criteria

Using site latitude and longitude coordinates of 38.42301°N and -122.38484°W, respectively, the following criteria is based on 2022 CBC guidelines, ASCE 7-16, and the USGS Earthquake Ground Motion Parameters:

Spectral Response Type & Description	Value (g)
S_S (0.2 second period)	1.858
S_1 (1.0 second period)	0.674
S_{MS} (0.2 second period)	1.672
S_{M1} (1.0 second period)	0.539
S_{DS} (0.2 second period)	1.115
S_{D1} (1.0 second period)	0.359
Peak Ground Acceleration (PGA)	0.776
Seismic Design Category	D

Title 24, Part 2, Section 1613.2.2, of the 2022 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 shallow test holes, we estimate that a Site Classification "B" 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, Catchment, and Pool Walls

Foundation support for retaining, catchment, and pool walls can be obtained from foundations bearing into firm bedrock designed in accordance with the recommendations presented above.

Retaining walls free to rotate (yield more than 0.1 percent of the wall height at the top of the backfill) and with backfill sloping up to 3:1 should be designed to resist an active lateral earth pressure (triangular distribution) of 45 pcf for drained conditions. Where the backfill slopes up between 3:1 and 2:1, the pressures indicated above should be increased to 55 pcf for drained conditions. We should be contacted if the backfill slopes up steeper than 2:1. Rigid walls with backfill sloping less than 3:1 which cannot yield should be designed for an "at-rest" lateral earth pressure of 60 pcf for drained conditions. A minimum factor of safety of 1.5 against overturning and sliding should be used in the design of retaining walls. If retaining walls will have undrained conditions, we should be consulted to provide increased active and at-rest lateral pressures.

Where applicable, seismic wall stability for cantilever retaining walls with level and sloping backfill may be evaluated based on an earth equivalent fluid pressure (triangular distribution) of 10 pcf and 29 pcf, respectively. Seismic wall stability for rigid retaining walls may be evaluated based on an earth equivalent fluid pressure (triangular distribution) of 31 pcf. This pressure is in addition to the active earth equivalent fluid pressures presented in this report. Incremental seismic lateral earth pressures should not be added to at-rest lateral earth pressures. The resultant force should be considered to act at a height of 0.33H on the wall. The factor of safety against instability under seismic loading should be at least 1.1.

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. Similarly, if stepped retaining walls are planned, we should be contacted to provide specific lateral surcharge pressures for the lower walls based on the final wall configuration. Walls subjected to vehicular traffic should be designed for a surcharge pressure equal to 2 feet of additional backfill.

Retaining walls should be provided with permanent backdrains to prevent the build-up of hydrostatic pressure. If walls are to have undrained conditions, we should be consulted to provide increased active and at-rest lateral pressures. The drains and backfill should be constructed as shown on Plate 9. The top of the perforated drainage pipe should be located at least 8 inches below any interior slabs and other adjacent areas to reduce the risk of seepage through walls and into interior areas.

Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed as specified by others. Backfill materials should be compacted in a manner to prevent over-stressing the wall structures. 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.

Expansive materials should not be used as retaining wall backfill. Expansive materials may only be used as backfill outside of the zone defined by a 1:1 projection from the top of the foundation. Non-expansive on-site soils may generally be used as backfill except as noted above. Backfill soils must be compacted in accordance with our previous recommendations. 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 or slabs be avoided within retaining wall backfills to avoid the potential for differential movements. Mitigation may include designing foundations or slabs to span from retaining walls to beyond the backfill area. We should be contacted to provide supplemental consultation if foundations or slabs will extend across retaining wall backfills.

Concrete Slabs

Critical use slabs-on-grade (e.g., living areas, pool decks, and other settlement-sensitive slabs) should be underlain entirely by firm bedrock or by engineered fill as previously discussed. Slab-on-grade subgrades should be a smooth, uniform and non-yielding surface. Subgrade should be maintained at a uniform moisture, at least 2 percent above optimum moisture content, until the concrete slabs are placed. During foundation installation and utility trench excavation and backfilling, previously compacted subgrade soils may become disturbed. Where this is the case, these soils should be uniformly moisture conditioned to above optimum moisture content and rerolled to provide a smooth, unyielding surface compacted to at least 90 percent relative compaction (93 percent where fill thickness differential exceeds 3 feet).

Less critical use slabs-on-grade (e.g., walkways, small equipment slabs, etc.) may be constructed on properly prepared subgrade provided: 1) the slabs are separated from foundations with felt paper, mastic, or other positive and low friction separation; 2) slabs are designed and reinforced to minimize cracking (i.e., reinforced and provided with control joints); and 3) some soil-related cracking and differential movements are considered acceptable. Properly prepared subgrade typically includes grubbing vegetation, removing rubble, debris and/or obstructions, and constructing a level pad. We should be contacted to provide supplemental recommendations if improved performance of less critical slabs is desired.

Slabs should be underlain with a capillary moisture break and cushion layer consisting of at least 4 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-on-grade. Where moisture migration through slabs is detrimental, waterproofing methods and specifications should be determined by others for incorporation into the project plans.

Slab thickness should be recommended by the structural engineer to support the anticipated loads and to reduce cracking. However, 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 (as opposed to welded wire mesh), placed on blocks as directed by the structural engineer. We have previously observed that wire mesh is often not properly located in the slabs. Control and expansion joints should be provided, as appropriate, to mitigate the effects of differential settlement.

Geotechnical Engineering Drainage

Ponding water will be detrimental to foundations and structural elements. Therefore, the site should be graded to provide positive drainage away from foundations, swimming pool edges and their adjacent slabs, other exterior slab edges, and underfloor areas if constructed. Roofs should be provided with gutters, and the downspouts connected to the site storm drain system discharging in erosion resistant areas well away from the structures and slopes. Roof downspouts and surface drains must be maintained entirely separate from subsurface drainage. Collected water must be discharged into non-perforated pipes and discharged into the site storm drainage, concrete slabs-on-grade, pavements, or erosion resistant areas.

Crawl space areas should be sloped to drain and provided with outlets to allow controlled drainage through foundations. It will be important to have proper crawl space ventilation, as designed by the project structural engineer.

In living area slab-on-grade areas, underslab drains should be provided to reduce the risk of water build up in the slab rock and moisture migration through the slab. The subdrain trenches should be 12 inches wide, 12 inches deep and cross the slab area, as directed by us. The slab rock should be connected to the subdrain rock. In non-living area slabs-on-grade (e.g. garage), outlets should be provided in the slab rock to reduce the risk of water build up in the slab rock. If desired by others, underslab drains could also be used for non-critical slab areas.

Keyway excavations for fills placed on slopes should be equipped with a keyway subdrain. A typical fill and subdrain detail is presented on Plate 8.

Retaining wall backdrains should be constructed to reduce hydrostatic pressures against retaining walls. The backdrains should be at least 12 inches wide and extend up to the height of the drained portion of the walls. Plate 9 presents criteria for retaining wall backdrains. Subdrains should consist of 4-inch diameter, perforated pipe, installed perforations down, placed at the bottom of the drain and sloped to drain to outlets by gravity. The subdrain 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 alternatives to standard drain rock and fabric, Class II permeable material complying with Section 68, “Caltrans” may be used without fabric or a prefabricated synthetic drainage structure such as Miradrain 6000 (or equivalent) may be used. The upper 12 inches of the drain should be backfilled with compacted, non-expansive clayey soil to exclude surface water. If groundwater seepage is encountered during grading, additional subdrains should be installed as recommended by us.

Supplemental Services

We should be contacted during final design to provide additional comments and recommendations, as needed. 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 materials; 2) fill placement and compaction; 3) preparation and compaction of subgrade; 4) excavations of foundations; and 5) installation of subdrains. 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 be contacted to perform a final observation and 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.

MAINTENANCE

Periodic land maintenance will be required. Surface and subsurface drains should be checked frequently and cleaned and maintained as necessary. Sloughing, landsliding or erosion that occurs should be repaired promptly before it can enlarge. A dense growth of deep-rooted ground cover should be maintained on all exposed slopes.

LIMITATIONS

We judge that construction in accordance with these recommendations will be generally stable, and that the risk of future instability is within the range generally associated with construction in the local area. Subsurface conditions are complex and may differ from those indicated by surface features and those encountered at the test hole locations. Additional exploration could reveal conditions not evident at this time. Therefore, we are unable to guarantee the stability of any hillside construction.

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.

Gregory D. Sarganis
Professional Geologist

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GREGORY D. SARGANIS
No. 7422
2-6-2025
GEOLOGIST
STATE OF CALIFORNIA

Arthur H. Graff
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
ARTHUR H. GRAFF
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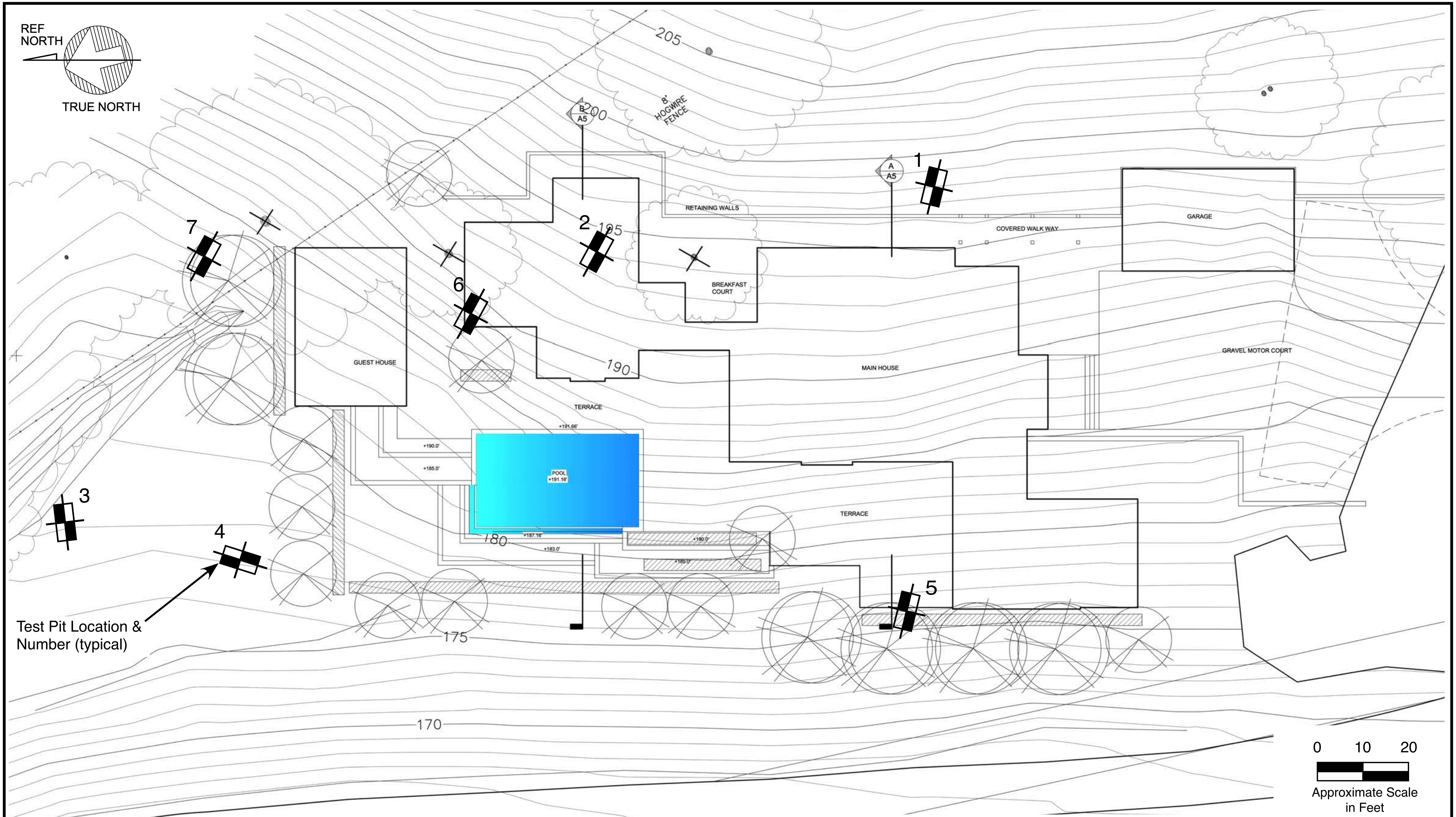
GDS/AHG (gi/yount mill (1201))
Attachments: Plates 1 through 9
Email only: bmueller@rmcigroup.com, brian@quallseng.com, gary@cohn-arch.com, andrew@quallseng.com



Reference: Adapted from Sheet A1.1, *Enlarged Partial Site Plan*, dated February 4, 2025, prepared by Cohn + Associates.
Note: The locations of all features are approximate and may vary. Scale as shown

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 4441.0 Date: 12/2024 By: GDS	**TEST HOLE LOCATION PLAN**	PLATE **1**

Test Pit Orientation: N77°W
Log Northeast Pit Sidewall

LOG OF TEST PIT 1

Equipment: Kubota KX057-5, 30" bucket
Date: November 7, 2024
Elevation: 198'*

Depth (ft) | Sample | Graphic Log | Weak Soils

1
2
3
4
5
6
7
8
9

DARK GRAY BROWN CLAYEY SANDY GRAVEL (GP)
loose, damp, porous, with fine roots, contains baserock (Variable Density Fill)

BROWN CLAYEY SANDY GRAVEL (GP)
medium dense, damp, porous, with volcanic cobbles and fine roots (Colluvium)

DARK GRAY DACITE
close to moderate fractures, hard, strong, little weathered
(Bedrock)

Practical excavation refusal at 4 feet
No groundwater seepage observed

Test Pit Orientation : N70°W
Log Northeast Pit Sidewall

LOG OF TEST PIT 2

Equipment: Kubota KX057-5, 30" bucket
Date: November 7, 2024
Elevation: 194'*

Depth (ft) | Sample | Graphic Log | Weak Soils

1
2
3
4
5
6
7
8
9

DARK GRAY BROWN CLAYEY SANDY GRAVEL (GP)
loose to medium dense, damp, porous, with volcanic cobbles, contains brick fragments and drain rock
(Variable Density Fill)

DARK GRAY DACITE
close to moderate fractures, hard, strong, little weathered
(Bedrock)

Practical excavation refusal at 3-1/2 feet
No groundwater seepage observed

* Elevations interpolated from Sheet A1.1, *Enlarged Partial Site Plan*, dated February 4, 2025, prepared by Cohn + Associates.

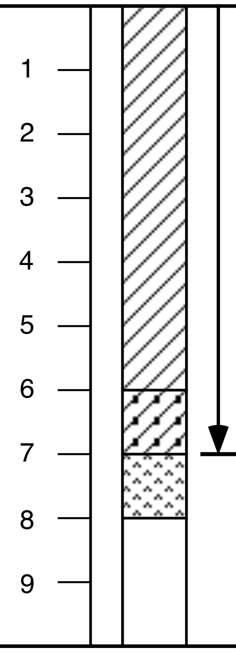
BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 4441.0 Date: 12/2024 By: GDS	**LOGS OF TEST PITS 1 & 2**	PLATE **2**

Test Pit Orientation: N83°E
Log Northwest Pit Sidewall

LOG OF TEST PIT 3

Equipment: Kubota KX057-5, 30" bucket
Date: November 7, 2024
Elevation: 178-1/2'*

Depth (ft) | Sample | Graphic Log | Weak Soils



BROWN GRAVELLY CLAY (CL)
medium stiff, damp to 2', porous with volcanic cobbles and boulders up to 2', contains plastic mesh, plastic pipe fragments, and straw
(Variable Density Fill)
variably medium stiff and stiff, moist from 2 to 6'

DARK BROWN CLAYEY GRAVEL (GC)
medium dense, moist, slightly porous (Colluvium)

DARK GRAY DACITE
close to moderate fractures, hard, strong, little weathered (Bedrock)

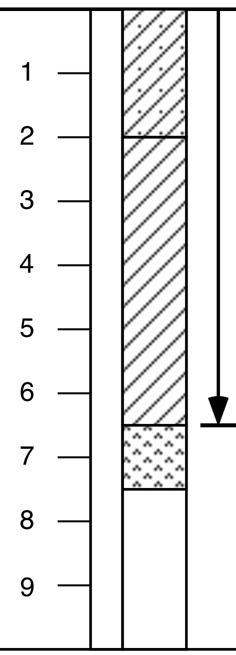
Practical excavation refusal at 8 feet
No groundwater seepage observed

Test Pit Orientation : N18°E
Log Northwest Pit Sidewall

LOG OF TEST PIT 4

Equipment: Kubota KX057-5, 30" bucket
Date: November 7, 2024
Elevation: 177'*

Depth (ft) | Sample | Graphic Log | Weak Soils



GRAY BROWN GRAVELLY CLAYEY SAND (SC)
loose to medium dense, damp, porous, contains plastic fragments
(Variable Density Fill)

BROWN GRAVELLY CLAY (CL)
stiff, moist, intermittently porous, with occasional volcanic cobbles and fine roots throughout, contains plastic pipe fragments
(Variable Density Fill)

DARK GRAY DACITE
close to moderate fractures, hard, strong, little weathered (Bedrock)

Practical excavation refusal at 7-1/2 feet
No groundwater seepage observed

* Elevations interpolated from Sheet A1.1, *Enlarged Partial Site Plan*, dated February 4, 2025, prepared by Cohn + Associates.

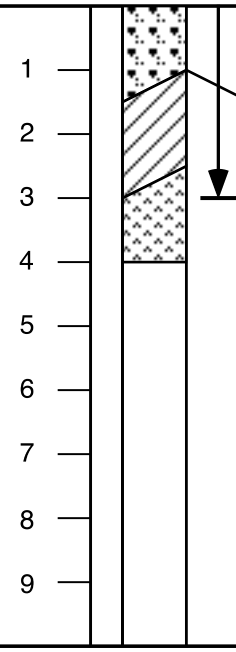
BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 4441.0 Date: 12/2024 By: GDS	**LOGS OF TEST PITS 3 & 4**	PLATE **3**

Test Pit Orientation: N80°W
Log Northeast Pit Sidewall

LOG OF TEST PIT 5

Equipment: Kubota KX057-5, 30" bucket
Date: November 7, 2024
Elevation: 176'*

Depth (ft) | Sample | Graphic Log | Weak Soils



DARK GRAY BROWN CLAYEY SANDY GRAVEL (GP)
loose, damp, porous, with fine roots
(Variable Density Fill)

DARK BROWN GRAVELLY CLAY (CL)
medium stiff, moist, porous, with volcanic cobbles and fine roots (Colluvium)

DARK GRAY DACITE
close fractures, moderately hard and hard, weak and strong, moderately weathered, appears to be locally ripped
(Bedrock)

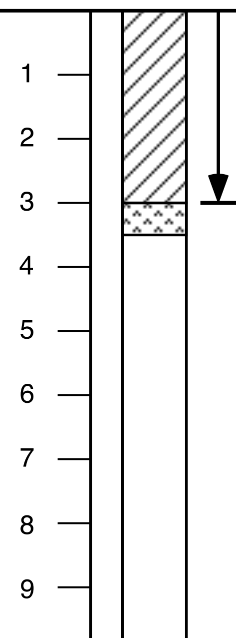
Practical excavation refusal at 4 feet
No groundwater seepage observed

Test Pit Orientation: N51°W
Log Northeast Pit Sidewall

LOG OF TEST PIT 6

Equipment: Kubota KX057-5, 30" bucket
Date: January 27, 2025
Elevation: 189'*

Depth (ft) | Sample | Graphic Log | Weak Soils



DARK BROWN GRAVELLY SANDY CLAY (CL)
medium stiff, moist to wet, porous, with fine roots
(Variable Density Fill)

GRAY DACITE
close fractures, moderately hard, moderately strong, moderately weathered
(Bedrock)

Practical excavation refusal at 3-1/2 feet
No groundwater seepage observed

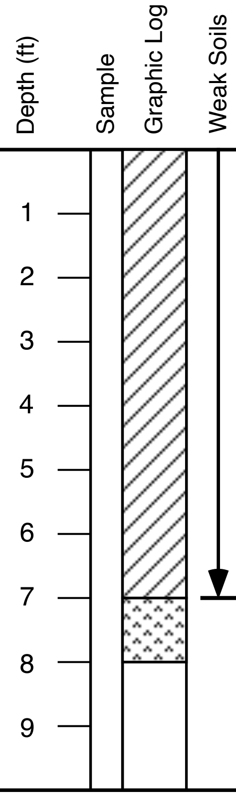
* Elevations interpolated from Sheet A1.1, *Enlarged Partial Site Plan*, dated February 4, 2025, prepared by Cohn + Associates.

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 4441.0 Date: 1/2025 By: GDS	**LOGS OF TEST PITS 5 & 6**	PLATE **4**

Test Pit Orientation: N57°W
Log Northeast Pit Sidewall

LOG OF TEST PIT 7

Equipment: Kubota KX057-5, 30" bucket
Date: January 27, 2025
Elevation: 183'*



BROWN GRAVELLY CLAY (CL)
medium stiff to stiff, wet, porous, with abundant volcanic cobbles and boulders up to 2-1/2 feet in size (Colluvium)

DARK GRAY DACITE
close fractures, hard, strong, moderately weathered (Bedrock)

Bottom of test pit at 8 feet
Heavy groundwater seepage observed from 5 to 7 feet

* Elevations interpolated from Sheet A1.1, *Enlarged Partial Site Plan*, dated February 4, 2025, prepared by Cohn + Associates.

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 4441.0 Date: 1/2025 By: GDS	**LOG OF TEST PIT 7**	PLATE **5**

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 CLAYS 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 %)
-No. 200 = % Passing
-No. 4 = % Passing
EI = Expansion Index

	Shear Strength, psf	Confining Pressure, psf	
Tx UU	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

California Modified Sampler (2.4 in. id)

No Sample Recovery

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

G Grab Sample

BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 4441.0 Date: 12/2024 By: GDS	**SOIL CLASSIFICATION CHART & KEY TO TEST DATA**	PLATE **6**

I. INDURATION OF SEDIMENTARY ROCKS; usually determined from unweathered samples. Largely dependent on cementation and compression.

N = Non-indurated – has not undergone any cementation
P = Poorly indurated – break apart easily by hand
M = Moderately indurated – easily broken with a hammer
W = Well indurated – difficult to break with a hammer

II. BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness in Feet	in Inches	Stratification
Massive	greater than 4.0	> 48	very thick bedded
Blocky	2.0 to 4.0	24 to 48	thick bedded
Slabby	0.2 to 2.0	3/16 to 24	thin bedded
Flaggy	0.05 to 0.2	1/16 to 3/16	very thin bedded
Shaly or Platy	0.01 to 0.05	1/64 to 3/16	laminated
Papery	less than 0.01	< 1/64	thinly laminated

III. FRACTURING

Intensity	Size of Pieces (ft)	(in)
Crushed	less than 0.05	< 1/16
Intensely Fractured	0.05 to 0.1	1/16 to 1/8
Closely Fractured	0.1 to 0.5	1/8 to 6
Moderately Fractured	0.5 to 1.0	6 to 12
Occasionally Fractured	1.0 to 4.0	12 to 48
Very Little Fractured	greater than 4.0	> 48

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

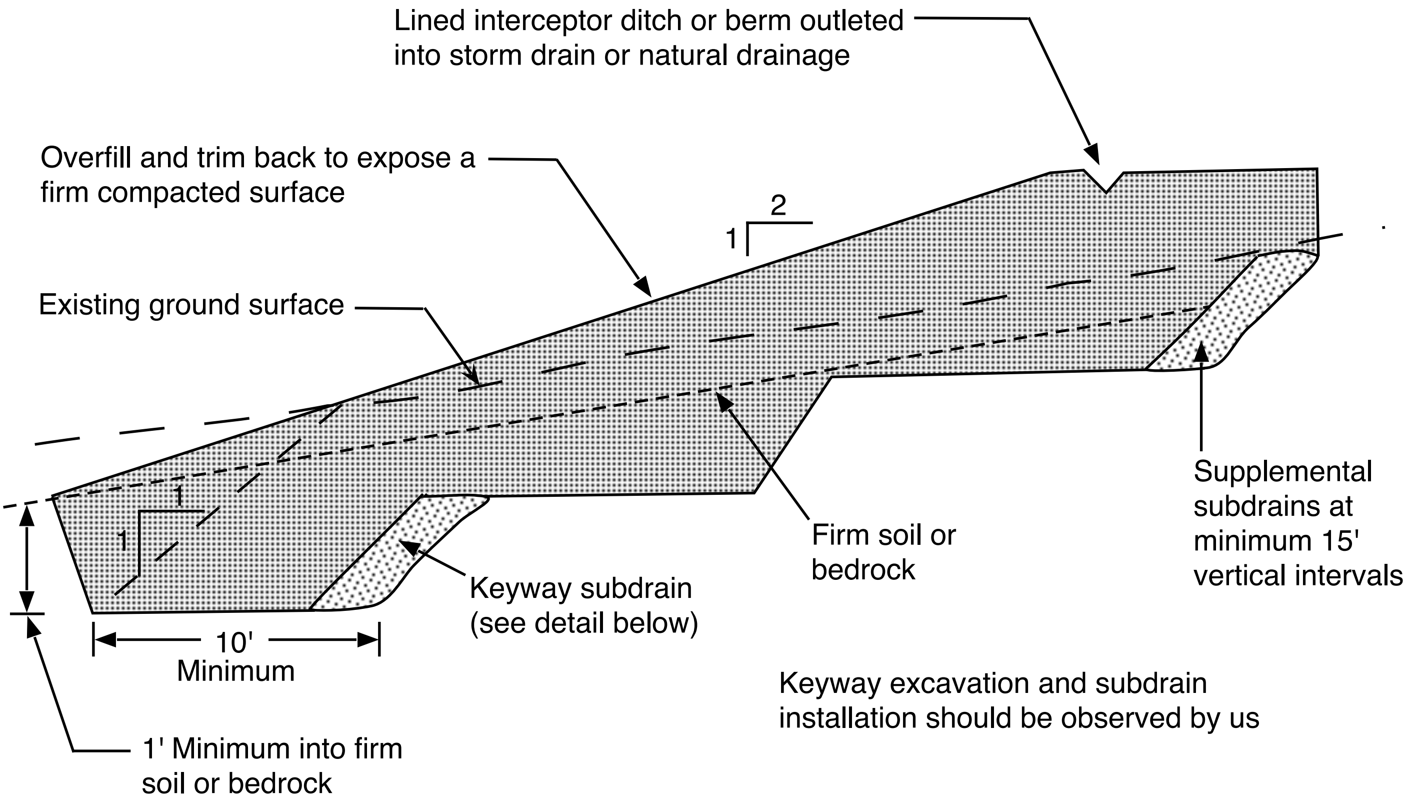
Plastic – Capable of being molded by hand.
Friable – Crumbles by rubbing specimen with fingers.
Weak – Crumbles under light hammer blows.
Moderately Strong – Withstands a few heavy hammer blows before fracturing.
Strong – Withstands a few heavy ringing hammer blows and usually yields large fragments.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

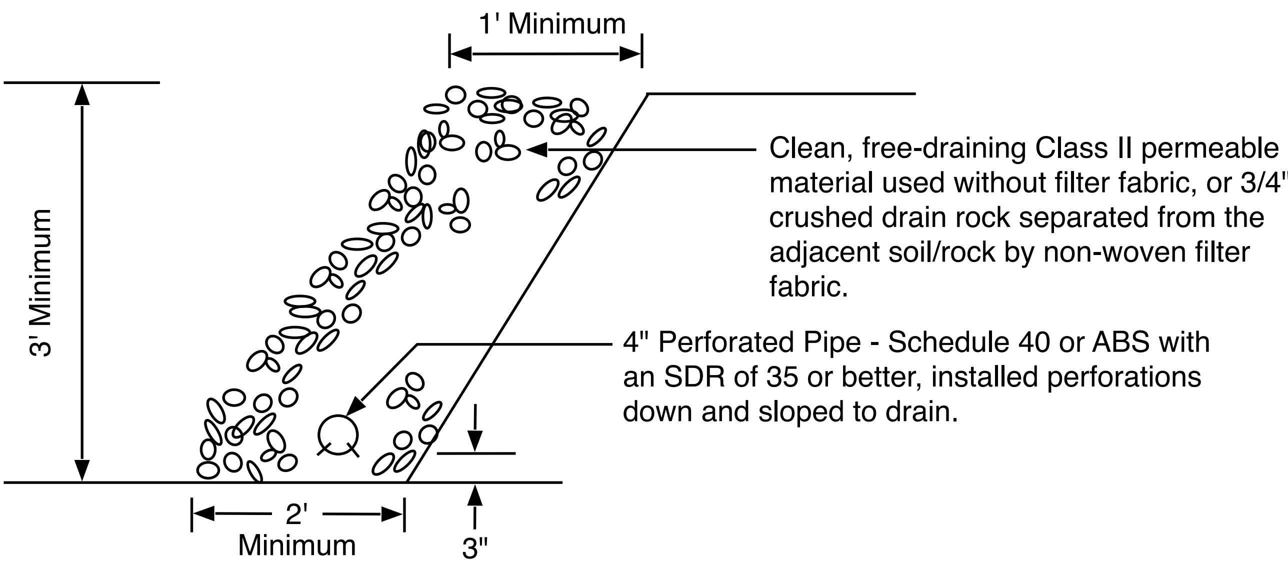
TYPICAL FILL SECTION - KEYWAY CONSTRUCTION

(Not to Scale)

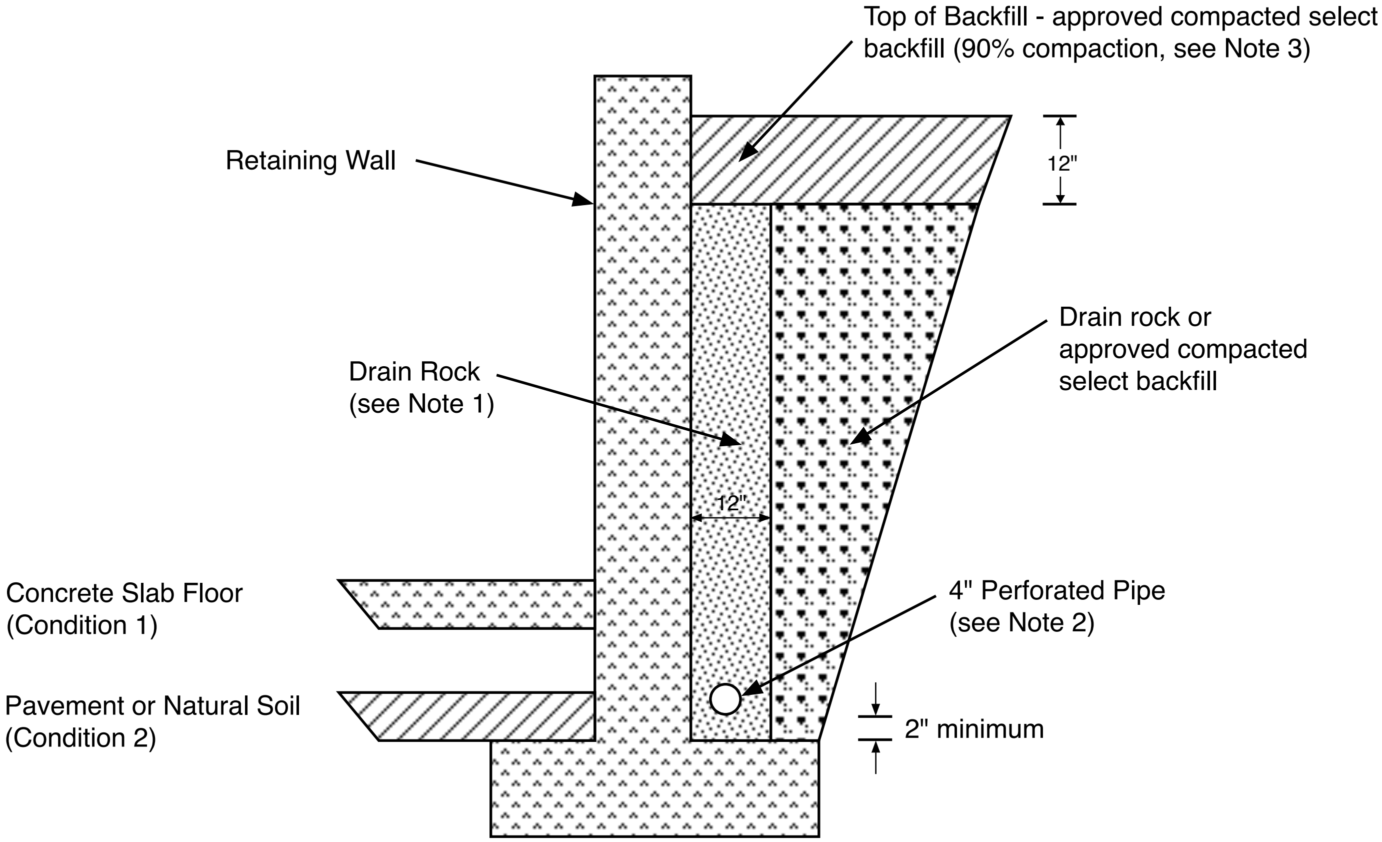


SUBDRAIN DETAIL

(Not to Scale)



BAUER ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	Job No: 4441.0 Date: 12/2024 By: GDS	**TYPICAL FILL SECTION AND SUBDRAIN DETAIL**	PLATE **8**



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: 4441.0 Date: 12/2024 By: GDS	**WALL DRAINAGE DETAIL**	PLATE **9**

LIST OF REFERENCES

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

American Society of Civil Engineers, 2021, ASCE 7 Hazard Tool, https://asce7hazardtool.online

California Building Standards Commission, 2022, California Building Standards Code, California Code of Regulations, Title 24.

California Geological Survey, 2024 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://maps.conservation.ca.gov/cgs/EQZApp/app/

Clahan, K.B., Wagner, D.L., Bezore, S.P., Sowers, J.M., and Witter, R.C., 2005, Geologic Map of the Rutherford 7.5' Quadrangle, Sonoma and Napa Counties, California: A Digital Database, Version 1.0: California Geological Survey, Scale 1:24,000.

Dwyer, M.J., Noguchi, N., and O'Rourke, J., 1976, Reconnaissance Photo-Interpretation Map of Landslides in 24 Selected 7.5-Minute Quadrangles in Lake, Napa, Solano, and Sonoma Counties: U.S. Geological Survey, Open File Report 76-74, 25 Plates, Scale 1:24,000.