Appendix D:
Geotechnical Supporting Information

THIS PAGE INTENTIONALLY LEFT BLANK

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

March 17, 2017

Job No. 202.5.13

Oakmont Senior Living
9240 Old Redwood Highway, Suite 200
Windsor, CA 95492
Attention: Steve McCullagh

Report
Soil Engineering Consultation
Emerald Isle
Gullane Drive
Santa Rosa, California

This report presents the results of our soil engineering consultation for the planned Emerald Isle project in Santa Rosa, California. The site is located on the north side of Thomas Lake Harris Drive, northwest of Fountaingrove Lake, and access to the site will be provided by an extension of Gullane Drive. We performed a geotechnical investigation and geologic hazards evaluation for a proposed Skilled Nursing Facility on the property, and the results were submitted in our report dated September 21, 2016.

We understand that the Emerald Isle project is no longer being considered as a Skilled Nursing Facility. The project will now be developed as a planned Assisted Living Facility. A revised site plan dated February 2017 prepared by Brelje & Race indicates that the Emerald Isle Assisted Living Facility will include a much smaller, two-story building than previously proposed, with three detached one-story garages. Further, the building will be served by asphalt-paved driveway and parking areas with underground utilities. Based on our knowledge of the subsurface conditions, we judge that the general conclusions and recommendations contained in our previous geotechnical investigation report would be applicable to the currently proposed assisted living project.

We trust this provides the information needed at this time. If you have questions or wish to discuss this in more detail, please do not hesitate to contact us.

Yours very truly,

REESE & ASSOCIATES

Brian F. Piazza
Project Geologist

Jeffrey K. Reese
Civil Engineer No. 47753

REGISTERED PROFESSIONAL ENGINEER
JEFFREY KENT REESE
No. C 47753
EXP. 12-31-17
STATE OF CALIFORNIA

BFP/JKR:nay/ra/Job No. 202.5.13
Copies Submitted: 3

134 LYSTRA COURT
TELEPHONE (707) 528-3078

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS

SANTA ROSA, CA 95403
FACSIMILE (707) 528-2837

Report
Geotechnical Investigation
Emerald Isle Skilled Nursing Facility
Santa Rosa, California

Prepared for
Oakmont Senior Living
9240 Old Redwood Highway, Suite 200
Windsor, CA 95492
Attention: Dave Hunter

By

REESE & ASSOCIATES
Consulting Geotechnical Engineers

Brian F. Piazza
Project Geologist

Jonathan D. Morris
Geotechnical Engineer No. 2911

REGISTERED PROFESSIONAL ENGINEER
JONATHAN DANIEL MORRIS
No. GE 2911
Exp. 9-30-17
STATE OF CALIFORNIA

Jeffrey K. Reese
Civil Engineer No. 47753

REGISTERED PROFESSIONAL ENGINEER
JEFFREY KENT REESE
No. C 47753
EXP. 12-31-17
STATE OF CALIFORNIA

Job No. 202.5.1
September 21, 2016

INTRODUCTION

This report presents the results of our geotechnical investigation and geologic hazards evaluation for the proposed Emerald Isle Skilled Nursing facility located in Santa Rosa, California. The site is located on the top of a hillside, just northwest of Fountaingrove Lake, as shown on the attached Plate 1.

The proposed development will consist of the construction of a one-story, wood-frame structure with a concrete slab-on-grade floor. The building will be served by asphalt-paved driveway and parking areas and underground utilities. Access to the site will be provided by an easterly extension of Gullane Drive. Site grading within the building and driveway/parking areas is anticipated to include cuts varying in depth up to about 16 feet, with fills in the access road area varying up to about 10 feet deep. Retaining walls may be needed as part of the site development.

A site plan indicating the approximate location of the proposed building and associated improvements are shown on Plate 2.

PURPOSE AND SCOPE

The object of our investigation, as outlined in our proposal dated March 31, 2016, was to review selected geologic references, explore subsurface conditions, measure depth to groundwater, and determine physical properties of the soils encountered. We then performed engineering analyses to develop conclusions and recommendations concerning:

1. Soil and groundwater conditions observed.

2. Site preparation and grading.

3. Mitigation measures for identified geologic hazards, if appropriate.

4. Foundation support and design criteria, including an estimate of anticipated total and differential settlements.

5. Support of concrete slab-on-grade floors.

6. Retaining wall design criteria.

7. Quality and compaction criteria for development of asphalt-paved driveways and parking areas.

8. Geotechnical engineering drainage.

9. Supplemental geotechnical engineering services.

In consultation with Wagner & Bonsignore, we performed a Geologic Hazards Evaluation of the site. The results are presented in our report dated September 20, 2016. A copy of that report is included herein as Appendix A. Mr. David Peterson, Certified Engineering Geologist of Wagner & Bonsignore was instrumental in planning the evaluation, exploring subsurface conditions, research, analyses and development of conclusions and recommendations and co-authored the report.

In addition, we retained Miller Pacific Engineering Group to develop site specific seismic design criteria for the project. The results are presented in their report dated June 9, 2016 and are included herein as Appendix B.

WORK PERFORMED

On April 27 through April 29, 2016 and May 5, 2016 our consulting certified engineering geologist and project geologist were at the site to map the general surface geology and explore subsurface conditions to the extent of seven (7) test trenches and ten (10) test pits. The approximate trench and pit locations are shown on the attached Plate 2. The test trenches and pits were excavated to depths ranging from about 3 to 11 feet with a track-mounted Takeuchi TB 145 excavator. Our geologists observed the excavations, logged the conditions encountered and obtained samples for visual classification and laboratory testing. In-place strength indicator determinations were conducted in the pit and trench walls with a penetrometer. Logs of the trenches and pits are presented on Plates 3a through 3g and Plate 4, respectively. The soils are classified in accordance with the Unified Soil Classification System explained on Plate 5. Rock physical characteristics are described on Plate 6.

Selected samples were tested in our laboratory to determine classification (Atterberg Limits and percent free swell). The test results are shown on Plate 7 and also include the penetrometer data. Detailed results of the Atterberg Limits tests are presented on Plate 8.

The test pit and trench locations indicated on Plate 2 are approximate and were established by visually estimating from existing surface features. The locations should be considered no more accurate than implied by the methods used to establish the data. All of the trenches and pits were backfilled at the completion of the exploration with the soil and rock materials generated during excavation.

PREVIOUS INVESTIGATIONS

Several geotechnical investigations have been performed on the subject property and in the immediate site vicinity. The results of the previous investigations have been summarized and discussed in detail in the appendixed Geologic Hazards Evaluation report. Of note, a soil investigation was performed for the subject site in 2006 by Giblin Associates (GA). In that report, a 70-foot building set-back zone was recommended near the top of a steep, descending slope, located at the southwest portion of the project area.

SITE CONDITIONS

General Site Description

The subject site consists of an undeveloped, isolated hill/knoll with moderate to steep side slopes. Elevations range from approximately 460 to 570 feet above sea level. The site is bounded by an existing golf course on all sides.

Surface drainage is poorly defined with runoff generally occurring as sheet flow. However, some runoff appears to be concentrated into subdued drainage swales.

Vegetation includes oak forest, poison oak, and localized open areas of grass and brush. An abundance of large, hard boulders occur over large portions of the project area.

Geology and Soils

A thorough discussion of the regional and site geology, including the bedrock units are provided in the appendixed Geologic Hazards Evaluation report. Soil deposits in the proposed building area are comprised of weak surface soils and expansive clays/silts and residual soils, with colluvium on the side slopes. The upper soils are generally thin, and form a veneer that obscures the bedrock.

In test pit excavations performed within or adjacent to the proposed building area (near the top of the knoll), we observed about 12 to 18 inches of sandy silt with varying amounts of andesite rock fragments, cobbles and occasional boulders. These surface soils were observed to be porous, likely from prior cultivation and decomposition.

The residual soils were generally observed in areas underlain by andesitic tuff breccia (Tstb) and tuff (Tst). These residual soils consist of expansive clays and silts and are generally localized to the northwest portion of the proposed building area. Based on laboratory tests, the material is of moderate to high expansion potential. Expansion potential is a measure of the tendency of soils to undergo strength and volume changes with seasonal variations in moisture content.

Test trench 6 and test pits 8, 9 and 10 were excavated along the proposed new access roadway. Trenches excavated along this west facing slope encountered a thin layer of soft, sandy silt topsoil underlain by very firm andesite breccia (Tsab). Extending downslope, the rock materials transition to a welded tuff. The soils observed between the topsoil and welded tuff become increasingly thicker and consist of medium dense tuffaceous sand with a significant fraction of silty/clayey fines. The tuffaceous sand was observed to be wet and underlain by a thin, discontinuous layer of expansive clay. No slickensides were observed at the contact between the tuffaceous sand and clay.

Areas interpreted to have locally deeper accumulations of natural soil/colluvium (3 feet or greater) were mapped as Qc and are shown approximately on Plate 2. Detailed soil descriptions and depths are provided on the test pit logs.

Groundwater

Neither groundwater nor seepage was observed in any of the test trenches or test pits. Our experience indicates that groundwater could be present seasonally in a perched condition. Perched groundwater commonly occurs where relatively permeable strata are underlain by low permeability soil or rock. The porous upper soils and soil near the underlying bedrock contact would be expected to become saturated seasonally resulting in seepage through the soil as well as along the soil-rock contact. Sporadic, localized seepage could also occur through rock fractures. Determination of the precise groundwater location, or the presence of a perched water condition, is beyond the scope of this investigation.

Slope Instability and/or Slope Creep

As discussed in the Geologic Hazards Evaluation report, in test trenches TT-3 and TT-7, near vertical soil-filled features were encountered that could be traced to the bottom of the trenches. Because no offset in the bedrock units in the form of down-dropped and/or displaced bedrock units were observed across these features, nor were there slickensided shear planes that would be suggestive of landsliding, we judge that this may represent weathering along fractures.

As indicated in the appendixed report, we did not observe any features indicative of past instability or secondary effects from the older S-1 bedrock slide that extends up to the current project site. Further, the proposed building is currently set-back about 75 feet from the potential headscarp.

We judge that the upper soils on the west-facing slope are undergoing soil creep. Creep is the gradual downslope movement of weak soil and soft rock, on the order of a fraction of an inch per year, under the force of gravity. In test trench 6, because of the presence of relatively soft, wet tuffaceous sand overlain by expansive clay, it is likely that soil creep affects both the soil deposits as well as uppermost about one foot of rock. The areas suspected to be undergoing creep are depicted schematically on the attached Plate 2.

DISCUSSION AND CONCLUSIONS

Based on the results of our investigation, we conclude that, from a geotechnical engineering standpoint, the site can be used for the proposed construction. The most significant geotechnical engineering factors that must be considered in design and construction are:

1) The presence of weak, porous topsoils and moderate to highly expansive soils;
2) Localized areas of soils subject to creep;
3) The steep descending slope associated with the S1 bedrock slide in the southwest portion of the project;

4) Areas of hard bedrock and/or large boulders;

5) A potential for very strong to possibly violent seismic ground shaking.

Weak, porous natural soils, such as those encountered at shallow depths throughout the project area, would be subject to significant settlements when under load, particularly when saturated. Where evaporation is inhibited by slabs, footings, or fill, eventual saturation could occur. Therefore, we judge that the weak, porous upper soils are not be suitable for foundation or slab support in their present condition.

Expansive soils can undergo significant strength and volume changes with seasonal variations in moisture content and can heave and distress lightly loaded footings and slabs. Therefore, for slab-on-grade support, it will be necessary to verify that expansive soils have not dried and cracked. Also, expansive soils encountered within building envelopes would need to be removed for their full depth or covered with a moisture confining and protecting blanket of approved on-site or imported materials of low expansion potential.

Where weak, porous and/or plastic, clayey or colluvial soils are on a slope, the materials are subject to soil creep. Creep is a common phenomenon that occurs to varying degrees on most hill slopes in Sonoma County. Such creep soil movements can impose lateral loads on foundations, and contribute to differential settlement of slabs, walkways, roads and other project improvements, and result in tilting, lateral displacement and/or more than normal cracking. Creep effects can be reduced by grading measures such as overexcavation of creep-affected soil and replacement as properly keyed, benched and compacted fill and by designing foundations and retaining walls to resist lateral creep soil loads. Therefore, we conclude that, where applicable, design and construction of fills, cuts, foundations, retaining walls and slabs must recognize the presence of creep-affected soils, as recommended in this report.

Based on our review of the preliminary conceptual plan provided to us, we understand that a preliminary building pad and finished floor grade have been established. Based on the indicated grades, it appears that the building will be positioned in areas of planned cut extending up to about 13 feet deep. Throughout the planned building envelope, the volcanic rocks are considered competent and capable of supporting the proposed structure. However, varying rock types were observed beneath the proposed building area that have differing engineering properties. Where more tuffaceous rocks where observed, they were locally deeply weathered and of lower strength than the flow rock. Accordingly, to help provide more uniform support and satisfactorily reduce the potential for differential settlements and resultant distress, we judge that satisfactory foundation support for the proposed structure can be obtained from either:

1) Spread footings bottomed at relatively shallow depths on properly compacted fill, or;

2) A mat- or post-tensioned slab foundation designed to recognize a potential for differential settlements.

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

The backhoe pits and test trenches were backfilled with the excavated materials, but the soils were not compacted. Therefore, the pit and trench backfills constitute local deep zones of material subject to significant settlement and/or erosion. Where encountered in planned improvement areas, compressible pit and trench backfills should be removed for their entire depth and replaced as properly compacted soils or foundation elements deepened accordingly.

Steep Descending Slope

The top of the steep descending slope on the southwest-facing side of the project area associated with the S1 bedrock slide is located about 75 feet from the corner of the proposed building. Because no secondary features associated with slope movement were observed in Test Trench TT-2, we judge that no mitigation measures are warranted.

Excavation of Bedrock and Large Boulders

The andesitic lava flow rocks and andesite flow breccia (Tsb and Tsab) bedrocks and bouldery soils were difficult to excavate with the small excavator used for the test pits. The hard, strong and highly variable composition of the volcanic rocks and bouldery soils may result in locally difficult excavation and/or drilling conditions. Hard volcanic rocks may be difficult to excavate using conventional grading techniques. Consequently, heavy ripping, jack hammering, and/or hoe ramming should be anticipated. Numerous large boulders could result in uneven cut surfaces, difficult trench and/or footing excavations, hard and slow footing excavation and the generation of considerable quantities of boulders requiring removal from the site, stockpiling and/or use as part of the project landscaping. Such oversized rocks could be used in deeper fills, but would need special placement and compaction procedures.

Nearby cut slopes along Thomas Lake Harris, Fountaingrove Expressway and at the golf course indicate the volcanic rocks perform adequately in cut slopes inclined at about 2:1 (horizontal to vertical) at heights up to 8 to 10 feet. Therefore, 2:1 cut slopes are considered appropriate for project planning. If steeper or higher cuts are planned, site specific evaluations would be appropriate.

SEISMIC DESIGN PARAMETERS

A discussion of the site specific seismic design criteria is provided in Appendix B of this report.

RECOMMENDATIONS

Site Grading

Areas to be graded should be cleared of any debris, rubble and vegetation. Designated trees should be removed and the root systems excavated. The resultant voids should be backfilled as subsequently recommended. The surface then should be stripped of upper soils containing root growth and organic matter. We anticipate that the depth of stripping will average about 3 inches. The strippings should be removed from the site or stockpiled for reuse in landscape areas.

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

After clearing and stripping, excavation should be performed as necessary. We judge that with the exception of organic matter and rocks or hard fragments larger than 4 inches in diameter, in general, the excavated materials would be suitable for use as compacted fill. However, expansive clayey or silty soils should not be reused as compacted fill within the upper portions of fill pads where minimum depth foundations and concrete slab-on-grade floors are planned, as subsequently discussed. Larger rocks, boulders or hard rock fragments could be generated in considerable quantities during excavations. These materials could be placed in deeper fills, but should not be placed where they could be encountered in subsequent foundation or utility trench excavations. Rocks should not be allowed to nest and should be placed such that proper compaction is attained in the surrounding fill soils. Where considerable quantities of excess boulders and/or cobbles are generated, we should provide specific recommendations as placement in compacted fills, if proposed.

Where minimum depth footings and conventional concrete slab-on-grade floors are desired, weak upper natural soils should be removed (overexcavated) for their full depth. Soil and rock materials exposed at planned subgrade level should be similarly overexcavated. Overexcavations in such areas should extend at least 5 feet beyond the building perimeter, 3 feet beyond the edge of building foundations, or 3 feet beyond adjacent exterior concrete slabs that abut the building, whichever is greater. Overexcavation depths then should be adjusted so as to provide space for at least 12 inches of approved compacted fill below all footings and slabs and, in expansive soil areas, to a sufficient depth so as to provide space for a minimum 24-inch-thick moisture confining and protecting blanket of approved on-site soils of low expansion potential or imported nonexpansive fill.

Overexcavation depths to remove weak porous natural soil in proposed building areas are anticipated to vary from about 2 to 4 feet below the existing ground surface. It should be understood that the indicated depths are only approximate, and are based on the conditions observed during our site exploration. Actual depths of overexcavations could vary substantially, and should be determined in the field by the soil engineer during the site grading work when the actual conditions are exposed for further observation. The indicated overexcavation depths are intended to provide a guideline for use in site grading cost estimates. Because of the possible presence of additional weak compressible soils that could be encountered within improvement areas during site grading, we recommend that the contract contain provisions for variations and additional overexcavation of weak compressible soils.

The surfaces exposed by stripping or overexcavation should be scarified at least 6 inches deep, moisture conditioned to at least 2 percentage points above optimum (at least 4 percentage points for expansive clayey soils) and compacted to at least 90 percent relative compaction[1]. The moisture conditioning should be sufficient so as to close shrinkage cracks, if any, for their full depth. Approved, excavated and/or imported fill should be placed in layers, similarly moisture conditioned and compacted to at least 90 percent.

Where fills are planned on slopes 8:1 or steeper or where it is recommended to remove and replace creep-affected soils, level keyways at least 10 feet wide should be excavated along the toe of the planned new fills. The keyway should bottom into firm underlying soil or bedrock below weak, upper soils and creep-affected materials. For estimating purposes, the depth of keyways can be assumed to extend at least 5 feet below the original ground surface, as measured on the downhill side (8 feet or more in heavy creep soil areas for the proposed driveway fill). Subsurface drainage facilities will be needed at the rear of the keyway(s) and may be needed at intermediate bench levels, as determined in the field by the soil engineer. As fill placement continues upslope, level benches should be excavated into firm underlying soil or bedrock to key the new fills into the hillside and remove the existing creep-affected materials. The face of finished fill slopes should be thoroughly compacted by slope rolling and trimming or constructed wider than planned and then trimmed to expose dense, well compacted material. Plate 9 shows a typical cross-section of our general recommendations for hillside fill placement. It should be understood that the cross-section is a typical representation of hillside grading and keyways. The

1 Relative compaction refers to the in-place dry density of fill expressed as a percentage of maximum dry density of the same material determined in accordance with the ASTM D 1557 laboratory compaction test procedure. Optimum moisture content refers to the moisture content at maximum dry density.

actual location, depth, and extent of keyways and subdrains should be determined in the field during grading when actual conditions are exposed. Therefore, we recommend that the contract contain provisions for variations in keyway excavation(s), and additional subdrainage quantities, if appropriate.

Imported fill, if used, should be nonexpansive and have a Plasticity Index of 13 or less. Imported fill material should be free of organic matter and rocks or hard fragments larger than 4 inches in diameter. Imported fill should be tested and approved by the soil engineer prior to importation to the site.

Cut Slope Criteria

For purposes of project planning, unsupported cut slopes should be inclined no steeper than 2:1. A range of different rock and soil conditions are present within the project site, and the physical properties of the rocks can vary widely over short distances from hard and strong to weak and friable. Potentially unstable conditions could be encountered during rock excavation because of the potential to expose adverse planes of weakness related to bedding surfaces and/or fractures and shears. Slopes excavated and subjected to periods of wet weather and/or long periods of time without support would be expected to have an increased potential for slope instability. We recommend that cut slopes and excavations be observed by the soil engineer and/or geologist during site grading to evaluate the need for modifications to cut slope inclinations or other measures to reduce the risk of future slope instability. Temporary stability of excavations during construction of retaining walls or other slope stabilization measures should be the responsibility of the contractor.

Foundations

Footings - Provided the site is graded in accordance with the recommendations outlined above, foundation support for the proposed building can be provided by spread footings bottomed on properly compacted fill. Footings should be at least 12 inches wide. Footings bottomed on at least 12 inches of properly compacted fill of low expansion can be a minimum of 18 inches deep, as measured below lowest adjacent compacted pad elevation. On slopes, footings should be stepped, as necessary, to provide level (and up to 10 percent slope) bottoms.

Spread footings can be designed to impose dead plus code live load (DL & LL) and total design load (TDL), including wind or seismic forces, bearing pressures of 2,000 and 3,000 pounds per square foot (psf), respectively.

Resistance to lateral loads can be obtained from passive earth pressures and soil friction. We recommend the following criteria for design:

Passive Earth Pressure	=	300 pounds per cubic foot (pcf) equivalent fluid, neglect the upper 1 foot, unless confined by pavement or slab, and within 8 horizontal feet from the face of the nearest slope
Soil Friction Factor	=	0.30

Footings should be reinforced and designed to span at least 3 feet of nonsupport. To help tie the foundation together, northeast/southwest-oriented footings or tie beams should be spaced no further apart than about 60 feet. Tie beams, if used, should be at least 12 inches square and contain at least two No. 5 (or three No. 4) reinforcing bars.

Mat-Slab - Following excavations to pad grade, as an alternative, the proposed building can be supported on a mat-slab. For design, an allowable bearing value of 1,000 psf can be used. Mat slabs should be at least 10 inches thick and contain two courses of steel, as determined by the project structural design engineer. Slabs should be provided with an additional 2-inch thickened edge for stiffening, and be reinforced so as to be capable of spanning a minimum distance of 6 feet in any direction. The slab should be designed to cantilever at least 3 feet at the perimeter.

Retaining Walls

Retaining walls that are free to rotate slightly and support level backfill should be designed to resist an active equivalent fluid pressure of 40 pcf acting in a triangular pressure distribution. Where the backfill slope is steeper than 3:1, the pressure should be increased to 55 pcf. If the wall is constrained at the top and cannot tilt, the design pressures for level and sloping backfill should be increased to 55 and 70 pcf, respectively. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an added surcharge pressure equivalent to 1½-foot of additional backfill.

As outlined in the 2013 CBC, it may be necessary to design retaining walls to resist additional lateral soil loads imposed during seismic shaking. Accordingly, based on the

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

Backslope Inclination (β)	Dynamic Component* of Total Thrust (lbs/ft)
$0 \leq \beta \leq 8{:}1$	$13H^2$
$8{:}1 < \beta \leq 4{:}1$	$19H^2$
$4{:}1 < \beta$	$29H^2$

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

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

Conventional Slab-on-Grade

Provided the building envelope is prepared as recommended above, conventional concrete slab-on-grade floors can be used. Such slabs should be underlain by properly compacted, approved on-site or imported fill materials of low expansion potential.

The floor slabs should also be underlain with a capillary moisture break and cushion layer consisting of at least 4 inches of free-draining gravel or crushed rock (slab rock). The gravel or crushed rock should be at least 1/4-inch and no larger than 3/4-inch in size. Moisture vapor will condense on the underside of slabs. Where migration of moisture vapor through slabs is detrimental, a 10-mil minimum vapor retarder conforming to ASTM E1745 Class C should be provided between the supporting base material and the slabs. Two inches of moist, clean sand could be placed on top of the membrane to aid in curing and to help provide puncture protection. However, the actual use of sand should be determined by the architect or design engineer. The use of a less permeable and stronger membrane should be considered if sand is not to be placed for puncture protection, or where the flooring manufacturer requires a vapor barrier. Concrete design and curing specifications should recognize the potential adverse affects associated with placement of concrete directly on the membrane.

Slabs should be at least 6 inches thick and be reinforced with bars. Actual slab thickness and reinforcing should be determined by the structural design engineer based on anticipated use and performance. Prior to placing the reinforcing or slab rock, the subgrade soils should be thoroughly moistened and be smooth, firm and uniform. Slab subgrade should not be allowed to dry prior to concrete placement.

To help provide an outlet for water that could accumulate in the underslab rock and reduce the risk of future moisture migration up through the floor slab, the installation of perforated plastic pipes, at least 40 feet long, embedded in the grade below the underslab rock should be considered. The underslab subdrain system, if installed, should be connected to a non-perforated outlet pipe that extends through or beneath the perimeter foundation to a suitable discharge point. A typical cross-section of our recommended underslab subdrain is shown on the attached Plate 10. We should provide additional consultation concerning the actual need for and configuration and location of the underslab subdrain system during final design.

Driveway and Parking Areas

The flexible pavement materials should conform to the quality requirements of the State of California Caltrans Standard Specifications, current edition, and the requirements of the City of Santa Rosa.

Prior to subgrade preparation, all underground utilities in the paved areas should be installed and properly backfilled. Subgrade soils should be uniformly moisture conditioned to slightly above optimum, compacted to at least 95 percent relative compaction, and provide a firm and unyielding surface. This may require scarifying and recompacting to achieve uniformity. The aggregate base materials should be placed in layers no thicker than 6 inches, be compacted to at least 95 percent, and form a firm and unyielding surface.

Geotechnical Drainage

Ponding water will cause softening of site soils and would be detrimental to foundations. It is important that the areas adjacent to the foundations be sloped to provide positive drainage away from foundations. A gradient of at least 1/4-inch per foot, extending at least 4 feet out, should be maintained. The roofs should be provided with gutters, and the downspouts should be connected to pipelines that discharge into planned storm drains or onto paved areas.

Careful attention to fine (finish) grading around the building should be provided. No loose or poorly compacted materials should be allowed adjacent to grade beams, and underslab drainage improvements, as previously discussed below, should be installed.

All surface drainage should be diverted away from cut and fill slope faces by means of top-of-slope ditches, berms or approved equivalents. Runoff should be directed to non-erosive drainage devices or ditches that channel the water away. Pavement surfaces should be constructed to provide positive drainage and not allow ponding.

In general, to inhibit buildup of moisture in aggregate base materials and subgrade soils, subgrade subdrains will be needed. Subgrade subdrains should consist of trenches that are at least 12 inches wide and extend at least 12 inches below compacted pavement subgrade elevation. Three-inch-diameter, perforated rigid plastic pipes (schedule 40 or equivalent) should be placed on a layer of crushed gravel or drainrock in the bottom of the trenches. The trenches should be backfilled up to planned subgrade level with similar crushed gravel or drainrock. The drainrock or gravel should be encased with a nonwoven geotextile fabric weighing at least 4 ounces per square yard. Subgrade subdrains should outlet by gravity to suitable discharge points, well away from the downhill edge of pavement. The actual need for and location of subgrade subdrains should be determined during final design once plans have been generated.

Where irrigated landscape areas abut the building, excess water can be introduced into soil layers along the edge of the building, tending to soften soils, and induce extra moisture into underfloor areas. We believe that the installation of the recommended compacted fill pad that extends to at least 5 feet beyond building perimeters should provide an effective barrier to the infiltration of excess waters from landscape areas. However, as an added precaution, such landscape areas should be provided with a drain that outlets into planned site drainage systems (gutters, storm drains, etc.). Also, hot-mopping or other methods of waterproofing the exterior sides of below grade cold joints in perimeter spread footing foundations should be performed.

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

Supplemental Services

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

We should observe footing excavations to verify that the conditions are as anticipated and to modify our recommendations, if warranted. Concrete placement and reinforcing should be checked as stipulated on the project plans or as required by the Building Department. It is our understanding that approval from the Building Department must be obtained prior to the placement of concrete in foundation elements. The soil engineer and/or geologist should observe keyways and benches, determine locations and extent of creep soil removal, overexcavations and subsurface drainage facilities and modify our recommendations, if warranted.

LIMITATIONS

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

Subsurface conditions are complex and may differ from those indicated by surface features or encountered at test pit and test trench locations. Therefore, variations in subsurface conditions not indicated on the logs could be encountered.

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

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

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

LIST OF PLATES

APPENDIX A

Report
Geologic Hazards Evaluation
Emerald Isle Skilled Nursing Facility
Santa Rosa, California
by Reese and Associates
dated September 20, 2016

APPENDIX B

Report
Site Specific Seismic Design Criteria
Emerald Isle Skilled Nursing Facility
Santa Rosa, California
by Miller Pacific Engineering Group
dated June 9, 2016

DISTRIBUTION

Copies submitted: 5

Oakmont Senior Living
9240 Old Redwood Hwy, Ste 200
Windsor, CA 95492
Attention: Dave Hunter

BFP/JKR:nay/ra/Job No. 202.5.1

REFERENCES

Association of Bay area Governments (ABAG), April 1995, *The San Francisco Bay Area – On Shaky Ground*, Publication Number P95001EQK.

California Geological Survey (formerly Division of Mines and Geology), September 27, 1982, *Fault Evaluation Report FER-141,* (Rodgers Creek fault)

California Geological Survey, March 13, 1997 *"Guidelines for Evaluating and Mitigating Seismic Hazards in California,"* Special Publication 117.

Dwyer, M. J. and others, 1976, *Reconnaissance Photointerpretation Map of Landslides in 24 Selected 7.5 Minute Quadrangles in Lake, Napa, Solano and Sonoma Counties, California,* U.S. Geological Survey Open File Map sheet 14 of 25.

Fox, K. F., 1983, *Tectonic Setting of Late Miocene, Pliocene and Pleistocene Rocks in Part of the Coast Ranges North of San Francisco, California*, U.S. Geological Survey Professional Paper 1239.

Fox, K. F., and others, 1973, *Preliminary Geologic Map of Eastern Sonoma county and Western Napa county, California*, U.S. Geological Survey Map MF-483.

Giblin Associates, March 28, 2006, *Assessment of Geologic Hazards, The Oaks, Phases 3, 4 and 5, Santa Rosa, California,* Job No. 3221.3.1-1, prepared for Aegis Senior Living.

Giblin Associates, March 31, 2006, *Geotechnical Investigation, The Oaks, Phases 3, 4 and 5, Santa Rosa, California,* Job No. 3221.3.1, prepared for Aegis Senior Living.

Giblin Associates, October 4, 2006 (Revised), *Geotechnical Investigation, The Oaks, Phases 4 and 5, Santa Rosa, California,* Job No. 3221.3.1, prepared for Aegis Senior Living.

Giblin Associates, December 7, 2007, *Investigation of Fault Surface Rupture Hazard, Fountaingrove Lodge, Santa Rosa, California, Volumes 1 and 2,* Job No. 3221.3.1-1, prepared or Aegis Senior Living.

Harding, Miller, Lawson & Associates, November 16, 1971, "Geologic Reconnaissance and Soil Engineering Feasibility Study, Fountain Grove Property, Santa Rosa, California."

Harding, Miller, Lawson & Associates, December 28, 1971, "Geophysical Investigation For Excavation Conditions, Fountain Grove Property, Santa Rosa, California."

Harding, Miller, Lawson & Associates, November 30, 1972, "Soil Investigation, Hewlett Packard Plant, Fountain Grove Property, Santa Rosa, California."

Harding-Lawson Associates, May 12, 1980, *Geologic Evaluation for EIR, TMI Property, Fountain Grove Ranch, Santa Rosa, California*, Job No. 5179,001.02, prepared for Teachers Management Investment Corporation.

Hart, Earl W., 1992, *Recently Active Traces of the Rodgers Creek Fault, Sonoma County,* California Division of Mines and Geology Open file report 92-7.

Huffman, M. E. and C. F. Armstrong, *Geology for Planning in Sonoma County, California* Division of Mines and Geology special Report 120.

Jennings, Charles W., 1988, *Preliminary Geologic Map of the Northwest Quarter of the Santa Rosa 7.5' Quadrangle, Sonoma County, California*, DMG Open File Report 88-5.

Jennings, Charles W., 1994, *Fault Activity Map of California and Adjacent Areas,* California Division of Mines and Geology, Geologic Data Map Series – Map No. 6.

McLaughlin, R. J., Langenheim V. E., Sarna-Wojcicki A.M., Fleck R. J., McPhee D. K., Roberts C. W., McCabe C. A., and Elmira Wan, 2008, *Geologic and Geophysical Framework of the Santa Rosa 7.5' Quadrangle, Sonoma County, California*, U.S. Geological Survey, Open-File Report 2008-1009.

McLaughlin, Robert J. and Sarna-Wojcicki, Andrei, 2003, *Geology of the Right Stepover Region between the Rodgers Creek, Healdsburg, and the Maacama Faults, Northern San Francisco Bay Region: A Contribution to the Northern California Geological Society Field Trip Guide*, June 6-8, 2003, U.S. Geological Survey OFR 2003-502

Moore & Taber, January 7, 1985, *Geotechnical Study, "Part I"– Faulting Evaluation, Phase III Cluster Residential ("North of Stonefield") Fountaingrove Ranch, Santa Rosa, California,* Job No. 3P3/583/102, prepared for Sonoma Financial Corporation

National Science Foundation (NSF), *Open Topography. A Portal to High-Resolution Topography Data and Tools,* http://www.opentopography.org/index.php

Reese & Associates, 2011, Report, Geotechnical Investigation, Fountaingrove Lodge, "Landslide Hazards Evaluation, Thomas Lake Harris Drive, Santa Rosa, California."

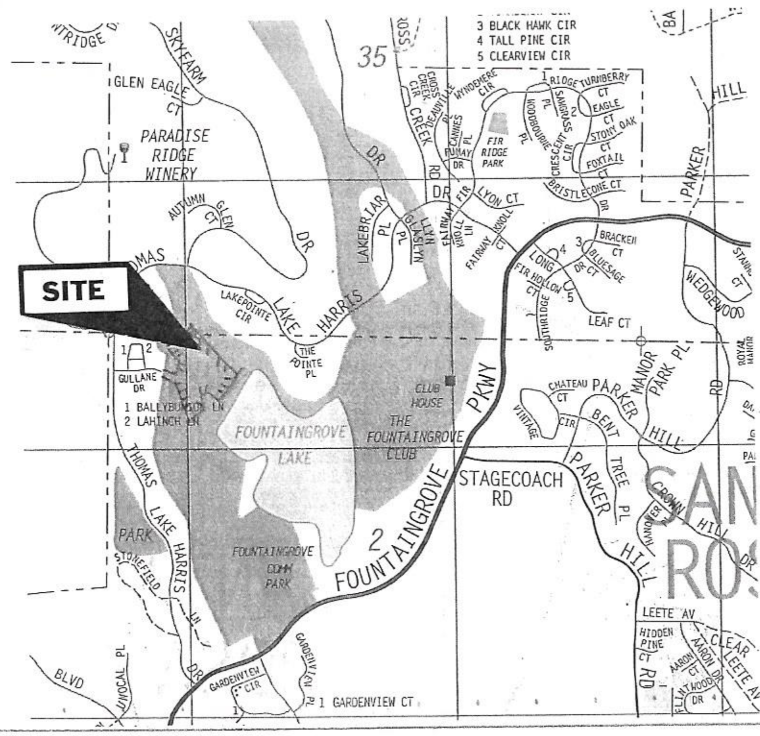
Taber Consultants, June 28, 1985, *Fault Evaluation ("Part 1") – Addendum, Phase-III Cluster Residential (North of Stonefield), Fountaingrove Ranch, Santa Rosa, California* Job No. 3P3/583/102-2, prepared for Sonoma Financial Corporation

Taber Consultants, September 15, 1988, *Geotechnical Study – Part II, Cluster Area No. 2 ("North of Stonefield"), Fountaingrove Ranch, Santa Rosa, California*, prepared for Braewood Development, Job No. 3P3/588/26.

Working Group on California Earthquake probabilities, 2002, *Earthquake Probabilities in the San Francisco Bay Region: 2002-2031*, U.S. Geological Survey Open-File Report 03-214.

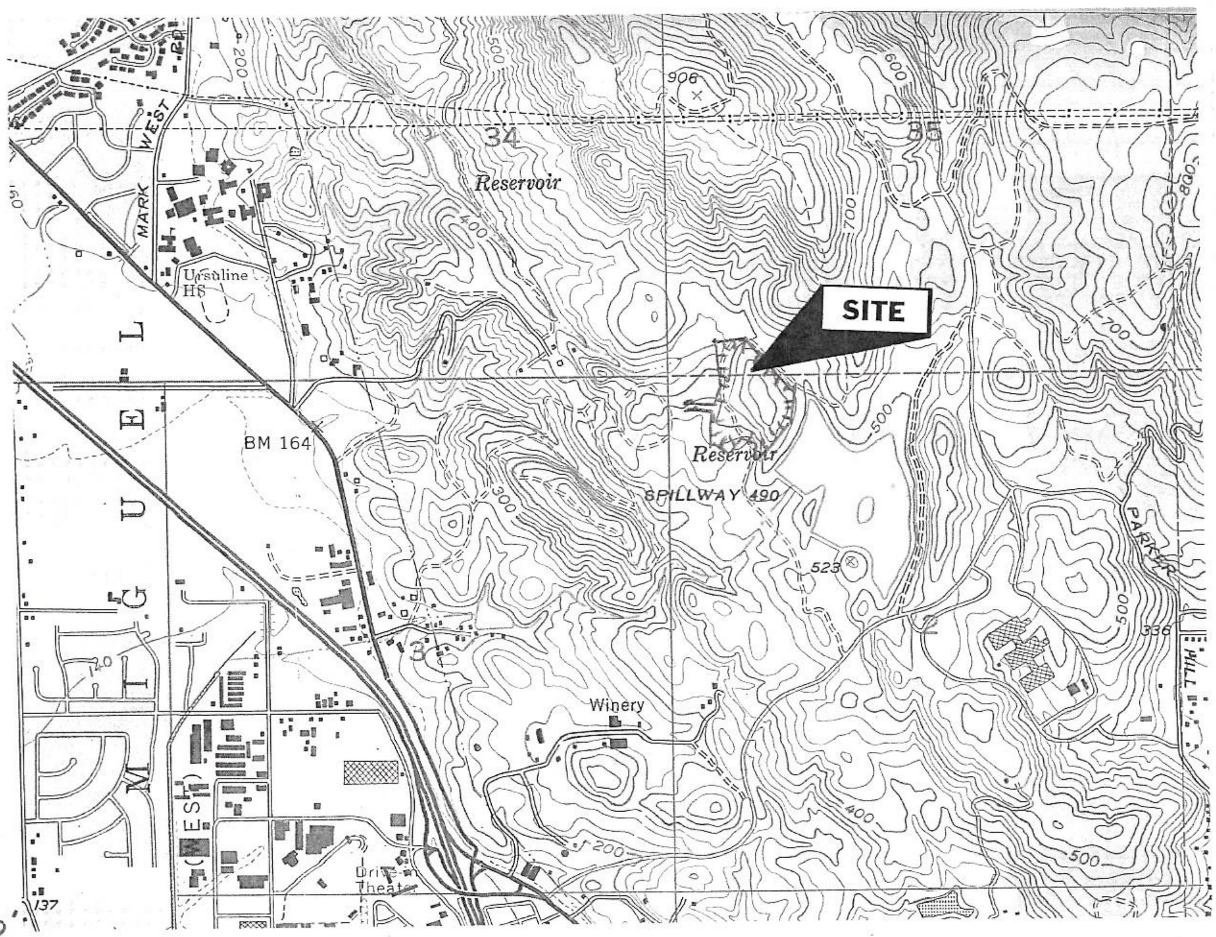
Aerial Photos

June, 10, 1961, CSH-6BB 106, 107; black and white, scale: 1/67,500
May 18, 1971, Cartwright Aerial Surveys, 3088-194, -195, scale: 1/24,000
March 13, 1973, Geo-graphics Aerial Survey, SR H-19, -20, scale (approx.): 1/6000
May 7, 1980, Sonoma Co. Assessor, BW-SON-18-18, -19, -20, scale: 1/24,000
May 19, 2000, Sonoma Co. Assessor, 50-013734, 48-22, -23, -24; scale (approx): 1/10,500



N

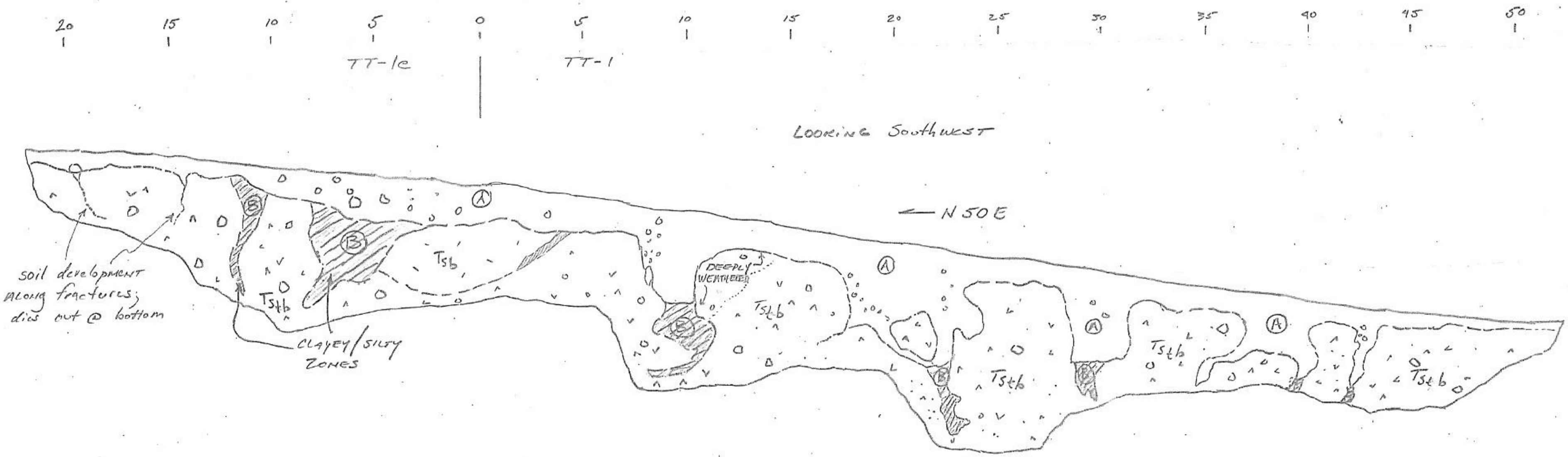
SCALE: 1" = 1900'



N

SCALE: 1" = 1900'

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	PROJECT LOCATION MAP EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE **1**



KEY

A: DARK BROWN SANDY SILT (ML), soft to medium stiff, moist, porous with rock fragments and abundant roots

B: DARK GRAY SANDY SILT (MH), stiff, wet, weathered zones within the bedrock

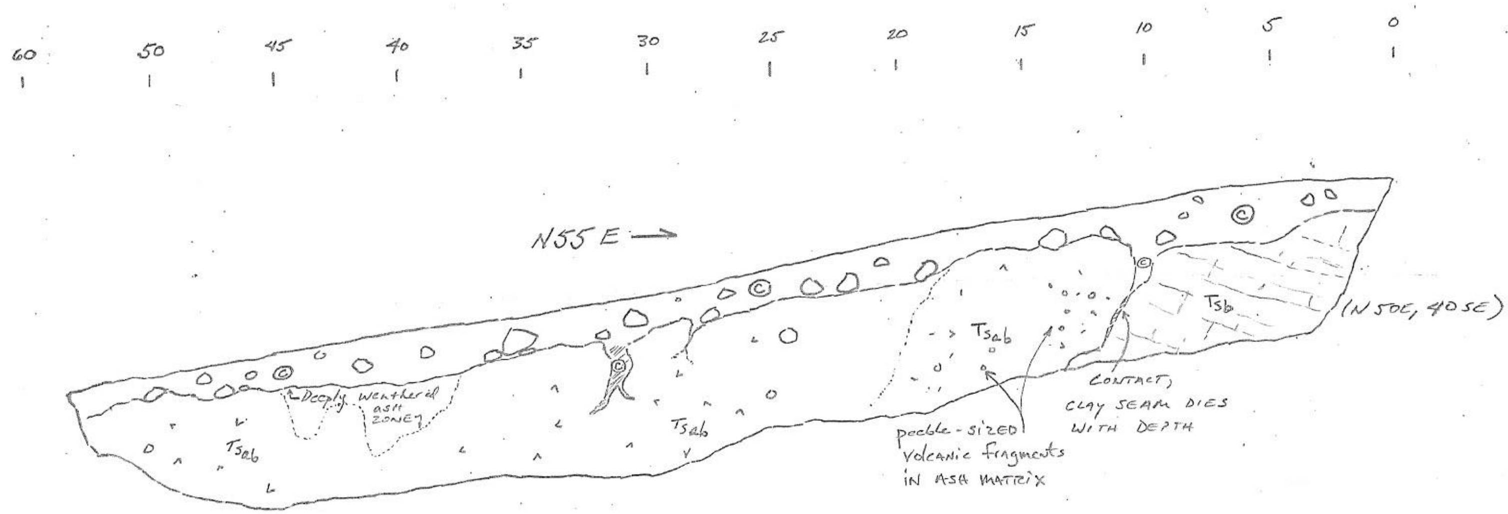
Tstb: YELLOW-BROWN ANDESITIC TUFF BRECCIA, compact, massive, moderately hard to hard, moderately strong to strong

Tsb: GRAY ANDESITIC LAVA FLOW, moderately hard to hard, moderately strong to strong

SCALE: 1" = 5'
0 5
(HORIZONTAL & VERTICAL)

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST TRENCH TT-1 AND TT-1e EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 3a

REDUCED FROM 11X17



EXPLANATION OF TT-2

C: DARK BROWN SILTY GRAVEL (GM), medium dense, moist, rubbly topsoil developed on Tsb, abundant roots

Tsb: GRAY ANDESITIC LAVA FLOW, moderately hard to hard, moderately strong to strong, moderately fractured, little weathered, with brown silt (ML) in fractures

Tsab: GRAY ANDESITIC BRECCIA, moderately hard to hard, moderately strong to strong, compact, massive, moderately fractured

Scale: 1" = 5'

0 5

(Horizontal & Vertical)

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

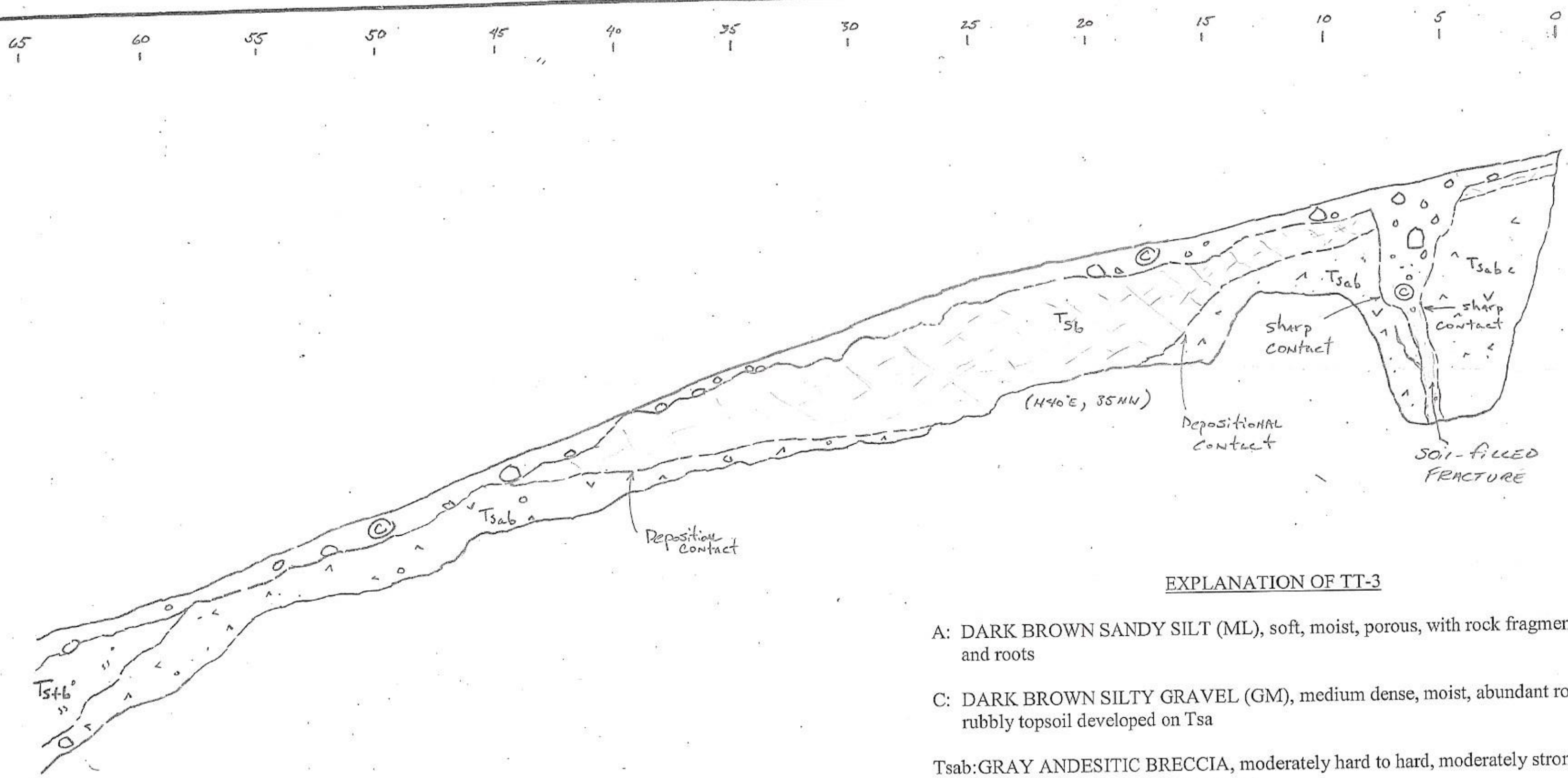
Job No: 202.5.1
Date: 08-29-16
Appr: BFP

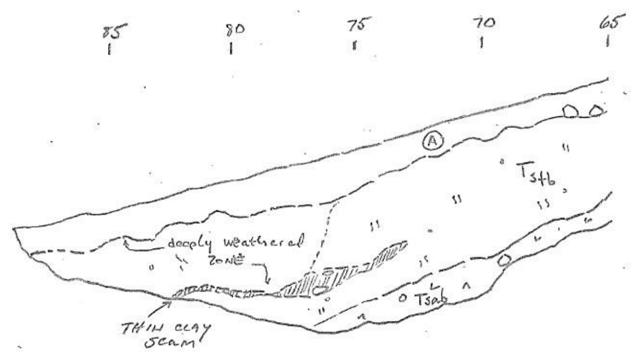
LOG OF TEST TRENCH TT-2

EMERALD ISLE SKILLED NURSING FACILITY
SANTA ROSA, CALIFORNIA

PLATE
3b

REDUCED FROM 11X17





SCALE: 1" = 5' HORIZONTAL & VERTICAL

0 5

EXPLANATION OF TT-3

A: DARK BROWN SANDY SILT (ML), soft, moist, porous, with rock fragments and roots

C: DARK BROWN SILTY GRAVEL (GM), medium dense, moist, abundant roots, rubbly topsoil developed on Tsa

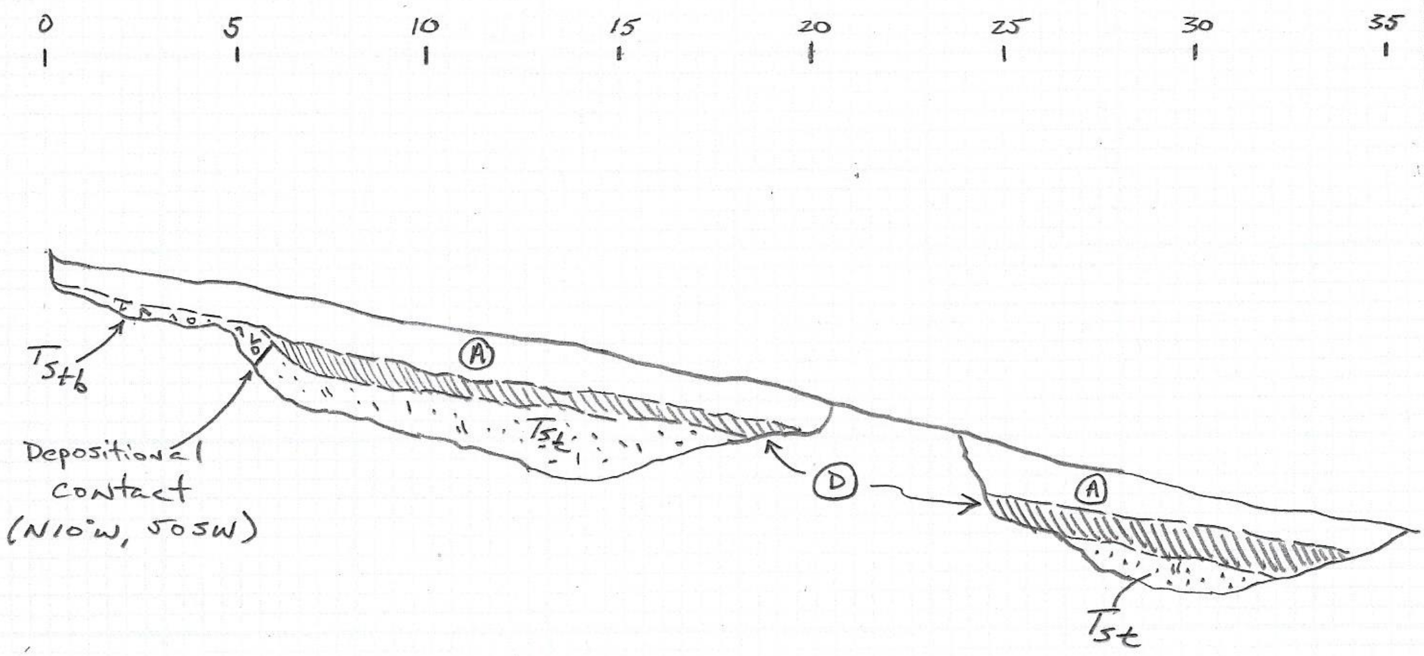
Tsab: GRAY ANDESITIC BRECCIA, moderately hard to hard, moderately strong to strong, massive to little fractured

Tsb: ANDESITIC LAVA FLOW, moderately hard to hard, moderately strong to strong, massive to little fractured

Tstb: YELLOW-BROWN TUFF BRECCIA, low hardness, friable, predominately ash with rock fragments 6 to 10 inches in diameter

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST TRENCH TT-3 EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 3c

REDUCED FROM 11X17



EXPLANATION OF TT-4

A: BROWN SANDY SILT (ML), soft to medium stiff, dry, with root, porous

D: GRAY- BROWN SANDY SILT (MH), stiff, moist, developed on Tst

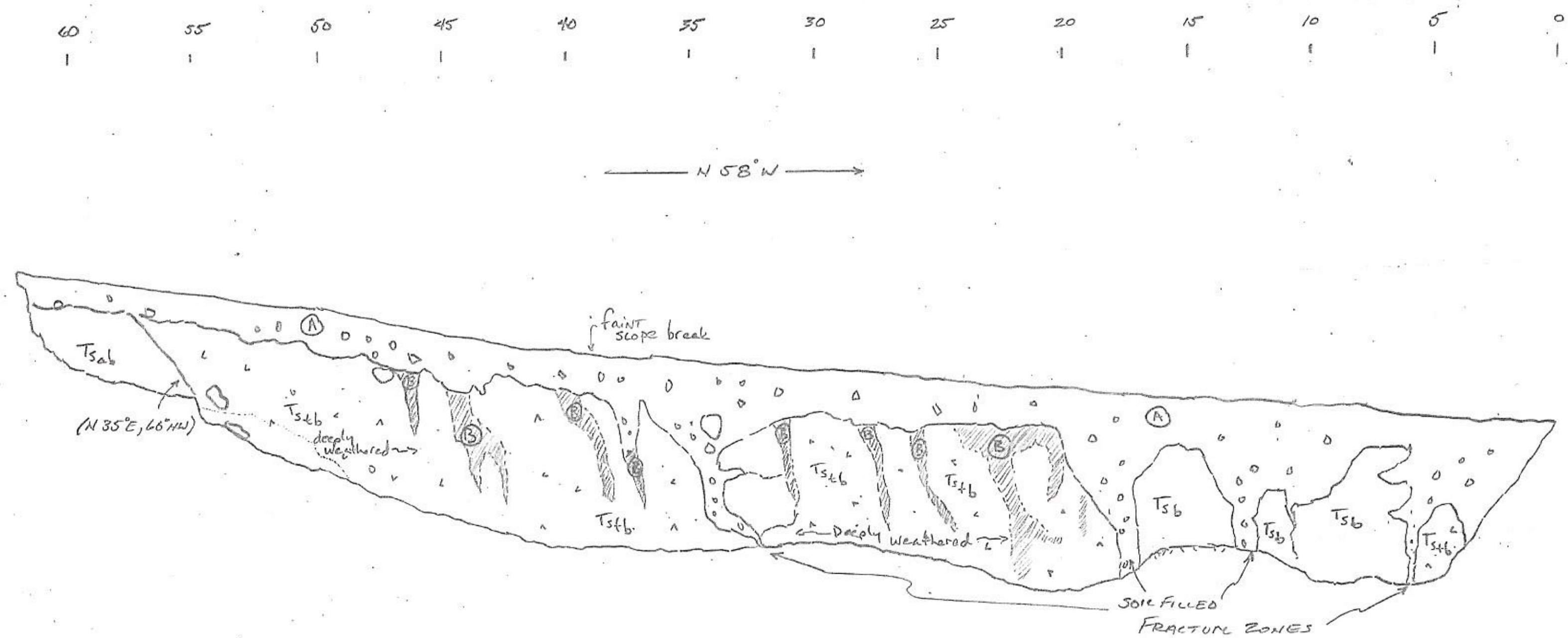
Tsab: YELLOW-BROWN TUFF BRECCIA, moderately hard, moderately strong, moderately weathered

Tst: LIGHT BROWN TUFF, low hardness, friable, massive, little fractured, fine to medium grained ash matrix

Scale: 1" = 5' Horizontal & Vertical

0 5

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST TRENCH TT-4 EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 3d



EXPLANATION OF TT-5

A: RED-BROWN SANDY SILT (ML), soft, moist, with abundant roots and rock fragments

B: RED-BROWN SANDY SILT (MH), stiff, wet, weathered zones within the bedrock

Tsab: GRAY-YELLOW-BROWN ANDESITIC BRECCIA, hard, massive, little fractured

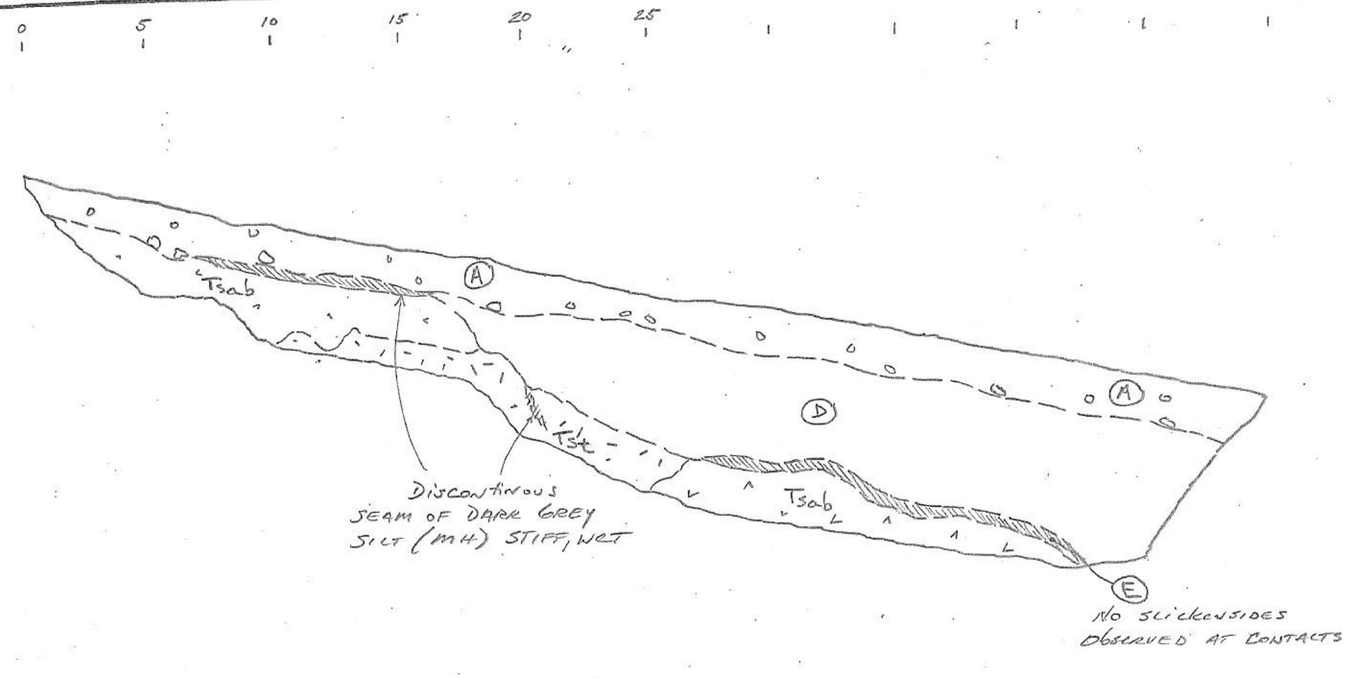
Tstb: BROWN TUFF BRECCIA, low to moderately hard, moderately strong, deeply weathered

SCALE: 1" = 5' HORIZONTAL : VERTICAL

0 5

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST TRENCH TT-5 EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 3e

REDUCED FROM 11X17



EXPLANATION OF TT-6

A: DARK BROWN SANDY SILT (ML), soft, dry with cobbles

D: MOTTLED RED-BROWN-LIGHT BROWN TUFFACEOUS SAND, medium dense, wet

E: LIGHT BROWN CLAY (CH), medium stiff, wet

Tsab: YELLOW-BROWN ANDESITIC TUFF BRECCIA, moderately hard to hard, moderately strong to strong

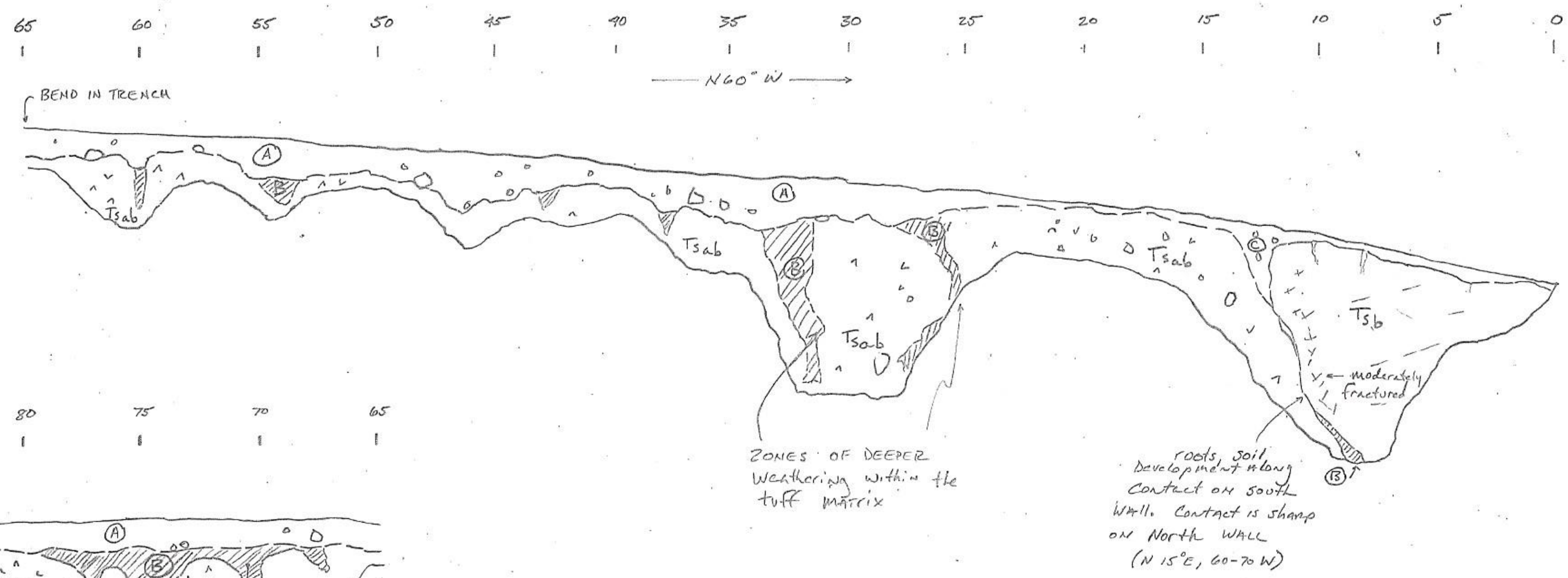
Tst: WHITE TO TAN TUFF, low hardness, friable, highly weathered

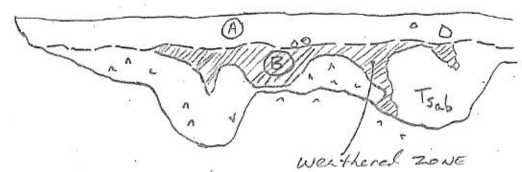
Scale: 1" = 5' Horizontal & Vertical

0 5

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST TRENCH TT-6 EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 3f

REDUCED FROM 11X17





EXPLANATION OF TT-7

A: RED-BROWN SANDY SILT (ML), soft, moist, with abundant roots and rock fragments

B: RED-BROWN SANDY SILT (MH), stiff, wet, weathered zones within the bedrock

Tsb: DARK GRAY ANDESITIC LAVA FLOW, moderately hard to hard, moderately strong to strong, massive, little fractured

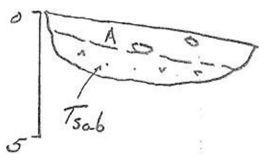
Tsab: GRAY ANDESITIC BRECCIA, moderately hard

SCALE: 1" = 5'
0 5

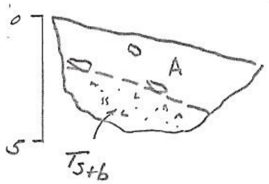
REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST TRENCH TT-7 EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 3g

REDUCED FROM 11X17

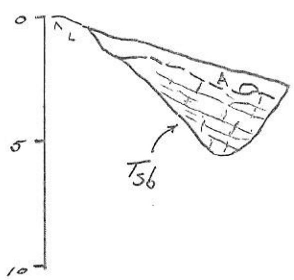
LOG OF TEST PIT #1



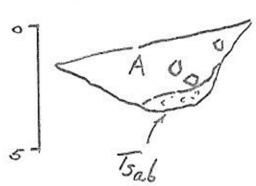
LOG OF TEST PIT #2



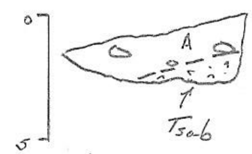
LOG OF TEST PIT #3



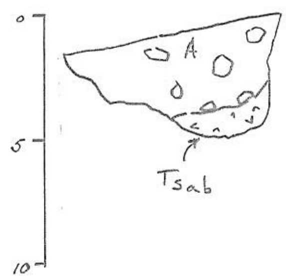
LOG OF TEST PIT #4



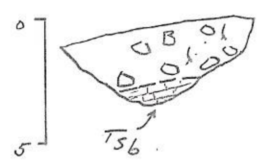
LOG OF TEST PIT #5



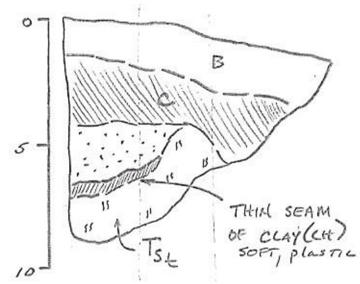
LOG OF TEST PIT #6



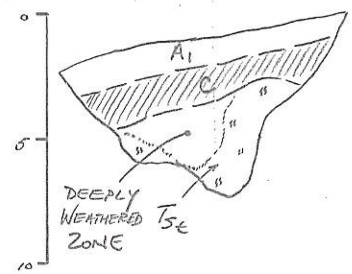
LOG OF TEST PIT #7



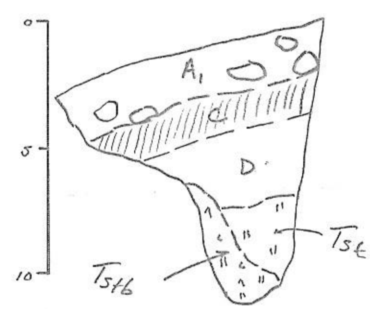
LOG OF TEST PIT #8



LOG OF TEST PIT #9



LOG OF TEST PIT #10



KEY

A: DARK BROWN SANDY SILT (ML), soft, dry, with occasional cobbles (topsoil)

A_1: RED-BROWN SANDY SILT (ML), soft, moist, with occasional cobbles (topsoil)

B: RED-BROWN SILTY GRAVEL (GM), medium dense, moist (topsoil)

C: DARK BROWN TO GRAY SANDY SILT (MH), stiff, wet

D: RED-BROWN SANDY SILT (MH), medium stiff, wet (residual soil from Tst)

Tsab: MOTTLED LIGHT BROWN-RED-LIGHT GRAY ANDESITIC FLOW BRECCIA, moderately hard to hard, moderately strong to strong

Tstb: MOTTLED LIGHT BROWN-BROWN ANDESITIC TUFF BRECCIA, moderately hard, moderately strong

Tsb: RED-GRAY ANDESITE, moderately hard to hard, moderately strong to strong, moderately fractured

Tst: LIGHT BROWN TO TAN, welded, low hardness, friable, highly weathered

SCALE: 1 inch = 5 feet

0 5

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST PITS 1 THROUGH 10 EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 4

REDUCED FROM 11X17

UNIFIED SOIL CLASSIFICATION SYSTEM

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

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

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

* Compressive Strength

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 8-29-16 Appr: BEP	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 5

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

1. U = unconsolidated
2. P = poorly consolidated
3. M = moderately consolidated
4. W = well consolidated

B: BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness (in feet)	Stratification
1. Massive	Greater than 4.0 ft	very thick bedded
2. Blocky	2.0 to 4.0 ft	thick bedded
3. Slabby	0.2 to 2.0 ft	thin bedded
4. Flaggy	0.05 to 0.2 ft	very thin bedded
5. Shaly or platy	0.01 to 0.05 ft	laminated
6. Papery	Less than 0.01 ft	thinly laminated

C: FRACTURING

Intensity	Size of Pieces (in feet)
1. Very little fractured	Greater than 4.0 ft
2. Occasionally fractured	1.0 to 4.0 ft
3. Moderately fractured	0.5 to 1.0 ft
4. Closely fractured	0.1 to 0.5 ft
5. Intensely fractured	0.05 to 0.1 ft
6. Crushed	Less than 0.05 ft

D: HARDNESS

1. **Soft** - Reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blow away.
4. **Hard** - can be scratched with difficulty; scratch produces little powder and is often faintly visible
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak

E: STRENGTH

1. **Plastic** - of very low strength.
2. **Friable** - Crumbles easily by rubbing with fingers.
3. **Weak** - An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

F: 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

1. **Deep** - Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or
2. **Moderate** - Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasional intense discoloration. Moderately coated fractures.
3. **Little** - No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
4. **Fresh** - Unaffected by weathering agents. No disintegration or discoloration.

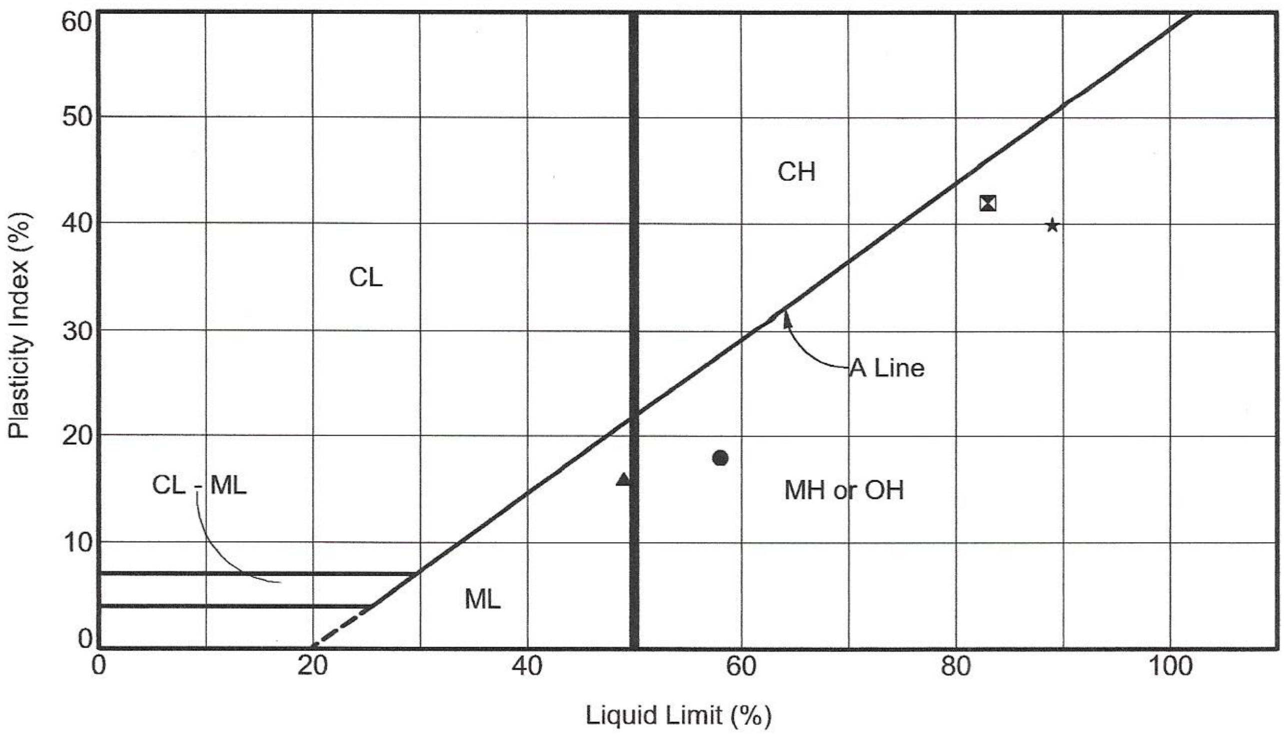
REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	PHYSICAL PROPERTIES FOR ROCK DESCRIPTIONS EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 6

TEST PIT/TRENCH	DEPTH	TEST TYPE*	TEST RESULTS
TT-1e	1.0	FS	45
	3.0	FS	45
	5.5	FS	115
TT-4	3.5	FS	40
TT-6	0.5	FS	45
	2.0	FS	95
TP=3	0.5	FS	30
	0.5	UC(P)	500
	1.0	UC(P)	4500+
	3.5	UC(P)	4500+
TP-4	0.5	FS	40
	0.5	UC(P)	500
	2.0	UC(P)	1000
	3.0	UC(P)	4500+
TP-5	0.5	FS	45
	0.5	UC(P)	500
	2.0	FS	40
	2.0	UC(P)	4500+
TP-6	1.0	FS	50
	4.0	FS	45
TP-7	1.0	FS	45

*Test Type

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

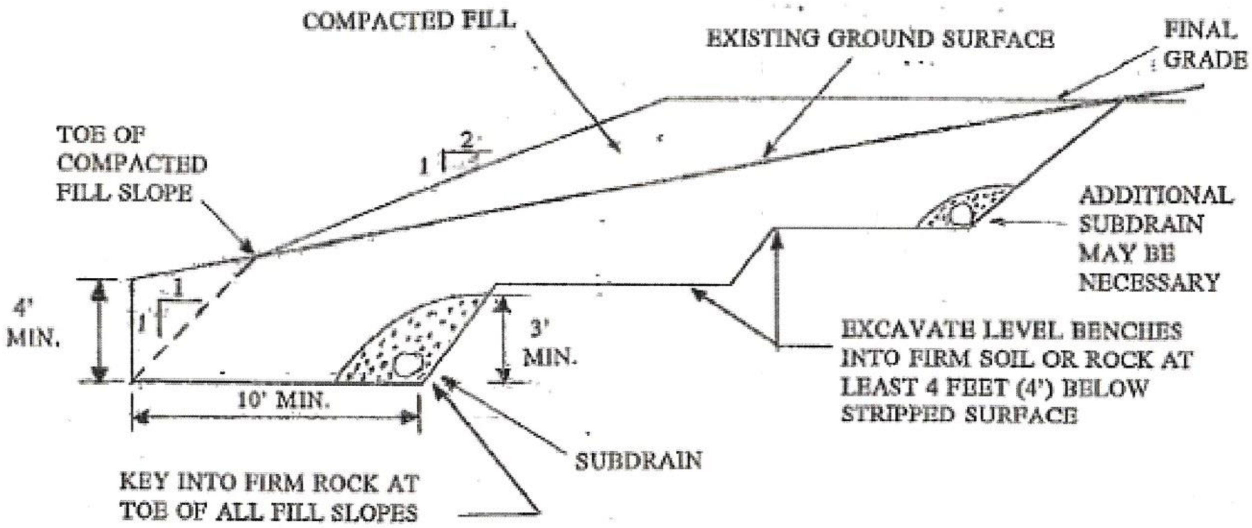
REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LABORATORY TEST DATA EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 7



ASTM D 4318-98

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)
●	**DARK RED-BROWN SILTY SANDY GRAVEL (GM)** **Test Pit 5 at 2.0 feet**	**58**	**40**	**18**	**40**
⧗	**DARK BROWN GRAVELLY SANDY SILT (MH)** **Test Trench 6 at 2.0 feet**	**83**	**41**	**42**	**--**
▲	**RED VERY GRAVELLY SANDY SILT (ML)** **Test Pit 7 at 1.0 feet**	**49**	**33**	**16**	**--**
★	**RED-BROWN GRAVELLY SANDY SILT (MH)** **Test Trench 1e at 5.5 feet**	**89**	**49**	**40**	**115**

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 8-29-16 Appr: BFP	ATTERBERG LIMITS TEST RESULTS	PLATE **8**



TYPICAL FILL SECTION
(NOT TO SCALE)

NOTES:

1. Dimensions shown are for estimating purposes. Actual dimensions and extent of keyways, benches, and subdrains will be determined in the field by the soil engineer.

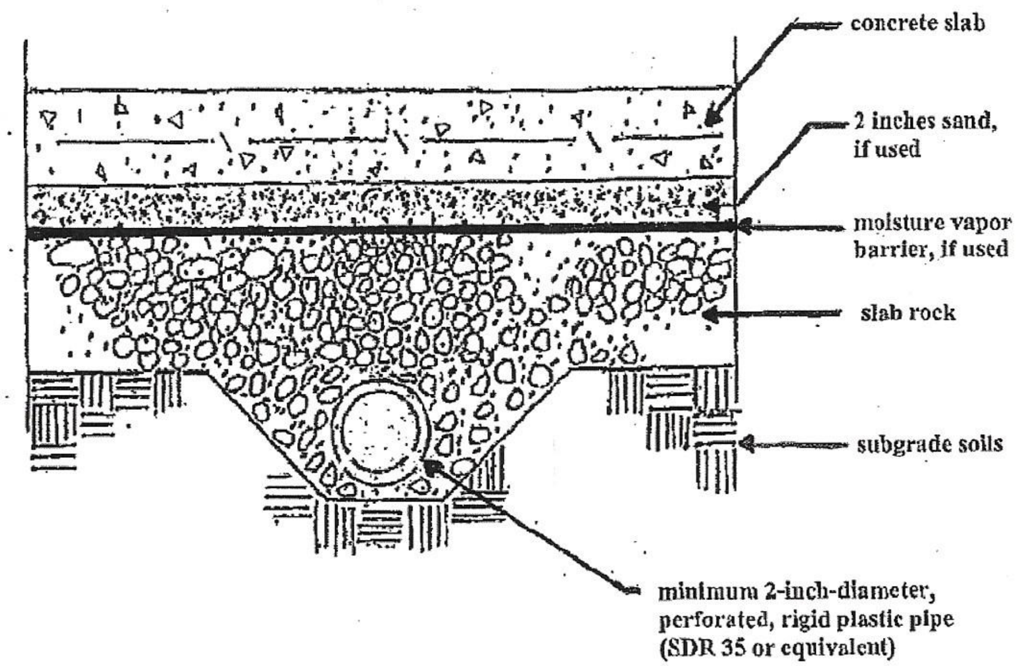
2. The upper 6 inches of soil exposed by excavation should be scarified, moisture conditioned and compacted to at least 90 percent relative compaction.

3. Fill should be placed in thin lifts and similarly compacted.

4. Slopes should be planted with deep-rooted vegetation (or protected by other suitable means) to reduce erosion.

5. Subdrains should consist of 4-inch-diameter perforated, rigid plastic pipe (SDR-35 or equivalent) with a gravity outlet and Class 2 Permeable material, or any drainrock encased in a nonwoven geotextile fabric weighing at least 4 ounces per square yard.

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BP	TYPICAL CROSS SECTION HILLSIDE GRADING EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 9



REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	**TYPICAL CROSS SECTION** **UNDERSLAB SUBDRAIN** **EMERALD ISLE SKILLED NURSING FACILITY** **SANTA ROSA, CALIFORNIA**	**PLATE** **10**

APPENDIX A

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

Report
Geologic Hazards Evaluation
Emerald Isle Skilled Nursing Facility
Santa Rosa, California

Prepared for
Oakmont Senior Living
9240 Old Redwood Highway, Suite 200
Windsor, CA 95492
Attention: Dave Hunter

By

REESE & ASSOCIATES
Consulting Geotechnical Engineers

Brian F. Piazza
Project Geologist

Co-Authored By - David H. Peterson
Certified Engineering Geologist No. 1186

Jeffrey K. Reese
Civil Engineer No. 47753

REGISTERED GEOLOGIST
DAVID H. PETERSON
No. 1186
CERTIFIED ENGINEERING GEOLOGIST
EXP 12-31-16
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JEFFREY KENT REESE
No. C 47753
EXP. 12-31-17
STATE OF CALIFORNIA

Job No. 202.5.1
September 20, 2016

This report summarizes the findings of the engineering geologic evaluation for the proposed Emerald Isle skilled nursing facility in the northern portion of the City of Santa Rosa, Sonoma County, California. The evaluation was performed concurrently with on-site geotechnical studies, to provide geologic input to project design, and to meet the requirements of the Office of Statewide Health Planning and Development (OSHPD). We understand that review of the engineering geologic evaluation will be performed by the California Geological Survey and this report has been formatted to generally follow the subject headings presented in CGS Note 48 (2011), *Checklist for Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings* (October 2013).

PROJECT LOCATION AND DESCRIPTION

Site Location

The facility is planned at the top of a knoll lying just east of Thomas Lake Harris Drive, as shown on the Location Map, Plate 1, adapted from the USGS Geological Survey, Santa Rosa 7.5-minute quadrangle. Currently, there is no street address for the project.

Site Coordinates

Latitude: 38.48867 N
Longitude: -122.71985 W

Project Description

The Site Plan and Geologic Map, showing the project, as well as current and prior test boring locations, is presented on Plate 2. The proposed development will consist of the construction of a one-story, wood-frame structure with a concrete slab-on-grade floor. The building will be served by asphalt-paved driveway and parking areas, and underground utilities. Site grading within the building and driveway/parking areas is anticipated to include cuts varying in depth up to about 16 feet, with fills in the access road area varying up to about 10 feet deep. Retaining walls may be needed as part of the site development.

PREVIOUS STUDIES

A soil investigation for the site was performed in 2006 by Giblin Associates of Santa Rosa, to evaluate the site (referred to at that time as The Oaks, Phase 3) for development as a 10-lot residential subdivision. The scope of that investigation included backhoe-excavated test pits throughout the site, at locations shown on Plate 2 herein. The investigation concluded the site could generally be developed as planned, but indicated that the area to the southwest and downslope of the current project (Phase 3, Lot 7) constituted a higher than normal risk of slope instability to upslope improvements because of steeper slope gradients. The report further judged it prudent to establish a 70-foot building setback zone from the top of the slope. If improvements were planned closer than 70 feet from the top of the steep slope, the report recommended a site-specific investigation and/or recommendations to reduce potential risks. As depicted in the Giblin Associates report, the planned Emerald Isle building footprint lies upslope and outside of the setback zone.

The site does not lie within an Earthquake Fault Zone (California Geological Survey, 1983), although the Zone established for the active Rodgers Creek fault lies just west of the property (see Plate 3). Based on prior fault trenching studies in the vicinity by Harding Lawson Associates (1980) and Moore and Taber (1985), the active traces of the fault mapped on the Earthquake Fault Zones map were identified as lying southwest and outside of the subject property. In an October 4, 2006 Assessment of Geologic Hazards report for the Fountaingrove Lodge project to the west, Giblin Associates reviewed available trenching data and indicated that sufficient previous work had been performed to conclude that active faults did not traverse that site. However, during preparation of the project Environmental Impact Report (EIR), the City's EIR consultant recommended additional evaluation of surface fault rupture hazard on that property, to evaluate possible discrepancies between the studies of Harding Lawson and Moore and Taber.

Giblin Associates subsequently performed additional fault rupture evaluation for the Fountaingrove Lodge site (previously called The Oaks, Phases 4 and 5), with project review provided by the City of Santa Rosa's geologic consultant. The Giblin investigation, summarized in a December 7, 2007 report, concluded that Holocene-active fault traces did not traverse the Fountaingrove Lodge site. However, during the course of Giblin's fault investigation, evidence of landslides and older slope movement were observed in the exploratory trenches. Based on their investigation, Giblin (2007) classified the slope movement type in three general categories:

> S1: Areas underlain by bedrock containing fractures and cracking from ancient bedrock movement that was probably seismically-induced. Radiocarbon dating of soils within and overlying the apparent fractures indicted these features represented pre-Holocene movement that occurred between 40,000 and 80,000 years ago;

S2: An area at the north end of the Fountaingrove Lodge project, underlain by landslide deposits and colluvium of intermediate age (middle to late Holocene); with evidence for movement in the past 5,000 years;

S3: Areas of west-facing slopes in the central project area, underlain by shallow landslide deposits showing evidence of recent minor movement.

A geologic and geotechnical investigation for the Fountaingrove Lodge was performed in 2011 by Reese & Associates. The investigation verified and further characterized the extent of the landslides previously identified in the 2007 fault study by Giblin Associates. In general, the S2 landslides appeared to be primarily older debris slides/flows with source areas north and outside of the subject property. Geologic mapping of the site vicinity indicates the debris slides lie downslope and outside of the Emerald Isle property. The younger S3 deposits were also limited to the lower, western portions of Fountaingrove Lodge property and were subsequently removed during site grading for that project.

The ancient, deep-seated, bedrock landslide (S1) underlying the Lodge property was: found to extend to depths ranging from about 50 to 60 feet; generally bounded at the base by a shear plane in claystone or weathered rhyolite pyroclastics; and, often associated with a thin lignite layer. The upslope extent of this zone of older bedrock movement was not verified during the 2011 investigation by Reese Associates, but was assumed to extend upslope and off of the Fountaingrove Lodge site. An approximate upslope limit was estimated, based on geomorphic features on aerial photographs, LiDAR imagery, and field reconnaissance. However, since dating of the S1 feature indicated at least 40,000 years had elapsed from the last movement, it was considered possible that well-defined geomorphic features associated with this older feature might not be preserved. Based on limited geomorphic evidence, the Reese & Associates report extended the limit of the S1 feature (shown as queried) to the steep, bowl-shaped slope southwest of the Emerald Isle facility (see Plate 2). Consequently, the current study included subsurface investigation to evaluate if possible secondary effects of older, S1 bedrock movement (weak, open, or fractured bedrock) extended up to the current project site.

ENGINEERING GEOLOGY/SITE CHARACTERIZATION

Regional Geology and Regional Fault Mapping

Published mapping reviewed included regional geologic maps compiled by Fox and others (1973) and Huffman and Armstrong (1980). The most recent published geologic map of the vicinity was the Santa Rosa 7.5-Minute Quadrangle, prepared by the U.S. Geological Survey (McLaughlin and others, 2008).

Vicinity Geology

The geologic setting of the site and vicinity, adapted from the mapping by McLaughlin and others (2008), is shown on Plate 4. While some discrepancies were noted in the earlier geologic maps and McLaughlin and others (2008), primarily with regard to mapping of active and inactive fault traces, the published maps show the site underlain by units of the late Tertiary-age Sonoma Volcanics. At the site, the Sonoma Volcanics are described as consisting of andesite, basaltic andesite, and basalt lava flows, flow breccia, and tuff breccia, with local water-lain andesite tuff and dacite ash-flow tuff (Map symbol *Tsb* on Plate 4, from McLaughlin and others, 2008).

Southwest of the property, in contact along the northwest-trending Rodgers Creek fault, are strata of the late Tertiary Petaluma formation (map symbol Tp). The Petaluma formation is described as consisting dominantly of sandy to silty gravel, silty sandstone, siltstone, and mudstone. Where mapped in depositional contact in the vicinity, the Petaluma formation underlies the Sonoma Volcanics.

Locally, McLaughlin and others (2008) map areas where the bedrock units are blanketed by alluvium (map symbol Qal). As mapped, these deposits appear to include alluvial deposits filling depressions along older faults, as well as depressions in the Sonoma Volcanics bedrock. Previous site investigations on the Fountaingrove Lodge property to the west and Canyon Oaks project to the northwest indicate that the deposits shown as alluvium may also include older, eroded remnants of debris slides and colluvium.

The published geologic maps reviewed, including the most recent mapping by McLaughlin and others (2008) and the *Landslides and Relative Slope Stability* map of Huffman and Armstrong (CDMG; 1980) do not show landslides adjacent to or underlying the subject property, although small landslides are mapped locally in the vicinity. The zone of older "S1" slope movements identified in the Fountaingrove Lodge study is not shown on the published maps. As classified on the map by Huffman and Armstrong (1980), the southeastern half of the site falls into Stability Category Bf, "locally level areas within hilly terrain, may be underlain or bounded by unstable or potentially unstable rock materials." The northwestern portion of the property is classified as Stability Category C, "Areas of relatively unstable rock and soil units, on slopes greater than 15 percent, containing abundant landslides."

Site Geology

The site geology is depicted on Plate 2. At the site, geologic mapping and subsurface exploration identified four volcanic units within the general area shown by McLaughlin and others (2008) as map symbol Tsb (andesite, basaltic andesite, and basalt). These units consist of: 1) gray porphyritic andesite lava flows that are hard, moderately fractured, and locally display platy jointing (symbol Tsb on map and trench logs); 2) gray and yellow brown andesitic tuff breccia with an ash/crystalline matrix that is generally massive and compact (symbol Tstb); 3) andesite flow breccia that is generally massive, little fractured, and hard near the ground surface (limiting backhoe excavation; symbol Tsab); and 4) light brown dacitic to rhyolitic tuff that is little fractured, massive, varying from friable to moderately hard (symbol Tst). Physical properties for rock description is shown on Plate 5.

Downslope and west of the planned Emerald Isle facility, the prior soil investigation by Giblin Associates (2006) encountered colluvial soil deposits consisting of sandy and gravelly clay containing cobble to boulder sized volcanic rock fragments. Where encountered in the test pits by Giblin Associates, the colluvium extended up to about 12 feet thick. The limits of the mapped colluvium, adapted from the prior study by Giblin Associates, is shown on Plate 2.

Subsurface Geology/Geologic Cross Sections

The logs of trenches TT-1 to TT-7 are shown on Plates 6a to 6g. Additionally, our interpretation of the subsurface conditions beneath the site is shown on the Geologic Cross Sections on Plates 7a and 7b. The trenches were generally configured to extend outward and downslope from the edge of the proposed building envelope, to evaluate the general bedrock conditions, as well as look for evidence of slope movement effects in bedrock. The conditions encountered in the trenches are summarized below:

TT-1 and TT-1 Extension Extending west and downslope from the planned central courtyard, the trench encountered andesite lava flows (Tsa) at the upslope (eastern) end that are closely to moderately fractured, hard and little weathered within a few feet of the ground surface. The lava flows are in depositional contact with massive, little to moderately fractured andesite tuff breccia (Tsab). Extension of the trench to the northeast revealed that the andesite lava flow was limited in extent and appeared to be deposited in a depression in the underlying tuff breccia. The andesite tuff breccia unit was noted to contain numerous zones of weathering and clay development near the ground surface, that were found to generally die out at depths of about 8 to 9 feet. Along the base of the trench, compact, massive tuff breccia was encountered.

TT-2 This trench was excavated at the southwest corner of the planned building and was extended as far as feasible with the backhoe equipment (total length of about 55 feet). The trench encountered andesite flows at the upslope end, in a depositional contact with andesite tuff breccia along a steeply west dipping (about 55 degrees) contact. Further downslope, little to moderately fractured andesite tuff breccia was encountered, that appeared massive and undisturbed throughout the trench.

TT-3 This trench encountered andesite lava flows overlying tuff breccia along a gently west-dipping depositional contact. The bedrock units were noted to be hard and moderately to little fractured. Near the upslope, east end of the trench, a near-vertical soil-filled fracture was encountered that could be traced to the bottom of the trench at 10 feet (the maximum depth excavatable by the backhoe). No offsets of bedrock units were observed across this feature. The fracture generally corresponds to a break in slope that steepens to the west and the soil-filled feature may be a zone of deeper soil infilling and development along a fracture.

TT-4 This trench was excavated to verify conditions on the slopes east of the building, where test pits by Giblin Associates had previously encountered pyroclastic (tuff) units. The trench encountered light brown massive, little fractured tuff (Tst) overlain by massive, moderately hard tuff breccia along a west-dipping depositional contact. The bedrock units were massive and did not appear disrupted by slope movement.

TT-5 Excavated north of trench TT-3 to further evaluate the conditions from the building footprint, out to the break in slope to the west. The trench encountered andesitic tuff breccia throughout most of the trench that was variably weathered and containing clayey zones to depths of about 6 to 8 feet. In the downslope, northern end of the trench, zones of soil-filled fractures and weathering in bedrock were encountered to the bottom of the trench. In addition, a zone of andesite flows were encountered over a length of about 11 to 12 feet in the trench wall that contained zones of soil filled fractures with rounded pebbles and cobbles that may have been infillings from shallower depth. However, no evidence of shearing or vertically displaced units was evident.

TT-6 Test Trench 6 was excavated along the proposed new access roadway. The trench encountered andesite breccia (Tsab). Extension of the trench to the southwest the andesite breccia transitioned to a little fractured, welded tuff (Tst). The soil observed between the topsoil and welded tuff breccia thicken downslope and consists of a tuffaceous sand. The tuffaceous sand was observed to be underlain by a thin relatively discontinuous layer of plastic clay to depths of about 7 feet. Further downslope the bedrock materials transitioned back to the andesite breccia. Transitions between the Tsab and Tst appeared depositional. Further, no slickensides were observed at the contact between the tuffaceous sand, clay and underlying andesite breccia. It therefore appears that the soils overlying bedrock in the trench consist of colluvium, without obvious slope movement.

Because of the downslope orientation of the clay layer between the tuffaceous sand and andesite breccia, test pit 8 was excavated for observation of the clay layer. The clay layer was observed to "pinch-out" further to the southwest. We judge the tuffaceous sand is relatively limited in extent and appeared to be deposited in a relatively low depression on the andesite breccia.

TT -7 Excavated upslope and overlapping the upslope portion of Trench TT-3 to further evaluate the soil filled fractures found in TT-3 and TT-5 and to extend subsurface information upslope into the building area. The downslope end of TT-7 encountered dark gray, hard and moderately fractured andesite flow rock. The andesite flows were in apparent depositional contact with hard, little fractured andesite breccia along a steeply dipping (60-70 degrees), west-dipping contact. Although roots and soil development were noted along the contact on the south side of the trench well, the same contact was sharp, with no soil development near the base on the north side. The weathered, soil-filled fractures found in the downslope portion of Trench TT-5 project along strike to this depositional contact, suggesting that the features seen in TT -5 are likely the result of weathering along steeply dipping volcanic units. Further upslope in TT-7, the andesite breccia contains zones of weathering beneath the gravelly clay surface soils that typically extended to depths up to about 8 feet. The andesite breccia was massive, moderately hard and generally difficult to excavate deeper than about 5 feet.

Summary of Trench Observations

Based on the observations in the exploratory trenches, the building area is underlain at shallow depth by moderately to little fractured andesite flows, breccia and tuff breccia that contain local zones of weathering and clay development to depths of about 8 feet. Within the western and southern parts of the building area, bedrock units are compact and do not appear to be affected by slope movement. As discussed, the building footprint lies upslope and outside of a building setback zone recommended in a prior soil investigation by Giblin Associates in 2006. Trench TT-2 also verifies that compact, little to moderately fractured volcanic rock extends downslope from the building envelope for the length of the trench, a distance of about 60 feet.

Further north, in Trenches TT-3 and TT-7, soil-filled fractures were encountered at the contact between andesite lava flows and breccia, at a transition to steeper slopes that appear to represent weathering along fractures and steeply dipping volcanic units. Evidence of down-dropped/displaced bedrock units or slickensided shear planes, which might suggest older landsliding, were not observed. Beneath the building envelope, bedrock units were noted to be compact, relatively little fractured and not disrupted by slope movement.

Active Faulting & Coseismic Deformation Across Site

In active faults have been mapped in the vicinity (McLaughlin, 2008). However, active faults (those experiencing surface rupture or seismic activity within the past 11,000 years) are not known to traverse the site. Review of prior fault studies for the adjacent Fountaingrove Lodge project, the Santa Rosa General Plan 2035, published fault and geologic maps (Wagner and Bortugno, 1982), and Alquist-Priolo Earthquake Fault Zones Maps (CDMG 1983) indicate that the nearest active fault is the Rodgers Creek. The potential for surface fault rupture from the Rodgers Creek fault was studied extensively by Giblin Associates on the adjacent property to the west and was summarized in their 2007 report. As previously discussed, active traces of the Rodgers Creek fault do not traverse the subject property. Based on the mapped trace presented on the Earthquake Fault Zones map, the nearest trace of the fault is about 0.46 kilometers to the southwest. A copy of a portion of the Earthquake Fault Zones Map, showing the site in relation to the Rodgers Creek Fault, is shown on Plate 3.

Recent studies in the Santa Rosa Creek floodplain beneath downtown Santa Rosa (Hecker and others, 2016) revealed a zone of pull-apart basins along the Rodgers Creek fault, indicative of deformation in alluvial deposits, varying in width up to about 375 meters. Further north, studies by Funning and others (2007) suggest that the northern portion of the Rodgers

Creek fault may be undergoing aseismic creep of up to about 6 millimeters per year. While regional in the mapping scale, it appears that creep effects shown in Funning and others (2007) are generally localized near the fault. While there is limited data to conclude the risk of coseismic deformation during future earthquakes, the site is located on bedrock and about 0.46 kilometers from the nearest identified Holocene traces of the Rodgers Creek fault. Based on the limited available data, it appears that the potential for coseismic ground deformations to occur in bedrock underlying the site is relatively low.

Geologic Hazards Zones (Liquefaction and Landslides)

A Seismic Hazards Zones map, associated with the Seismic Hazards Mapping Act (Ch. 7.8, Div. 2 of California Public Resources Code) has not been prepared for the Santa Rosa 7.5-minute quadrangle. Review of Noise and Safety Element (figure 12-3) of the Santa Rosa General Plan 2035 identifies the site as lying within areas of "Violent Groundshaking During an Earthquake on the Rodgers Creek Fault," and "Areas of Relatively Unstable Rock on Slopes greater than 15 percent." The risk of seismically-induced liquefaction is not shown in the General Plan, although published mapping by the USGS (Witter and others, 2006) indicate the site, underlain by bedrock, does not lie in an area of liquefaction potential.

Geotechnical Testing/Consideration of Geology in Geotechnical Engineering Recommendations

A geotechnical site investigation is currently being performed by Reese and Associates that includes additional subsurface investigation, laboratory testing, and geotechnical engineering analysis to develop recommendations for site design and construction. The engineering geologic and geotechnical investigations were performed in collaboration with the undersigned Engineering Geologist and Reese & Associates Engineers. Site geologic conditions and conclusions from the engineering geologic study have been incorporated into the engineering recommendations.

SEISMOLOGY & CALCULATION OF EARTHQUAKE GROUND MOTION

Evaluation of Historical Seismicity

Rodgers Creek Fault - The Rodgers Creek fault has been mapped as a discontinuous zone of sub-parallel breaks extending from San Pablo Bay, north to Geyserville. The Rodgers Creek fault (and associated Healdsburg fault) has been the source of several historic earthquakes, including magnitude 5.6 and 5.7 events in 1969. Ground surface rupture was not confirmed, although ground cracking was observed in areas of young creek alluvium, possibly the result of liquefaction (Youd and Hoose, 1979). Ground shaking was widely felt throughout Sonoma County, although damage was mainly to older buildings in downtown Santa Rosa. Based on a return interval of 222 years, the probability of a magnitude 6.7 or larger earthquake occurring on the Rodgers Creek fault in the next 30 years has been estimated by the U.S. Geological Survey at 32 percent (Michael and others, 1999), one of the highest probabilities for Bay Area faults.

Maacama Fault - The Maacama is considered to be the northernmost segment in a system of northwest-striking faults that include the Calaveras and Hayward faults in the southeastern San Franciscan Bay Area and the Healdsburg-Rodgers Creek faults extending north to Healdsburg in Sonoma County. The Maacama fault is generally considered to consist of three subparallel segments that extend a total distance of about 95 miles. Portions of the zone display surface topographic or geologic evidence indicative of an active fault, while other portions do not appear to have experienced activity during the Holocene epoch (past 11,000 years). Available data about the southern segment of the Maacama fault, that is about 6 miles to the north of the Emerald Isle site, indicates that the recurrence interval for the estimated maximum moment magnitude earthquake of 6.9 is about 220 years (Peterson and others, 1996).

San Andreas Fault - While the San Andreas fault is about 20 miles (33 km.) southwest of the property, it was responsible for the largest historic earthquake in northern California. This earthquake occurred in 1906, had an estimated Richter magnitude of 7.9, and caused damage and strong ground shaking throughout northern California. The intensity of ground shaking from the 1906 earthquake in the Santa Rosa area was estimated at up to about VIII to X on the Rossi-Forel intensity scale and approximately equivalent to the Modified Mercalli Intensity Scale (Lawson, 1908). Recent studies have estimated a 21 percent probability of a large (magnitude 6.7 or greater) earthquake on the San Andreas fault in the next 30 years (Michael and others, 1999).

Historic Seismicity - A search of earthquake records from the period of 1800 to 2007 was performed using the program EQSEARCH (Blake, 1993, v.2.01) for the years 1800 to 1993 and information from the Northern California Earthquake Data Center for 1994 to 2016. Our search found 211 seismic events in the range of magnitude 4.0 to 9.0 within a 60 mile (100 km.) radius of the site. Using the magnitude-distance attenuation relationships developed by Campbell (1993), EQSEARCH calculates that the maximum ground acceleration at the site from the period of 1800 to 1993 was about 0.30 gravity (g), associated with a magnitude 5.7 earthquake on October 2, 1969, centered in the Santa Rosa area, about 3 miles (4 km.) from the site.

Classification of Geologic Subgrade (Site Class)

Review of the previous soil investigation data by Giblin Associates and the observations of the current investigation indicate the site is underlain at shallow depth by volcanic bedrock and that characterization of the geologic subgrade using Standard Penetration Test methods (i.e., sample blow counts) was not a feasible method. To the depth explored, the site subgrade conditions consist of bedrock units of the Sonoma Volcanics, consisting primarily of volcanic lava flows and flow breccia, with lesser tuff and tuff breccia that are judged to be competent, with moderate fracturing and weathering. Review of published values of shear wave (Vs) velocities for volcanic rocks, in general, yielded a broad range of values, from about 1,600 to 1,800 m/sec for andesite (Brocher, 2005), and up to about 2,800 to 3,400 m/sec (Mavko, Undated, Stanford Univ. Rock Physics Laboratory), and 3,200 to 3,600 m/s (Dobrin, 1976) for basalt. In general, the velocity ranges by Brocher (2005) were lower than other references reviewed.

Based on the compact, moderately fractured and weathered character of the near-surface bedrock (to a depth of only about 10 feet) and range of published shear wave values, it appears that the site meets the criteria of Site Class B, to a depth of 100 feet (30 meters).

General Procedure Ground Motion Analysis

A Site Class B subgrade (i.e., moderately weathered and fractured rock) and the site latitude and longitude were used as input to the U.S. Geological Survey's Design Maps program (2015) to develop the following design parameters:

Mapped Spectral Acceleration Parameters (g) (Site Class B)		Site Coefficients (Site Class B)		Spectral Response Acceleration Parameters (Site Class D) (g)		Design Spectral Acceleration Parameters (Site Class D) (g)	
SS	S1	Fa	Fv	SMS	SM1	SDS	SD1
2.461	1.023	1.0	1.0	2.461	1.023	1.641	0.682

Seismic Design Category

As indicated in the above table, the mapped spectral response acceleration parameter at 1-second period S1 is greater than 0.75. In reviewing the Occupancy Category of Buildings and Other Structures, Table 1604A.5 of CBC, the planned occupancy appears consistent with Occupancy Category I-2. Therefore, Seismic Design Category E appears appropriate. For Seismic Design Categories D through F, the Mapped Maximum Considered Geometric Mean (MCEG) Peak Ground Acceleration (PGA) derived from the USGS site is 0.952 g, which from ASCE 7 Table 11-8.1, yields a Site Coefficient of $F_{PGA} = 1.0$.

Site-Specific Ground Motion Analysis

A site-specific ground motion analysis was performed by Miller Pacific Engineering Group of Petaluma, California, and is summarized in their June 9, 2016 report. A copy of that report is attached to the geotechnical report as an appendix.

Seismic Source Parameters

A search of faults with Holocene activity was performed using the program EQFAULT (1993) and published geologic mapping (CDMG, 1983; Wagner and Bortugno, 1982; Jennings, 1994, McLaughlin and others, 2008). Of the identified active faults, those nearest the property are summarized in the following table:

Fault System	Distance from Site (Kilometers)	Direction from Site	Slip Rate (mm/yr)	Recurrence Interval (yrs)	Maximum Moment Magnitude	Peak Ground Acceleration (g)
Rodgers Creek	0.46-0.50*	Southwest	9	222	7.0	0.58
Maacama (South)	10	North	9	220	6.9	0.27
West Napa	32	Southeast	1	701	6.5	0.06
San Andreas (1906)	33	Southwest	24	210	7.9	0.15

*Range reflects distances to individual fault traces

Earthquake magnitudes are expressed in terms of the moment magnitude scale (Mw) and were obtained from Tables of California Fault Parameters in Peterson and others (1996) and Cao and others (2003). Peak ground accelerations (PGA) are estimated for the Maximum Considered Earthquake (MCE), using older generation attenuation relationships developed by Campbell (1993) and for a site underlain by bedrock. The PGAs given in the table above are not intended to be seismic design criteria, but rather, are shown to indicate the relative potential shaking effects from the various identified source faults.

Liquefaction/Seismic Settlement Analysis

As discussed, the site is underlain at the surface by bedrock units of the Sonoma Volcanics. Additionally, the site is not shown in an area of seismic liquefaction potential (Witter and others, 2006). Therefore, no further analysis of liquefaction potential was deemed necessary.

Slope Stability Analysis

As located, the planned building will be situated on a graded building pad on the relatively flat crest of a volcanic knoll. Our site investigation did not identify landslides or unstable slopes beneath or adjacent to the planned building footprint. Therefore, a slope stability analysis for the building area was not performed. Trench TT-6, excavated along the planned access driveway, encountered colluvial soil deposits that may be susceptible to settlement or slope creep, and are proposed to be upgraded by removal and replacement as a compacted, engineered fill. Specific recommendations for site preparation and grading are presented in the geotechnical investigation report by Reese & Associates.

OTHER GEOLOGIC HAZARDS OF ADVERSE SITE CONDITIONS

Corrosive/Reactive Geochemistry of Geologic Subgrade

Samples of the near-surface soils and weathered bedrock material were collected during the site investigation and submitted to Environmental Technical Services (ETS) of Petaluma, California for analysis of potential corrosive or reactive geochemistry. The laboratory report, with data interpretation by ETS, is attached to the geotechnical investigation report as an appendix.

Conditional Geologic Assessment

Of the items listed in Note 48, none were deemed a potential hazard to the site. The site does not lie with a 100-year flood zone (Item C). Clays susceptible to seismic softening (Item I) do not underlie the site and are not considered to pose a hazard at the site.

CLOSURE

We trust this provides the information needed by CGS to perform their project review. If you have questions regarding this report or require additional information, please contact us.

LIST OF PLATES

References

Aerial Photos and Remote Imagery:

- June, 10, 1961, CSH-6BB 106, 107; black and white, scale: 1/67,500
- May 18, 1971, Cartwright Aerial Surveys, 3088-194, -195, scale: 1/24,000
- March 13, 1973, Geo-graphics Aerial Survey, SR H-19, -20, scale (approx.): 1/6000
- May 7, 1980, Sonoma Co. Assessor, BW-SON-18-18, -19, -20, scale: 1/24,000
- May 19, 2000, Sonoma Co. Assessor, 50-013734, 48-22, -23, -24; scale (approx): 1/10,500
- Google Earth, 2016, (http://earth.google.com); includes historic aerial photos from 1993 to 2015.
- OpenTopography.org, 2010, Lidar Imagery of Northern California Fault Zones, viewed as Google Earth kml files.

Blake, T.F., Computer Services and Software:

- EQFAULT, v. 2.01, 1993, A computer program for the deterministic prediction of peak horizontal acceleration from digitized faults.
- EQSEARCH, v. 2.01, 1993, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs.

Brocher, T.M., 2005, *Compressional and Shear Wave Velocity Versus Depth in the San Francisco Bay Area, California: Rules for USGS Bay Area Velocity Model 05.0.0*: U.S. Geological Survey Open File Report 05-1317, 58p., with figures.

California Division of Mines and Geology, 1983, *Special Studies Zones, Santa Rosa Quadrangle, Revised Official Map Effective July 1, 1983*: Scale 1:24,000 (subsequently renamed "Earthquake Fault Zones").

California Division of Mines and Geology, 2011, *Checklists for the Review of Geologic/Seismic Reports for California Public Schools, Hospitals, and Essential Services Buildings, California:* Division of Mines and Geology Note 48, 2p, revised October 2013.

Campbell, K.W., 1993, *Empirical Prediction of Near-Source Ground Motion from Large Earthquakes*: Proceedings of the International Workshop on Earthquake Hazard and Large Dams in the Himalaya, Sponsored by the Indian National Trust for Art and Cultural Heritage (INTACH), New Delhi, India.

Cao, T., Bryant, W.A., Rawshandel, B., Branum, D., and Wills, C.J., 2003, *The Revised 2002 California Probabilistic Seismic Hazards Maps, Appendix A - 2002 California Fault Parameters*: California Geological Survey Online publication: http://www.conservation.ca.gov/CGS/rghm/psha/Pages/Index.aspx

City of Santa Rosa, 2009, *General Plan 2035: Noise and Safety Element*: Section 12-4, Geology and Seismicity, Figure 12-3, November 3, 2009.

Dobrin, M.B., 1976, *Introduction to Geophysical Prospecting*, 3rd Edition: McGraw Hill Book Company, New Your.

Fox, K. F., and others, 1973, *Preliminary Geologic Map of Eastern Sonoma county and Western Napa county, California*, U.S. Geological Survey Map MF-483

Funning, G.J., Burgmann, R., Ferretti, A., Novali, F., and Fumagalli, A., 2007, *Creep on the Rodgers Creek fault, northern San Francisco Bay Area from a 10 Year PS-InSAR Dataset*: Geophys. Res. Lett., v. 34, L19306, 5p.

Giblin Associates, 2006, *Report, Geotechnical Investigation, The Oaks, Phases 3,4, and 5, Santa Rosa, California*: Unpublished report prepared for Aegis Senior Living, March 31, 2006.

Giblin Associates, 2007, *Investigation of Fault Surface Rupture Hazard, Fountaingrove Lodge, Santa Rosa, California, Volumes 1 and 2,* Job No. 3221.3.1-1, prepared for Aegis Senior Living, December 7, 2007.

Harding-Lawson Associates, May 12, 1980, *Geologic Evaluation for EIR, TMI Property, Fountain Grove Ranch, Santa Rosa, California*, Job No. 5179,001.02, prepared for Teachers Management Investment Corporation.

Hecker, S., Langenheim, V.E., Williams, R.A., Hitchcock, C.S., and DeLong, S.B., 2016, *Detailed Mapping and Rupture Implications of the 1 km Releasing Bend in the Rodgers Creek Fault at Santa Rosa, Northern California*: Bulletin of the Seismological Society of America, V. 106, No. 2, April 2016.

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

Jennings, Charles W., 1988, *Preliminary Geologic Map of the Northwest Quarter of the Santa Rosa 7.5' Quadrangle, Sonoma County, California*, DMG Open File Report 88-5.

Jennings, C.W., 1994, *Fault Activity Map of California and Adjacent Areas, with Locations and Ages of Recent Volcanic Eruptions*: California Division of Mines and Geology, Geologic Data Map No. 6, Scale 1:750,000.

Mavko, G., undated, *Overview of Rock and Fluid Factors that Impact Seismic Velocity and Impedence*: Stanford University, Rock Physics Laboratory, available at https://pangea.stanford.edu/courses/gp262/Notes/8.SeismicVelocity.pdf

McLaughlin, R.J., Langenheim, V.E., Sarna-Wojcicki, A.M., Fleck, R.J., McPhee, D.K., Roberts, C.W., McCabe, C.A., and Wan, E, 2008, *Geologic and Geophysical Framework of the Santa Rosa 7.5' Quadrangle, Sonoma County, California*: U.S. Geological Survey Open File Report 2008-1009, Scale 1:24,000.

Michael, A.J., Ross, S.L., Schwartz, D.P., Hendley, J.W. II, and Stauffer, P.H., 1999, *Major Quake Likely to Strike Between 2000 and 2030 - Understanding Earthquakes in the San Francisco Bay Region*: U.S. Geological Survey Fact Sheet 152-99, 4p.

Moore & Taber, January 7, 1985, *Geotechnical Study, "Part I"– Faulting Evaluation, Phase III Cluster Residential ("North of Stonefield") Fountaingrove Ranch, Santa Rosa, California,* Job No. 3P3/583/102, prepared for Sonoma Financial Corporation

Peterson, M. and Frankel, A.D., 1996, *Probabilistic Seismic Hazard Assessment for the State of California*: California Division of Mines and Geology Open File Report OFR 96-08 (table of California Fault Parameters).

Reese & Associates, 2011, *Report, Geotechnical Investigation, Fountaingrove Lodge, Landslide Hazard Evaluation, Thomas Lake Harris Drive, Santa Rosa, California*: unpublished consultants report to Oakmont Senior Living, 48p, with illustrations.

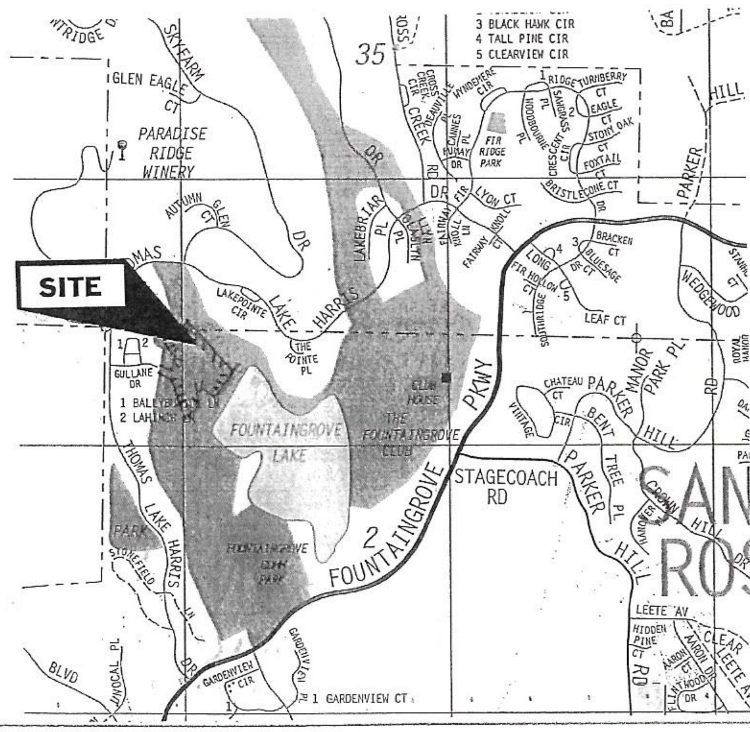
U.S. Geological Survey, Earthquake Hazards Program, 2016, *U.S. Seismic Design Maps*: online seismic ground motion calculator; http://earthquake.usgs.gov/designmaps/us/application.php

Wagner, D.L. and Bortugno, E.J., 1982, *Geologic Map of the Santa Rosa Quadrangle, California*: California Division of Mines and Geology Regional Geologic Map Series, Map No. 2A (Geology), Scale 1:250,000.

Witter, R.C, Knudson, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., and Randolph, C.E., 2006, *Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California*: U.S. Geological Survey Open File Report 2006-1-37, 2 sheets, Scale 1:200,000.

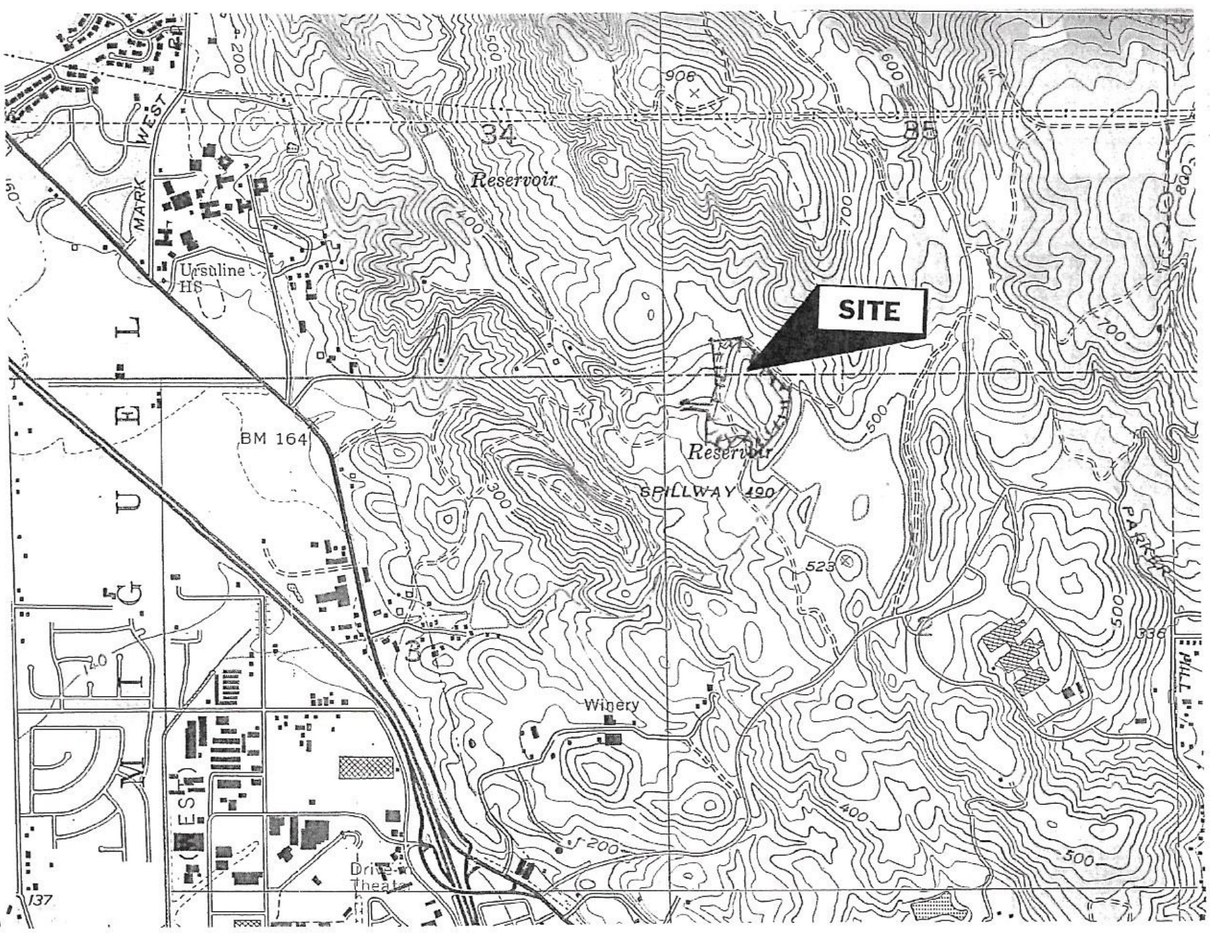
Working Group on California Earthquake probabilities, 2002, *Earthquake Probabilities in the San Francisco Bay Region: 2002-2031*, U.S. Geological Survey Open-File Report 03-214.

Youd, T.L. and Hoose, S.N., 1979, *Historic Ground Failures in Northern California Triggered by Earthquakes*: U.S. Geological Survey Professional Paper 993.



N

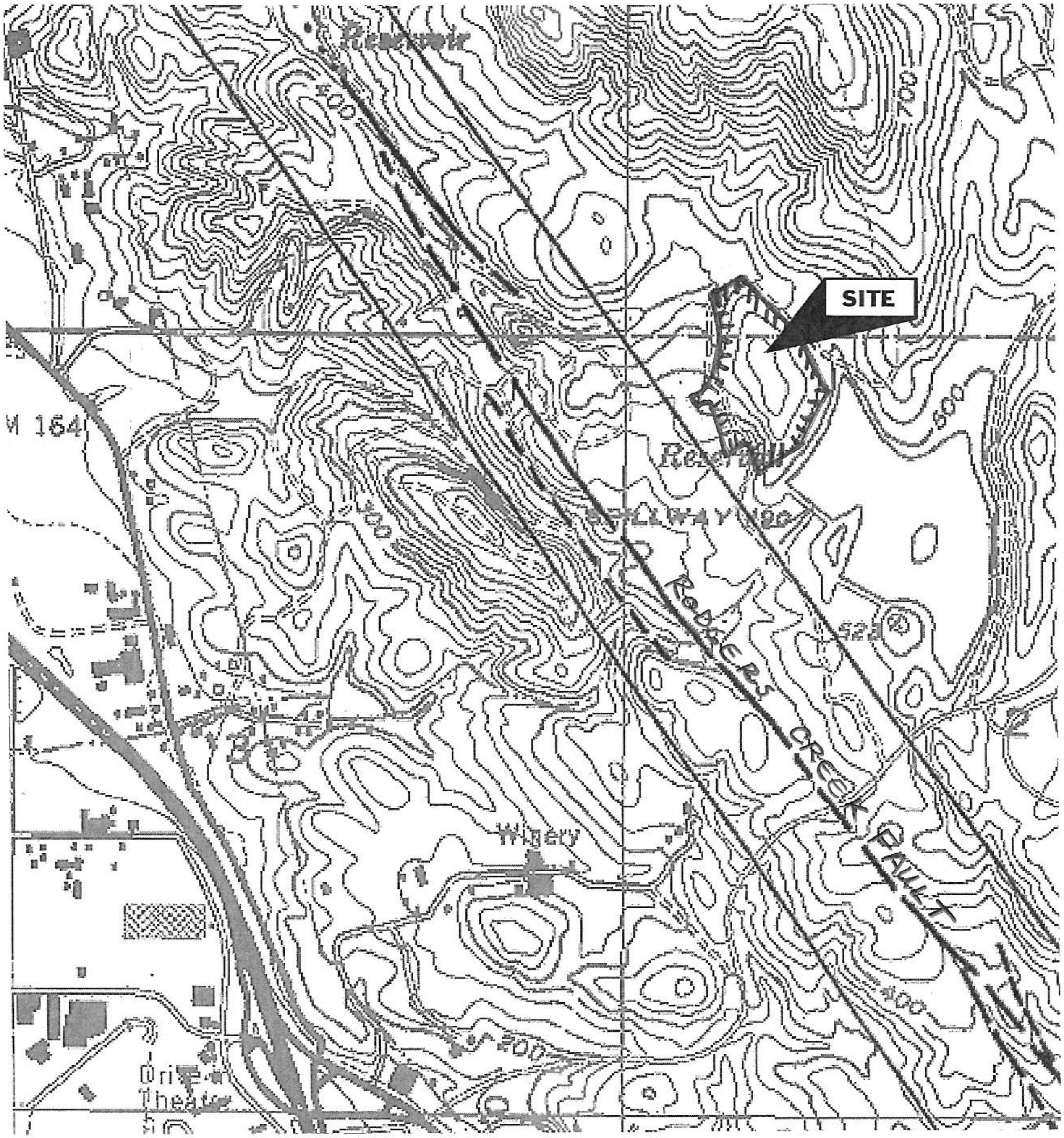
SCALE: 1" = 1900'



N

SCALE: 1" = 1900'

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	VICINITY MAP EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 1



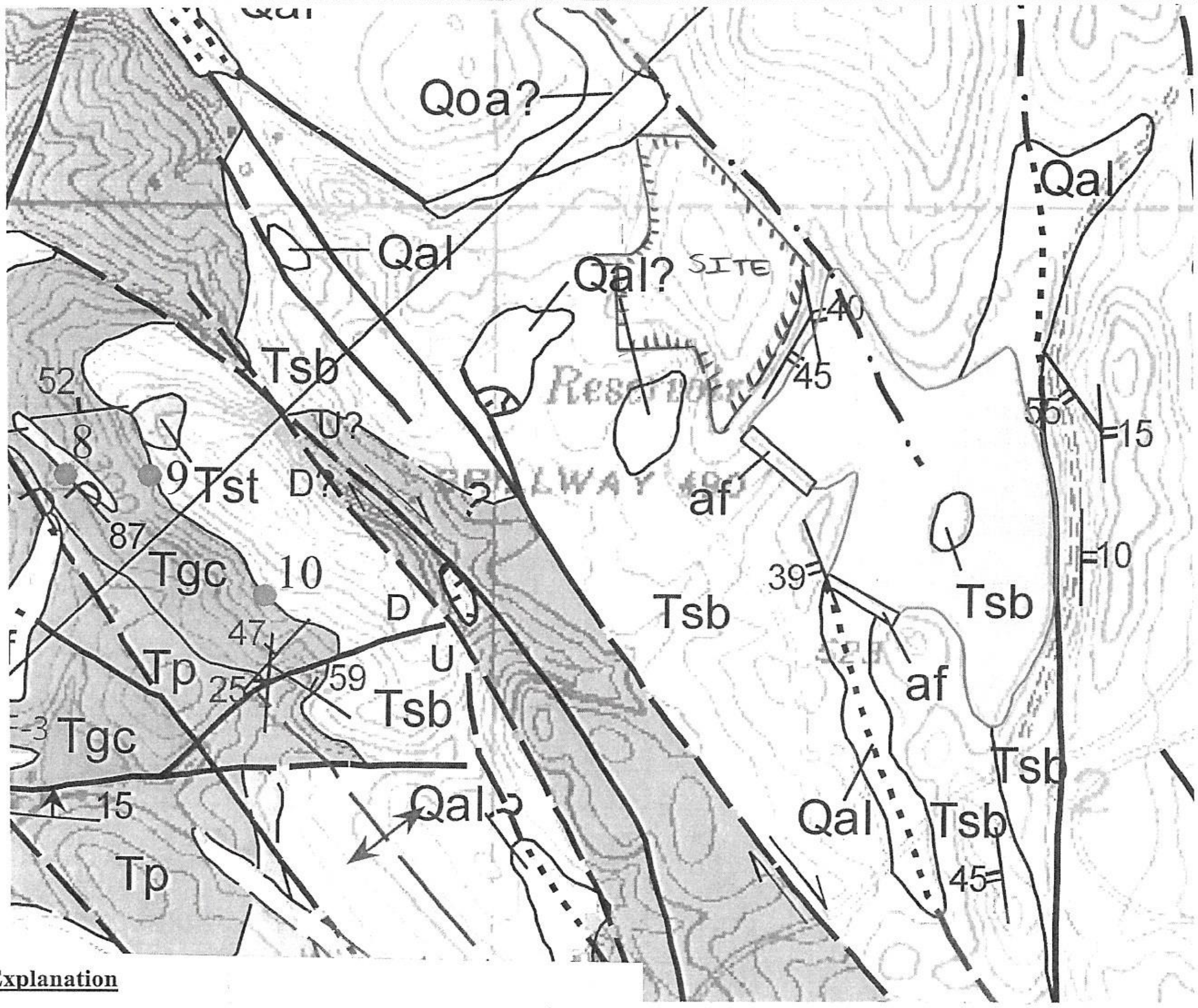
SCALE: 1 inch = 1,100 feet (±)

Special Studies Zone/Earthquake Fault Zones

Santa Rosa Quadrangle, 1983

CDMG

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	EARTHQUAKE FAULT ZONES MAP EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 3



Explanation

af - Artificial Fill

Qal - Stream and Valley Alluvium

Qoa - Older Alluvium

Qls - Landslides; arrows show direction of movement

Qt - Stream Terrace Deposits

Tge - Glen Ellen Formation

Tp - Petaluma Formation - Sandstone and conglomerate, some siltstone, locally interbedded tuff

Tst - Sonoma Volcanics - Rhyolitic to dacitic and minor andesitic pumiceous tuff

Tsb - Sonoma Volcanics- Andesite, basaltic andesite and basalt

Symbols

contact; dashed where approximate

dotted where concealed

queried where uncertain

fault dashed where approximate

dotted where concealed

queried where uncertain

14 strike and dip of bedded rocks

39 Strike and dip of volcanic flow unit

Scale: 1 inch = 1000 feet

Geologic Map prepared by USGS, 2008

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	REGIONAL SITE GEOLOGIC MAP EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 4

A: CONSOLIDATION OF SEDIMENTARY ROCKS; usually determined from unweathered samples.

Largely dependent on cementation

1. U = unconsolidated
2. P = poorly consolidated
3. M = moderately consolidated
4. W = well consolidated

B: BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness (in feet)	Stratification
1. Massive	Greater than 4.0 ft	very thick bedded
2. Blocky	2.0 to 4.0 ft	thick bedded
3. Slabby	0.2 to 2.0 ft	thin bedded
4. Flaggy	0.05 to 0.2 ft	very thin bedded
5. Shaly or platy	0.01 to 0.05 ft	laminated
6. Papery	Less than 0.01 ft	thinly laminated

C: FRACTURING

Intensity	Size of Pieces (in feet)
1. Very little fractured	Greater than 4.0 ft
2. Occasionally fractured	1.0 to 4.0 ft
3. Moderately fractured	0.5 to 1.0 ft
4. Closely fractured	0.1 to 0.5 ft
5. Intensely fractured	0.05 to 0.1 ft
6. Crushed	Less than 0.05 ft

D: HARDNESS

1. **Soft -** Reserved for plastic material alone.
2. **Low hardness -** can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blow away.
4. **Hard** - can be scratched with difficulty; scratch produces little powder and is often faintly visible
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak

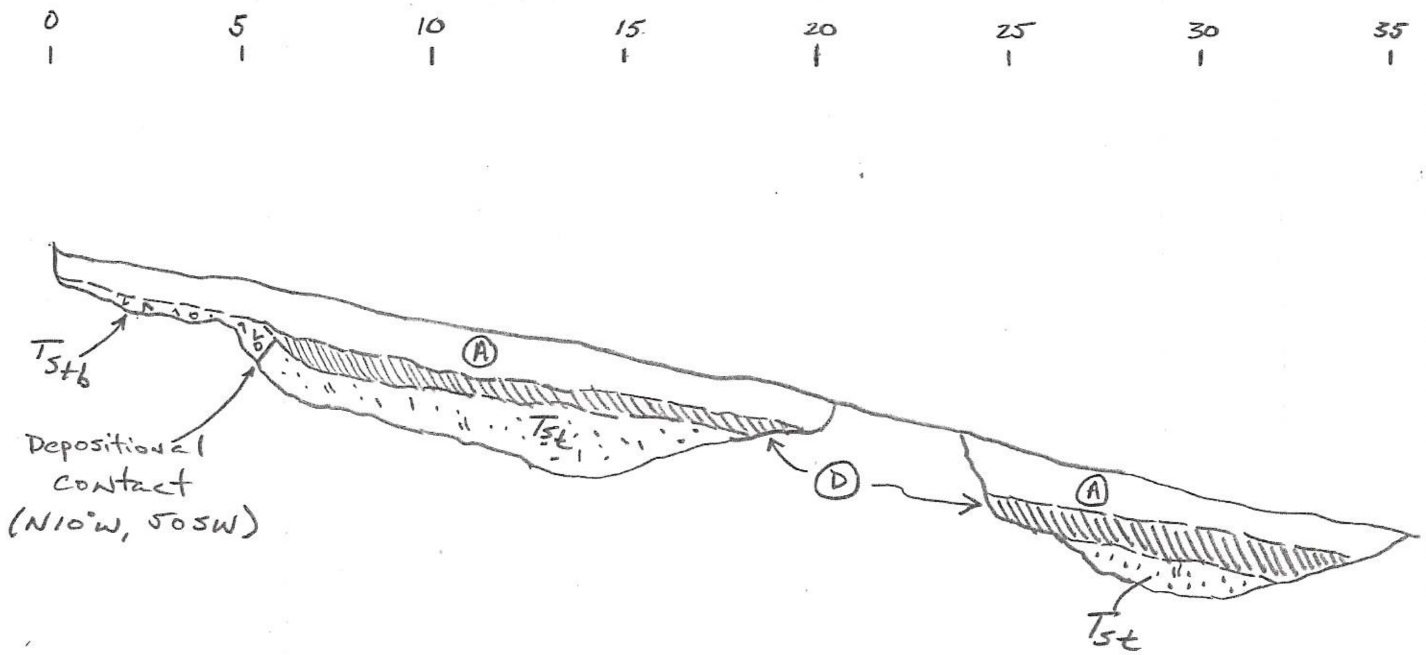
E: STRENGTH

1. **Plastic** - of very low strength.
2. **Friable -** Crumbles easily by rubbing with fingers.
3. **Weak** - An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong -** Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong -** Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

F: 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

1. **Deep -** Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or
2. **Moderate -** Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasional intense discoloration. Moderately coated fractures.
3. **Little -** No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
4. **Fresh -** Unaffected by weathering agents. No disintegration or discoloration.

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	**PHYSICAL PROPERTIES FOR ROCK DESCRIPTIONS** **EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA**	**PLATE** **5**



EXPLANATION OF TT-4

A: BROWN SANDY SILT (ML), soft to medium stiff, dry, with root, porous

D: GRAY- BROWN SANDY SILT (MH), stiff, moist, developed on Tst

Tsab: YELLOW-BROWN TUFF BRECCIA, moderately hard, moderately strong, moderately weathered

Tst: LIGHT BROWN TUFF, low hardness, friable, massive, little fractured, fine to medium grained ash matrix

SCALE: 1" = 5' HORIZONTAL & VERTICAL

0 5

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.5.1 Date: 08-29-16 Appr: BFP	LOG OF TEST TRENCH TT-4 EMERALD ISLE SKILLED NURSING FACILITY SANTA ROSA, CALIFORNIA	PLATE 6d

APPENDIX B

MILLER PACIFIC
ENGINEERING GROUP

June 9, 2016
File: 2317.001altr.doc

Reese & Associates
134 Lystra Court
Santa Rosa, California 95403

Attn: Mr. Jeffrey Reese

Re: Site Specific Seismic Design Criteria
Emerald Isle Skilled Nursing Facility
Santa Rosa, California

Introduction

This letter presents our site specific seismic design criteria for the planned Emerald Isle Skilled Nursing Facility located in Santa Rosa, California. The purpose of our services is to provide a site specific seismic hazard analysis as outlined in the 2013 California Building Code and the 2010 ASCE-7. Our work is being performed in general conformance with our Agreement dated July 15, 2015.

B. Seismic Design

The project site is located in a seismically active area and approximately 0.61 kilometers (0.38-miles) northeast of the Rodgers Creek Fault. Therefore, the proposed structures should be designed in conformance with the seismic provisions of the 2013 California Building Code (CBC) to mitigate the effects of potential ground shaking to the proposed structures. However, since the goal of the building code is protection of life safety, some structural damage may still occur during strong ground shaking.

Based on conversations with you, we understand the project site is underlain by relatively shallow soils overlying weathered bedrock. Additionally, based on the subsurface conditions you classified the site as a Site Class "B", weathered bedrock conditions.

Due to the proximity of the project site to the Rodgers Creek Fault the 2013 CBC Mapped spectral acceleration parameter at a period of 1.0 second (S_1 = 1.03 g) is greater than 0.75 g. Per 2013 CBC Section 1613A.3.5 and the structure's risk category, the site should be assigned to Seismic Design Category E if the planned structure is not considered an essential facility. If the structure is considered an essential facility the Seismic Design Category should be downgraded to F. Per the 2013 CBC Section 1616A.1.3, a ground motion hazard analysis shall be performed per ASCE 7-10 Chapter 21 for sites assigned a Seismic Design Categories E or F. Therefore, we performed a Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Hazard Analysis per ASCE 7-10 Section 21.2, as outlined below.

Probabilistic (MCE_R) Ground Motions: Method 1 – A probabilistic acceleration response spectrum, corresponding to a 2% chance of exceedance in 50-years (2,475 return period) was generated utilizing the United States Geologic Survey (USGS) interactive de-aggregation 2008 (https://geohazards.usgs.gov/deaggint/2008/index.php) for a Weathered Rock Profile (V_S^{30} = 760 m/s). The accelerations given were modified by the risk coefficients C_{RS} and C_{R1}, 0.94 and 0.93, respectively. The accelerations were further converted to the probabilistic spectral response acceleration in the maximum horizontal response utilizing the procedures outlined by Shahi and Baker, 2013. These modifications to the probabilistic spectra correspond to a response with a risk targeted level of 1% probability of collapse within a 50-year period. The resulting probabilistic spectra is presented on Figure 1.

Deterministic (MCE_R) Ground Motions – A deterministic acceleration response spectrum was generated utilizing the NGA attenuation models outlined by Campbell & Borzognia (2008), Chiou & Youngs (2008), and Boore & Atkinson (2008) NGA models for a Weathered Rock Profile (V_S^{30} = 760 m/s). The geometric average of the 84th percentile spectral accelerations from the aforementioned attenuation relationships were modified to the maximum horizontal direction, utilizing the procedures outlined by Shahi and Baker, 2013, as shown on Figure 1. The lower limit Deterministic MCE_R, as described in ASCE 7-10 Figure 21.2-1, is also plotted on Figure 1.

Site Specific MCE_R – The site specific MCE_R spectral response acceleration at any period shall be taken as the lesser of the response accelerations from the probabilistic ground motions and the deterministic ground motions, and is presented on Figure 2. Additionally, per ASCE 7-10 Chapter 21.3, the design response spectra is 2/3rd the site specific MCE_R spectra, as shown on Figure 2. However, per ASCE 7-10 Section 21.3, the site specific response spectrum should not be less than 80% of the general response spectrum (ASCE 7-10 Section 11.4.5), as shown on Figure 2. Based on the aforementioned procedures, the Site Specific Design Spectra is presented on Figure 3.

Per ASCE 7-10 Section 21.4, the Site Specific MCE_R spectral response acceleration parameters shall be taken from the Site Specific Design Spectra as follows and are presented on Table A below:

- S_{DS} – The S_{DS} parameter shall be taken as the spectral acceleration at a period of 0.2-seconds. However, S_{DS} shall not be less than 90% of the peak spectral acceleration at any period greater than 0.2-seconds.
- S_{D1} – The S_{D1} parameter shall be taken as the larger of the spectral acceleration at 1.0-second or two times the spectral acceleration at 2.0-seconds.
- S_{MS} – The S_{MS} parameter is equal to 1.5 times the S_{DS} value.
- S_{M1} – The S_{M1} parameter is equal to 1.5 times the S_{M1} value.

TABLE A
SITE SPECIFIC SEISMIC COEFFICIENTS
Emerald Isle Skilled Nursing Facility
Santa Rosa, California

Factor Name	Coefficient	Site Specific Value
Spectral Response (short)	SM_S	2.70 g
Spectral Response (1-sec)	SM_1	1.02 g
Design Spectral Response (short)	SD_S	1.80 g
Design Spectral Response (1-sec)	SD_1	0.68 g

Notes:
1. Parameters based on 2013 CBC & ASCE 7-10
2. Site coefficients adjusted for a Site Class B with a shear wave velocity of 760 meters per second in the upper 30 meters.

We hope this provides you with the information you require at this time. Please do not hesitate to contact us with any questions or concerns.

Very Truly Yours,
MILLER PACIFIC ENGINEERING GROUP

REGISTERED PROFESSIONAL ENGINEER
BENJAMIN S. PAPPAS
No. 2786
Exp. 9/30/16
GEOTECHNICAL
STATE OF CALIFORNIA

Benjamin S. Pappas
Geotechnical Engineer No. 2786
(Expires 9/30/16)

ETS
Environmental Technical Services
-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support
975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
e-mail: entech@pacbell.net

Serving people and the environment so that both benefit.

COMPANY: Reese & Associates,			ANALYST(S)	SUPERVISOR
ATTN: Brian Piazza		DATE of	S. Santos	D. Jacobson
JOB NAME: Emerald Isle, Fountaingrove area, Santa Rosa,	DATE RECEIVED	COMPLETION	G. Hernandez	LAB DIRECTOR
JOB #: 202.2.1 California	6/20/2016	6/29/2016		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL MIN RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
069130-1	EI1/SR	Sample #1 (1 @ 1', 1e @ 1', 3 @ 0.5' 4 @ 0.5', 5 @ 0.5', 6 @ 0.5')	5.34	1,521	[657]	90	38
06930-2	EI2/SR	Sample #2 {1e @ 5.5', 9 @ 3.5'}	5.49	1,025	[976]	75	45
Method	Detection	Limits -->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
069129-1	BS1/SF	#1 @ 7.5'				+338.6	
						+371.9	
Method	Detection	Limits -->	---	0.1	0.1	1	0.1

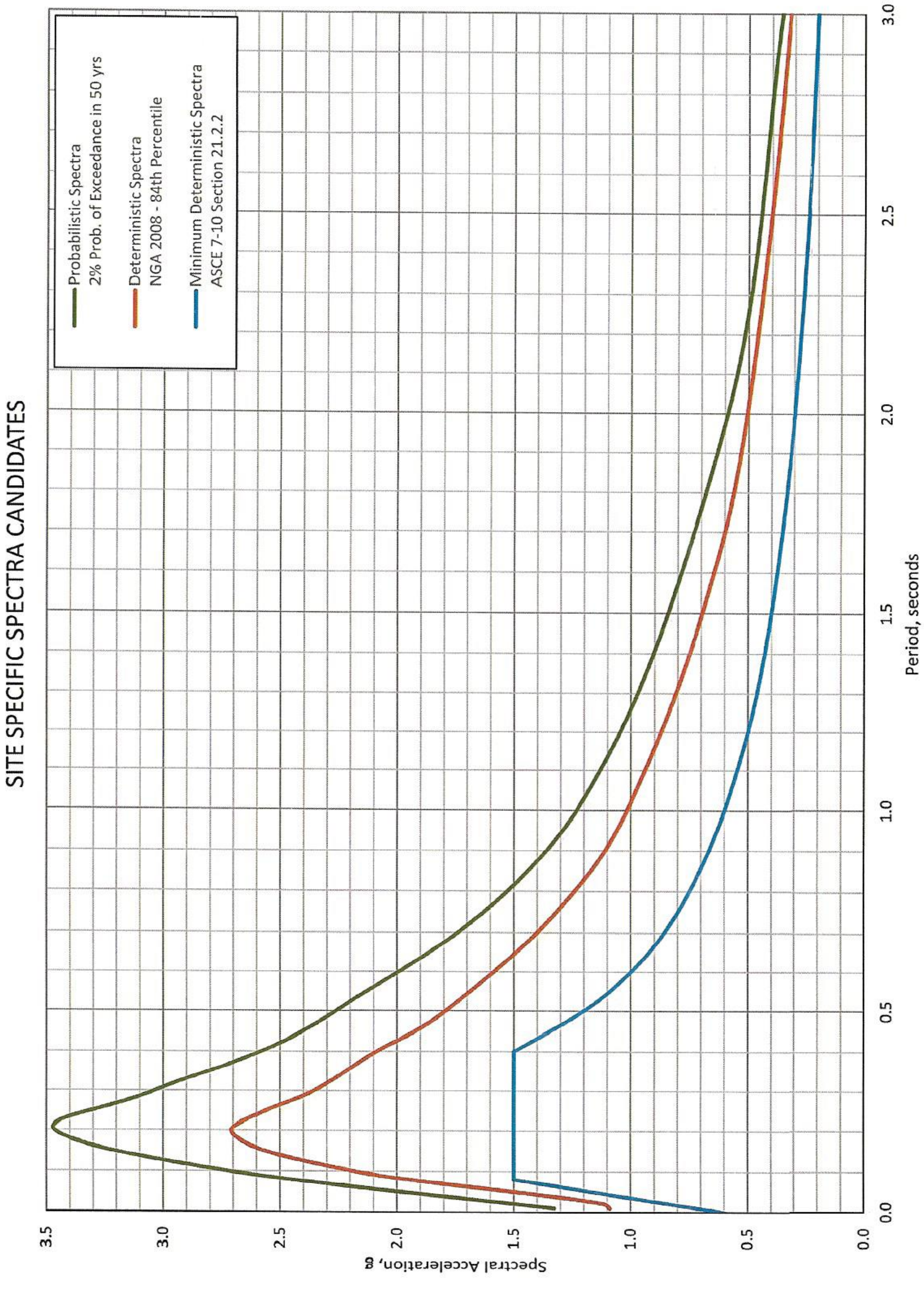
COMMENTS

Resistivities are at >1,500 ohm-cm, i.e., mediocre, and >1,000 ohm, i.e., very low; soil reactions (i.e., pHs) are mildly acidic; sulfates are low (i.e., @ <200 ppm), and chlorides are low (i.e., @ >100 ppm); and soils are very mildly reduced; [see table below on right for assigned point values and ranges]. The CalTrans (CT) times to perforation and full depth pitting times (followin Uhlig) for these soils are determined based on pertinent parameters [see table at left below]. Sulfate is not an issue for concret cement, mortar or grout; and chloride is low enough that it would not have any adverse impact on rebar or buried steel. Lime, mild cement (@ 1%-2%) or regular cement (@ ~5%+) treatment, in principle, could be of benefit in that raising sols pHs to the 7.5-8.5 range would increase 18 ga, 12 ga and 2 mm depth times to perf as indicated below. Lime treatment tends to be relatively temporary in the open environment, I.e., it is only long term under protected installations, and mild cement treatment can have a greater effective lifetime in semi-exposed locations as it results in more matrix binding. Regular cement treatment is lon term resulting in greatest matrix binding adding greater strength but also more rigidity. To increase metals longevity any more i these soils would require steel upgrading or other actions. At times, structural strength considerations may require heavier gau steel than is used in the presented examples such that perf and pitting times can be beyond specified life span. Where this is r true, cathodic protection along with coating or wrapping steel assets is one potential solution. Other options include increased/ specialized engineering fill, use of a polymer coating, or use of plastic, fiberglass or concrete assets. Based on these results, standard concrete mixes should be fine in these soils.

SAMPLE ID	CT 18 ga	CT 12 ga	2 mm (Uhlig)
EI1/SR	<9 yrs	~19.5 yrs	<7 yrs
Treated	<30 yrs	~65 yrs	~21.5 yrs
EI2/SR	~7 yrs	>15 yrs	~6 yrs
Treated	~25 yrs	~55.5 yrs	~18 yrs

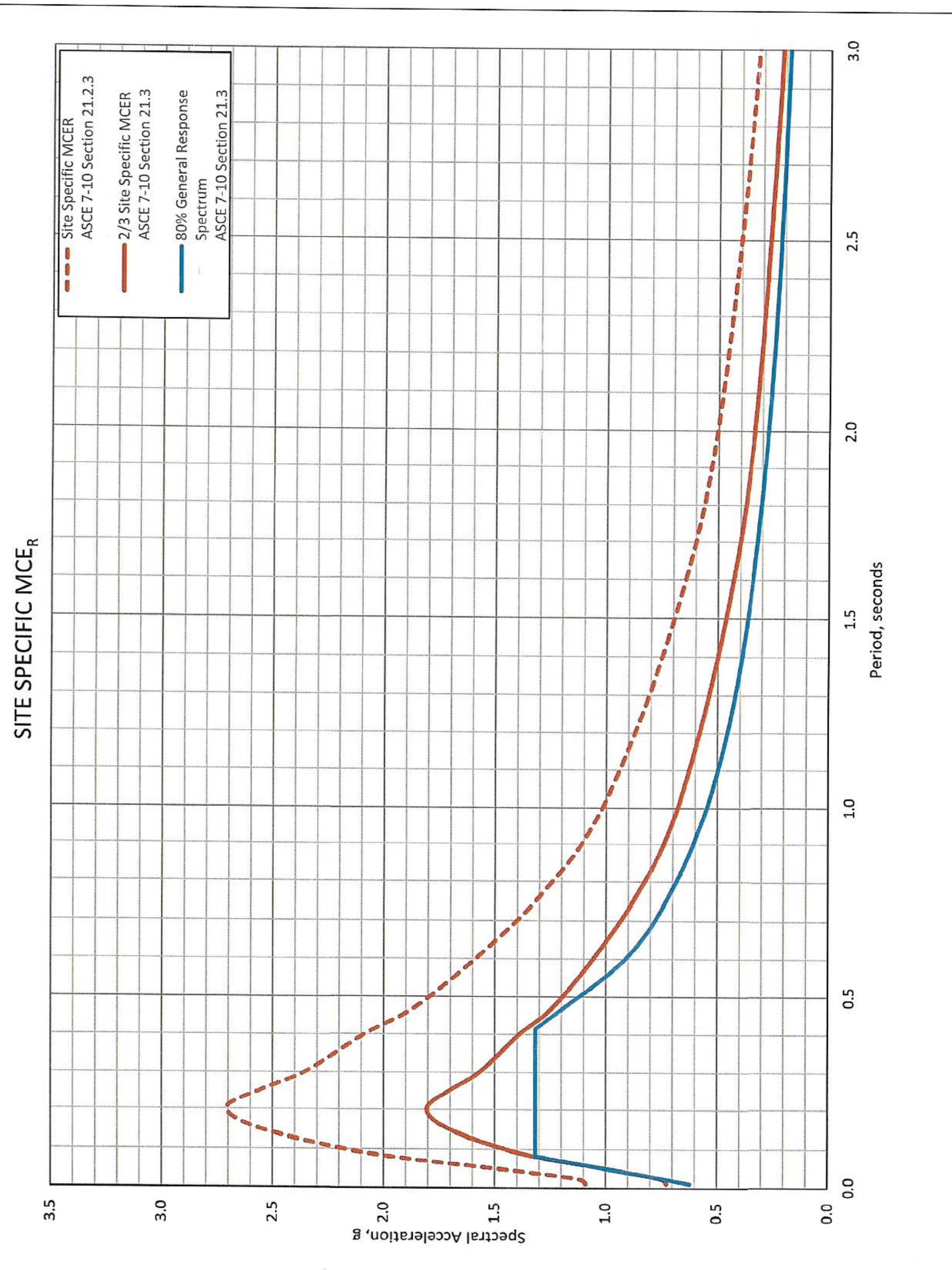
PARAMETER/ID	EI1/SR	EI2/SR
pH	Ø	Ø
Rs	1-8	5-10
SO4	Ø	Ø
Cl	Ø	Ø
Redox	Ø-3.5	Ø-3.5
TOTALS	1-11.5	5-13.5

\\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

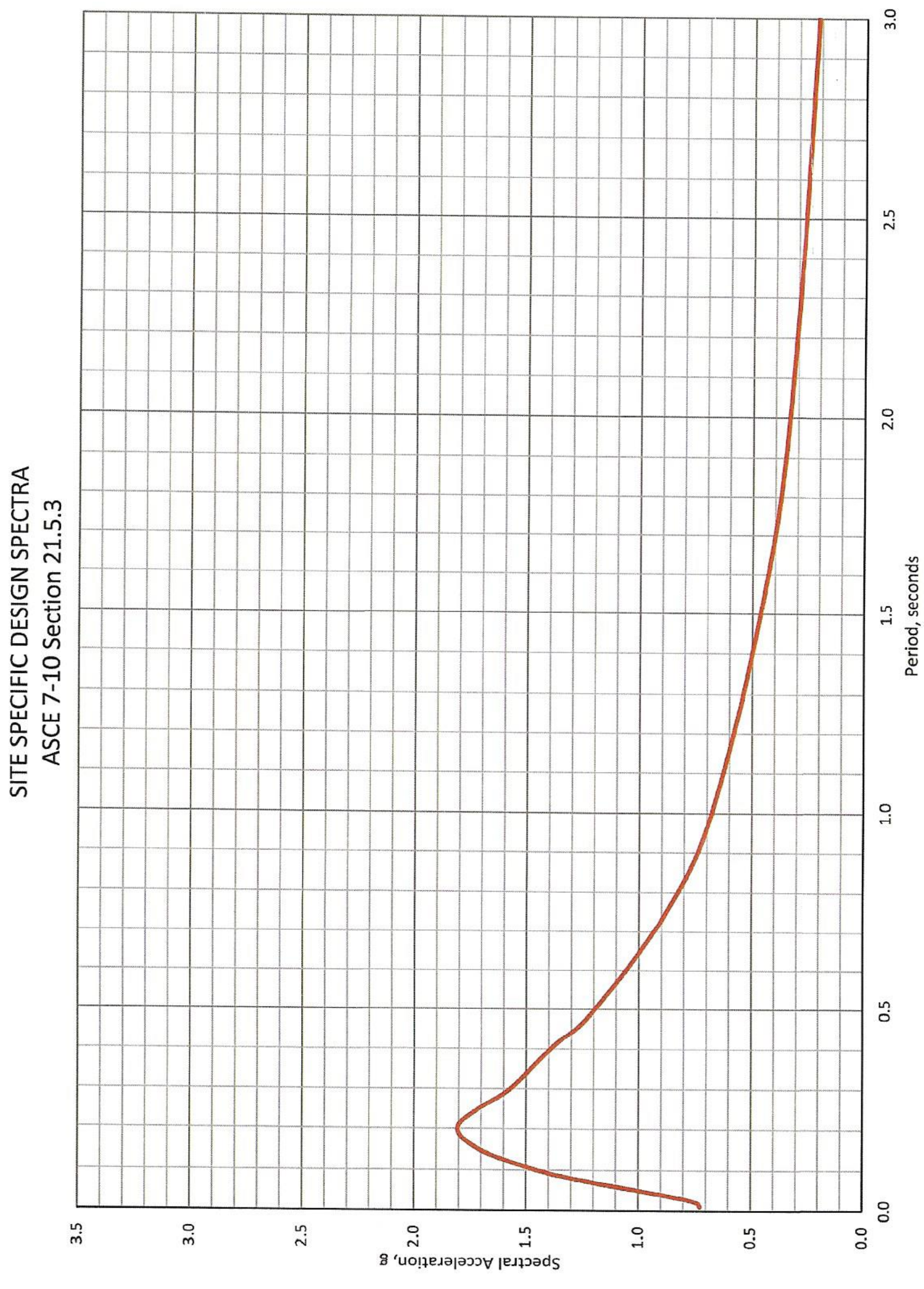


MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE SPECIFIC SPECTRA CANDIDATES Emerald Isle Skilled Nursing Facility Santa Rosa, California Project No. 2317.001 Date: 6/9/2016	Drawn BSP Checked	1 FIGURE

FILE: 2317.001 Spectra.dwg



MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE SPECIFIC MCE	
		Emerald Isle Skilled Nursing Facility Santa Rosa, California	Drawn BSP Checked
FILE: 2317.001 Spectra.dwg		Project No. 2317.001 Date: 6/9/2016	2 FIGURE



MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE SPECIFIC DESIGN SPECTRA	
		Emerald Isle Skilled Nursing Facility Santa Rosa, California	Drawn BSP Checked
FILE: 2317.001 Spectra.dwg		Project No. 2317.001 Date: 6/9/2016	3 FIGURE

THIS PAGE INTENTIONALLY LEFT BLANK