SCOTTS VALLEY WATER DISTRICT

GEOTECHNICAL INVESTIGATION

BETHANY WATER TANK REPLACEMENT
SCOTTS VALLEY, CALIFORNIA

FOR
SCOTTS VALLEY WATER DISTRICT
SCOTTS VALLEY, CALIFORNIA

Pacific Crest
ENGINEERING INC

CONSULTING GEOTECHNICAL ENGINEERS

24069-SZ34-D53
JUNE 2025
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Pacific Crest ENGINEERING INC

GEOTECHNICAL | ENVIRONMENTAL | CHEMICAL | MATERIAL TESTING | SPECIAL INSPECTIONS

June 13, 2025 Project No. 24069-SZ34-D53

Mr. Nate Gillespie
Scotts Valley Water District
2 Civic Center Drive
Scotts Valley, CA 95066

Subject: **Geotechnical Investigation – Design Phase**
Bethany Tank Replacement
Scotts Valley, California

Dear Mr. Gillespie,

In accordance with your authorization, we have performed a geotechnical investigation for the proposed replacement of the Bethany Water Tank project located in Scotts Valley, California. The accompanying report presents our conclusions and recommendations as well as the results of the geotechnical investigation on which they are based.

Pacific Crest Engineering Inc has also performed a geologic investigation for this project. The geologic investigation report is presented in Appendix A of this report and should be reviewed in conjunction with this report.

The conclusions and recommendations presented in this report are contingent upon our review of the plans during the design phase of the project, and our observation and testing during the construction phase of the project.

Very truly yours,

PACIFIC CREST ENGINEERING INC.

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GEOTECHNICAL INVESTIGATION REPORT

Bethany Water Tank Replacement
Scotts Valley, California

I. INTRODUCTION

PURPOSE AND SCOPE

This report describes the geotechnical investigation and presents our conclusions and recommendations for the proposed water tank replacement project in Scotts Valley, California.

Our scope of services for this project have consisted of:

1. Site reconnaissance to observe the existing conditions.

2. Review of the following published maps:
   - Geologic Map of Santa Cruz County, California, Brabb, 1997.
   - Preliminary Map of Landslide Deposits in Santa Cruz County, California, Cooper-Clark and Associates, 1975.
   - Map Showing Geology and Liquefaction Potential of Quaternary Deposits in Santa Cruz County, California, Dupré, 1975.
   - Map Showing Faults and Their Potential Hazards in Santa Cruz County, California, Hall, Sarna-Wojcicki, Dupré, 1974.
   - U.S. Geological Survey (and the California Geologic Survey), 2018, Quaternary fault and fold database for the United States, accessed in May of 2024, from USGS web site: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults

3. Review of geological investigation, "Bethany Tank Replacement" by Pacific Crest Engineering, Project Number 24069, dated June 3rd, 2025 (Appendix A).

4. Review of previous geotechnical investigation, "Bethany Water Tank Upgrade Project" by Bauldry Engineering, Inc. Project Number 1020-SZ932-H73, dated July 30th, 2010 (Appendix B).

5. Engineering analysis of compiled data.

6. Preparation of this report documenting our investigation and presenting geotechnical recommendations for the design and construction of the project.

PROJECT LOCATION

The project site is currently comprised of a water storage facility located at the northern terminus of Tabor Drive in Scotts Valley, California. Please refer to Figure No. 1, Topographic Index Map in Appendix A for the general vicinity of the project site, which is approximately located by the following coordinates:

	Tank Site	
Latitude	=	37.077773
Longitude	=	-121.995776

PROPOSED IMPROVEMENTS

Based on our discussions with the project design team, it is our understanding that the existing water tank is scheduled to be replaced. The new water storage tank(s) will be installed in the same general location as the existing water storage tank. Due to site constraints, the design team may opt to install two smaller tanks to replace the existing single tank. The proposed water storage tank(s) are expected to consist of welded steel tanks supported on a structural mat foundation.

II. FINDINGS AND ANALYSIS

GEOLOGIC SETTING

Please refer to the geologic investigation report in Appendix A for a detailed discussion regarding site geology.

SURFACE CONDITIONS

The tank site is located at the northern terminus of Tabor Drive in a single-family rural residential subdivision of the Santa Cruz Mountains and is currently occupied by an approximately 400,000-gallon welded steel water tank and associated utilities. The tank site is a relatively flat cut pad located on a narrow ridge crest, with very steep topography to either side of the ridge crest.

SUBSURFACE CONDITIONS

Our subsurface exploration consisted of two exploratory trenches excavated as part of our geologic investigation (Appendix A). We also reviewed subsurface boring data from a 2010 geotechnical investigation (Appendix B). The exploratory trenches extended up to 6 feet below existing grade and ranged from 69 to 85 feet in length. The soil profile is shown on Plate 2 in Appendix A. General subsurface conditions are described below.

Artificial fill and surficial soils were initially encountered near the ground surface at our exploratory trench locations. This fill thickness ranged on the order of a few inches to a few feet near the outboard edges of the ridge crest. The fill/surficial soils were generally described as sandy silt with gravel. Underlying the surficial soils a highly fractured “crushed zone” of Purisima Formation Bedrock was encountered. Immediately underlying the crushed zone, intact Purisima Formation Bedrock was encountered.

Please refer the trench logs, Plate 2 in Appendix A, for a more detailed description of the subsurface conditions encountered in our exploratory trenches.

Groundwater was not encountered in our exploratory trenches, nor in the 2010 Bauldry Engineering borings to a maximum depth of 24½ feet, however it must be anticipated that perched and regional groundwater tables may vary with location and could fluctuate with variations rainfall, runoff, irrigation, and other changes to the conditions existing at the time our measurements were made. It should be anticipated that the groundwater table may rise or fall significantly during the course of a given year.

FAULTING AND SEISMICITY

Faulting

Mapped faults which have the potential to generate earthquakes that could significantly affect the subject site are listed in Table No. 1. The fault distances are approximate distances based on the U.S. Geological Survey and California Geological Survey, Quaternary fault and fold database, accessed in May 2025 from the USGS website (https://www.usgs.gov/programs/earthquake-hazards/faults) and overlain onto Google Earth.

Table No. 1 - Distance to Significant Faults

Fault Name	Distance (miles)	Direction
Zayante-Vergeles	1½	Northeast
San Andreas	4½	Northeast
Sargent	5	Northeast
San Gregorio	14	Southwest

Seismic Shaking and CBC Design Parameters

Due to the proximity of the site to active and potentially active faults, it is reasonable to assume the site will experience high intensity ground shaking during the lifetime of the project. Structures founded on thick, soft soil deposits are more likely to experience more destructive shaking, with higher amplitude and lower frequency, than structures founded on bedrock. Generally, shaking will be more intense closer to earthquake epicenters. Thick, soft soil deposits large distances from earthquake epicenters, however, may result in seismic accelerations significantly greater than expected in bedrock.

Selection of seismic design parameters should be determined by the project structural designer. The site coefficients and seismic ground motion values shown in the table below were developed based on CBC 2022 incorporating the ASCE 7-16 standard, and the project site location.

Table No. 2 - 2022 CBC Seismic Design Parameters [Note 1]

Seismic Design Parameter	ASCE 7-16 Value
Site Class	C
Spectral Acceleration for Short Periods	S_s = 2.358g
Spectral Acceleration for 1-second Period	S_1 = 0.944g
Short Period Site Coefficient	F_a = 1.2
1-Second Period Site Coefficient	F_v = 1.4
MCE Spectral Response Acceleration for Short Period	S_{MS} = 2.83g

Seismic Design Parameter	ASCE 7-16 Value
MCE Spectral Response Acceleration for 1-Second Period	S_{M1} = 1.322
Design Spectral Response Acceleration for Short Period	S_{DS} = 1.886g
Design Spectral Response Acceleration for 1-Second Period	S_{D1} = 0.882

Note 1: Design values have been obtained by using the ASCE Hazard Tool at https://asce7hazardtool.online

The recommendations of this report are intended to reduce the potential for structural damage to an acceptable risk level, however strong seismic shaking could result in damage to improvements and the need for post-earthquake repairs. It should be assumed that exterior improvements such as pavements or sidewalks may also need to be repaired or replaced following strong seismic shaking.

GEOTECHNICAL HAZARDS

A quantitative analysis of geotechnical hazards was beyond our scope of services for this project. In general, however, the geotechnical hazards associated with the project site include seismic shaking (discussed above), ridge top shattering, liquefaction, expansive soils and lateral spreading. A qualitative discussion of these hazards is presented below. Please refer to Appendix A for discussion regarding geologic hazards, which include coseismic ground cracking and landsliding.

Liquefaction and Lateral Spreading

Based upon our review of the Santa Cruz County GIS Hazard Maps, the project site is not mapped within a liquefaction hazard zone.

Liquefaction tends to occur in loose, saturated fine-grained sands and coarse silt, or clays with low plasticity. The site is underlain by bedrock within a few feet of the ground surface, which is typically not considered susceptible to liquefaction. Consequently, it is our opinion that the potential for liquefaction to occur at the subject site may be considered low.

Liquefaction induced lateral spreading occurs when a liquefied soil mass fails toward an open slope face or fails on an inclined topographic slope. Our analysis indicates that the site has a low potential for liquefaction, consequently the potential for lateral spreading is also considered low.

Landsliding

Based on our review of the Santa Cruz County GIS website, the site is not mapped on a known landslide deposit, however our geological investigation did note some areas of historic landsliding. The subject tank is underlain by shallow bedrock. Provided that the recommendations of this report and our geological report are closely followed, it is our opinion that the potential for shallow landsliding to occur and adversely affect the proposed water tank(s) can be mitigated to an acceptable risk level.

Slope failures can also occur where surface drainage is allowed to concentrate onto unprotected slopes. Appropriate landscaping and good control of surface drainage around the project area becomes very important to reduce potential for shallow slumping of slopes. Erosion control measures should be implemented and maintained. Under no circumstances should surface runoff be concentrated and directed toward, or discharged upon, any topographic slopes.

<u>Expansive Soils</u>

Expansive soils tend to heave during the rainy season and contract during the summer and this shrink/swell action extends down to the depth of seasonal moisture change. When this cyclical volume change occurs on sloping ground it results in "soil creep" due to the downward vector of the shrink/swell action. Seasonal moisture fluctuation and subsequent expansion and contraction of these types of soils typically occurs more near the ground surface where the seasonal moisture fluctuation is the greatest and decreases with depth below ground surface.

The surficial soils at the site consist of non-expansive sandy silt with gravel underlain by Purisima Formation bedrock. Consequently, we consider the hazard due to expansive soils to be low.

<u>Ridge Top Shattering</u>

Structures founded upon ridge tops may experience particularly destructive shaking and intense seismic forces due to the phenomenon known as "ridge top shattering" or "ridge top spreading" during seismic events. The interaction between the geometry of a ridge and seismic shock waves can cause concentration and amplification of the seismic energy near and at the ridge crest. The consequences of this concentration and amplification of seismic energy is increased seismic shaking and potentially severe fracturing of the ground surface along and near the ridge top.

The proposed tank site is located near the top of a ridge in the Santa Cruz Mountains. During the 1989 Loma Prieta Earthquake large fractures were mapped in the vicinity of this project site which were possibly due to the ridge top shattering phenomenon. The potential for this phenomenon to occur in the future during a comparable seismic event is high to very high. Please refer to Appendix A for more information regarding ridge top spreading.

The recommendations of this report are intended to minimize, but will never eliminate, the potential for damage that ridge top shattering may cause to structures and foundation systems at this site.

III. <u>DISCUSSION AND CONCLUSIONS</u>

<u>GENERAL</u>

1. The results of our investigation indicate that the proposed improvements are feasible from a geotechnical engineering standpoint, provided our recommendations are included in the design and construction of the project.

2. Grading and foundation plans should be reviewed by Pacific Crest Engineering Inc. during their preparation and prior to contract bidding.

3. Pacific Crest Engineering Inc. should be notified at least four (4) working days prior to any site clearing and grading operations on the property in order to observe the stripping and disposal of unsuitable materials, and to coordinate this work with the grading contractor. During this period, a pre-construction conference should be held on the site, with at least the client or their representative, the grading contractor, and one of our engineers present. At this meeting, the project specifications and the testing and inspection responsibilities will be outlined and discussed.

4. The findings, conclusions and recommendations provided in this report are based on the understanding that Pacific Crest Engineering will remain as Geotechnical Engineer of Record throughout the design and construction phase of the project. The validity of the findings, conclusions and recommendations contained in this report are dependent upon our review of project plans as well as an adequate testing and observation program during the construction phase. Field observation and testing must therefore be provided by a representative of Pacific Crest Engineering Inc., to enable us to form an opinion as to whether the extent of work related to earthwork or foundation excavation complies with the project plans, specifications, and our geotechnical recommendations. Pacific Crest Engineering assumes no responsibility for any site work that is performed without the full knowledge and direct observation of Pacific Crest Engineering Inc.

PRIMARY GEOTECHNICAL CONSIDERATIONS

5. The following section provides geotechnical considerations for the design and construction of the proposed water tank(s) and are intended for use in design of the project and preparation of the project plans and specifications. It is neither the intent nor within the scope of this investigation to recommend construction procedures or methods used by the Contractor. It is the responsibility of the Contractor to use sound construction procedures and methods of the industry in accordance with local, state and federal safety standards.

6. Variations in soil conditions due to local grading or seismic activity can occur and should be expected. Therefore, subsurface conditions at some locations may differ from those observed or inferred from this investigation. The presence of pre-existing utilities and variable trench backfill therein could also impact the site conditions and construction operations.

7. Based upon the results of our investigation, it is our opinion that the primary geotechnical issues associated with the design and construction of the proposed project are the following:

a. *Ridge Top Shattering:* The proposed tank site is located near the top of a bedrock ridge that experienced ridge top shattering during the 1989 Loma Prieta Earthquake; therefore, the potential for ridge top shattering at the site is high to very high. The effects of ridge top shattering can be reduced (but never eliminated) by supporting the tanks on structural mats underlain by geogrid reinforced earthen fills.

   The recommendations provided in this report assume all structural improvements will be sited entirely within the green shaded *"zone of weathered bedrock at the ground surface"* on Plate 3 in Appendix A. Significant design changes will be required should structural and/or ground

improvements extend beyond this zone. Please refer to our Geologic Investigation Report in Appendix A.

b. *Shallow Hard Bedrock:* Hard bedrock will likely be encountered at shallow depths during construction of the proposed improvements. All earthwork contractors should be aware of these conditions and employ the appropriate earthwork equipment to achieve the proper excavation depths.

c. *Remnant Effects of Demolition Operation:* It is our understanding that the existing tank and its foundation will be demolished as part of this project. The exact method of demolition of the existing water tank is not known at this time, but it is possible that that the upper 2-3 feet of soil/bedrock will be highly disturbed during this process. These soils will need to be sub-excavated and recompacted as engineered fill. Refer to the Site Preparation section of this report for recommendations.

d. *Unknown Conditions Under Existing Tank:* The recommendations developed in this report are based on subsurface data outside the footprint of the existing tank. Our office should observe exposed site conditions following the demolition of the tank and provide additional recommendations, if needed, to address unforeseen conditions.

e. *Landsliding:* As discussed, it is our understanding that areas of historic landsliding have been observed at site. Our Geology report recommends that the tank(s), foundation elements and other improvements be located away from these areas. Please refer to Plate 3 in Appendix A for additional information.

f. *Strong Seismic Shaking:* The project site is located within a seismically active area and strong seismic shaking is expected to occur within the design lifetime of the project. Improvements should be designed and constructed in accordance with the most current CBC and the recommendations of this report to minimize reaction to seismic shaking. Structures built in accordance with the latest edition of the California Building Code have an increased potential for experiencing relatively minor damage which should be repairable, however strong seismic shaking could result in damage to improvements and the need for post-earthquake repairs.

IV. RECOMMENDATIONS

TRENCHING, OPEN-CUT EXCAVATIONS AND SHORING

1. We anticipate moderately hard to hard bedrock conditions to be encountered within the planned replacement tank(s) location. The contractor should employ the appropriate equipment to assure excavation to the design depths.

2. It must be understood that on-site safety is the sole responsibility of the contractor, and that the contractor shall designate a competent person (as defined by CAL-OSHA) to monitor the slope excavation prior to the start of each workday, and throughout the workday as conditions change. The competent person designated by the contractor shall determine if flatter slope gradients are more

appropriate, or if shoring should be installed or modified to protect workers in the vicinity of the slope excavation. Refer to Title 8, California Code of Regulations, Sections 1539-1543. All excavations must be evaluated for stability prior to entry. The contractor must act in accordance with the project specifications, Cal/OSHA and/or any other applicable government regulation concerning excavation safety and shoring.

3. All excavations must meet the requirements of 29 CFR 1926.651 and 1926.652 or comparable OSHA approved state plan requirements.

4. Based on the results of our investigation, we recommend that for sloping and benching purposes, the soils within the project site may be preliminarily classified as Type B soils in accordance with Cal/OSHA. The contractor's competent person must base their sloping and benching systems on the actual soil and groundwater conditions encountered in the field at the time of construction.

5. The "top" of any temporary cut slope should be set-back at least ten feet (measured horizontally) from any nearby structure or property line. Any excavation that cannot meet these side slope gradients will need to have a shoring system designed to support steeper sidewall gradients.

6. Should temporary shoring be required, the shoring wall system chosen by the designer should be designed using the geotechnical design criteria presented in the "Lateral Pressures" section of this report. The contractor should submit a detailed shoring plan to the project civil, structural, and geotechnical engineers for review at least three weeks prior to the start of construction.

EXCAVATABILITY

8. The contractor should anticipate hard, Purisima Formation bedrock will be encountered during the excavation. It is the contractor's responsibility to independently assess the excavatability of the soil and bedrock at the project site, and to choose suitable equipment and/or excavation methods.

EARTHWORK

Clearing and Stripping

9. The initial preparation of the site is expected to consist of demolition of the existing tank and abandoned utilities, and the removal of debris. All demolished structures, including foundations, utilities and their debris must be completely removed from the construction area. The extent of this removal will be designated by a representative of Pacific Crest Engineering Inc. in the field. This material must be removed from the site.

10. Any voids created by the removal of abandoned structures and utilities must be backfilled with properly compacted engineered fill which meets the requirements of this report.

11. Any organically contaminated topsoil, if present, should then be removed ("stripped") from the areas to be graded. In addition, any remaining debris or large rocks must also be removed (this includes

asphalt or rocks greater than 2 inches in greatest dimension). This material may be stockpiled for future landscaping.

Subgrade Preparation

12. Man made fill was encountered within our exploratory trenches at the project site. It is possible that there are areas of man-made fill at the site that our field investigation did not detect. Areas of man-made fill, if encountered in the construction area, will need to be completely excavated to undisturbed native material. The excavation process should be observed, and the extent designated by a representative of Pacific Crest Engineering Inc., in the field. Any voids created by fill removal must be backfilled with properly compacted engineered fill.

13. After clearing and stripping are completed, the exposed subgrade at tank pad(s) should be removed to a minimum depth of 3 feet below the design subgrade elevation or as designated by a representative of Pacific Crest Engineering Inc. The excavation should extend a minimum of 5 feet horizontally beyond all foundation elements, unless site constraints preclude such horizontal limits. Any non-engineered fill remaining within the proposed construction areas will need to be completely excavated to undisturbed native material.

14. Following the excavation to design depth and the removal of all existing fill, the base of the excavation should be observed by a representative of Pacific Crest Engineering prior to any additional earthwork activities.

15. Once the base of the excavation is approved by Pacific Crest Engineering, the excavation should be brought to the design subgrade elevation as follows:

a. The exposed soils at the base of the excavation should be scarified, moisture conditioned and compacted as an engineered fill in accordance with the compaction requirements provided below.

b. Following the compaction of the base, a two-element geosynthetic barrier comprised of geogrid (Tensar InterAx NX-850 or approved equivalent) should be placed across the base of the excavation. The geogrid must be installed in accordance with the manufacturer's recommendations. The geosynthetics should be overlapped a minimum of 2 feet or in accordance with the manufacturer's instructions, whichever is greatest. The geosynthetics should be pulled taught and secured in place using stakes or other means before the backfill is placed. The placement and securing of the geosynthetics should be observed and approved prior to placement of fill.

c. The geosynthetic barrier should then be covered with 12 inches of compacted native soil, followed by another layer of geogrid similar to that noted above. The backfill material should be placed in accordance with the compaction requirements for soils provided below.

d. With the excavation now approximately 24 inches below the design subgrade elevation, the next 12 inches of the excavation should be backfilled with Class 2 aggregate base followed by another layer of geogrid. The aggregate base should be placed in accordance with the compaction requirements for aggregate base provided below. This lift should be proceeded by a final lift of another 12 inches of compacted aggregate base up to the design subgrade elevation.

e. The result should be a 3-foot-thick section of engineered fill comprised of 1 foot of recompacted native soil and 2 feet of aggregate base, reinforced with geogrid every 12 inches. All sections should extend a minimum of 5 feet horizontally beyond all foundation elements unless site constraints preclude such horizontal limits.

16. Any proposed import materials should be submitted to our office for approval at least three weeks before job site delivery.

17. Final depth of subexcavation should be determined by a representative of Pacific Crest Engineering Inc., in the field.

18. If wet or unstable subgrades are encountered at the base of excavations, they may need to be further subexcavated and replaced with stabilization fabric, crushed rock or other materials to create a stable working surface. The depth of over-excavations and method used should be determined in the field at the time of construction. All subexcavations should be observed by a representative of Pacific Crest Engineering Inc. and modified as necessary to establish a stable subgrade.

<u>Material for Engineered Fill</u>

19. The native/fill surficial soils are non-expansive and generally considered suitable for use as engineered fill below improvements or as trench backfill. All structural foundation elements should be underlain by non-expansive native and imported engineered fill as discussed above.

20. Native or imported soil proposed for use as engineered fill should meet the following:

a. free of organics, debris, and other deleterious materials,
b. free of “recycled” materials such as asphaltic concrete, concrete, brick, etc.,
c. granular in nature, well graded, and contain sufficient binder to allow trenches to stand open,
d. free of rocks in excess of 2 inches in size.

21. In addition to the above requirements, import fill should have a Plasticity Index between 4 and 12, and a minimum Resistance “R” Value of 30, and be non-expansive.

22. Samples of any proposed imported fill planned for use on this project should be submitted to Pacific Crest Engineering Inc. for appropriate testing and approval not less than fifteen (15) working days before the anticipated jobsite delivery. This includes proposed import trench sand, drain rock and aggregate base materials. Imported fill material delivered to the project site without prior submittal of samples for appropriate testing and approval must be removed from the project site.

Engineered Fill Placement and Compaction

23. Following sub-excavation and any required subgrade preparation, the excavation should be backfilled to finish grade with engineered fill that is moisture conditioned and compacted according to the recommendations of this report.

24. Engineered fill should be placed in maximum 8-inch lifts, before compaction, at a water content which is within 1 to 3 percent of the laboratory optimum value.

25. The minimum compaction requirements are outlined in the table below:

Table No. 3 - Minimum Compaction Requirements

Percent of Maximum Dry Density	Location
95%	• All aggregate base • The upper 8 inches of subgrade in pavement areas • Utility trench backfill in pavement areas
95%	• Native subgrade soils and the lower grid reinforced fill layer below foundations
90%	• All remaining compacted material

26. The maximum dry density will be obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557. This test will also establish the optimum moisture content of the material. Field density testing will be performed in accordance with ASTM Test #D6938 (nuclear method).

27. We recommend field density testing be performed in maximum 1 foot elevation differences. In general terms, we recommend at least one compaction test per 50 linear feet of utility trench or retaining wall backfill. This is a subjective value and may be changed by the geotechnical engineer based on a review of the final project layout and exposed field conditions.

Cut and Fill Slopes

28. No permanent cut or fill slopes are anticipated for this project. Should cut or fill slopes be proposed, supplemental geotechnical engineering recommendations will be required.

Soil Moisture and Weather Conditions

29. If earthwork activities are done during or soon after the rainy season, the on-site soils both above and below the water table may be too wet in their existing condition to be used as engineered fill. These materials may require a diligent and active drying and/or mixing operation to reduce the moisture content to the levels required to obtain adequate compaction as an engineered fill. If the on-site soils or other materials are too dry, water may need to be added. In some cases, the time and effort to dry the on-site soil may be considered excessive, and the import of aggregate base may be required.

Utility Trench Backfill

30. Utility trenches that are parallel to the sides of buildings should be placed so that they do not extend below a line sloping down and away at a 2:1 (horizontal to vertical) slope from the bottom outside edge of all footings.

31. Utility pipes should be designed and constructed so that the top of pipe is a minimum of 24 inches below the finish subgrade elevation of any road or pavement areas. Any pipes within the top 24 inches of finish subgrade should be concrete encased, per design by the project civil engineer.

32. For the purpose of this section of the report, backfill is defined as material placed in a trench starting one foot above the pipe, and bedding is all material placed in a trench below the backfill.

33. Unless concrete bedding is required around utility pipes, we recommend free-draining clean sand be used as bedding. Sand bedding should be compacted to at least 95 percent relative compaction. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

34. Approved material conforming to engineered fill materials as described above should be used as utility trench backfill. Backfill in trenches located under and adjacent to structural fill, foundations, concrete slabs and pavements should be placed in horizontal layers no more than 8 inches thick. This includes areas such as sidewalks and other hardscape areas. Each layer of trench backfill should be water conditioned and compacted to at least 95 percent relative compaction.

35. Utility trenches which carry "nested" conduits (stacked vertically) should be backfilled with a control density fill (such as 2-sack sand\cement slurry) to an elevation one foot above the nested conduit stack. The use of pea gravel or clean sand as backfill within a zone of nested conduits is not recommended.

36. A representative from our firm should be present to observe the bottom of all trench excavations, prior to placement of utility pipes and conduits. In addition, we should observe the condition of the trench prior to placement of sand bedding, and to observe compaction of the sand bedding, in addition to any backfill planned above the bedding zone.

37. Jetting of the trench backfill is not recommended as it may result in an unsatisfactory degree of compaction.

38. Trenches must be shored as required by the local agency and the State of California Division of Industrial Safety construction safety orders.

STRUCTURAL MAT FOUNDATION – WATER TANK

39. At the time we prepared this report, the project plans had not been completed, and the structure location(s) and foundation details had not been finalized. We request an opportunity to review these items during the final design stages to determine if supplemental recommendations will be required.

40. The recommendations provided herein assume all structural improvements will be sited entirely within the green shaded *"zone of weathered bedrock at the ground surface"* on Plate 3 in Appendix A. Significant design changes will be required should structural and/or ground improvements extend beyond this zone. Please refer to our Geologic Investigation Report in Appendix A for more information.

41. To reduce the potential for adverse effects to the tank(s) due to ridge top shattering, we recommend that the tanks be supported upon structural mat foundation(s) bearing upon a zone of geogrid reinforced engineered fill as detailed in the preceding sections of this report.

42. The structural mat foundation system shall be designed to span voids, withstand differential settlement, and allow the structure to move as a single unit. The loading should be kept as even as possible in all areas of the structure.

43. The structural mats should be designed and constructed to span a 3-foot diameter void appearing anywhere beneath the structure.

44. The structural mat foundations should be designed for an allowable bearing capacity of 1,500 psf (dead plus live load), which may be increased by one-third for wind or seismic loads.

45. Provided the subgrade preparation recommendations provided in this report are strictly followed, we estimate total post-construction foundation settlement of less than 1 inch, and post-construction differential foundation settlement of less than ½-inch acting over a horizontal distance of 30 feet.

46. The structural mat should be designed with a thickened edge beam that extends a minimum of 12 inches below the lowest adjacent subgrade, not including sand or gravel sections.

47. A unit modulus of subgrade reaction of 800 pounds per cubic inch may be assumed. This value is based on a 1-foot square bearing area; the subgrade modulus can be proportioned for the width of the relative footing reaction area by the expression:

$$K_o = K_1 \left[\frac{B+1}{2B}\right]^2$$

Where:

B = The effective width of the footing reaction area in feet.
K_1 = Unit modulus of subgrade reaction.
K_o = Reduced or actual modulus of subgrade reaction to use in elastic design.

48. The embedded portion of the mat may be assumed to have a lateral bearing pressure resistance value of 300 psf/ft for the section of mat embedded below the ground surface. The upper 1 foot of soil should be ignored when calculating passive soil resistance.

49. The mat may be assumed to have a resistance to lateral sliding of 0.30.

50. Where both friction and passive pressure are utilized for sliding resistance, the passive pressure should be reduced by 50%.

51. Slab thickness, reinforcement, and doweling should be determined by the project structural engineer in accordance with applicable CBC or ACI Standards.

52. If a sand layer is chosen as a cushion for slabs without floor coverings, it should consist of a clean sand. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

53. Requirements for pre-wetting of the subgrade soils prior to the pouring of the slabs will depend on the specific soils and seasonal moisture conditions and will be determined by a representative of Pacific Crest Engineering Inc. at the time of construction. It is important that the subgrade soils be properly moisture conditioned at the time the concrete is poured. Subgrade moisture contents should not be allowed to exceed our moisture recommendations for effective compaction and should be maintained until the slab is poured.

54. Utility connections to the tank(s) should be designed with flexible connections to accommodate up to 4 inches of differential movement between the geogrid reinforced tank pad and the adjacent, unreinforced native ground.

<u>PAVEMENT DESIGN</u>

55. The design of the pavement section was beyond our scope of services for this project. To have the selected pavement sections perform to their greatest efficiency, it is very important that the following items be considered:

a. Properly scarify and moisture condition the upper 8 inches of the subgrade soil and compact it to a minimum of 95% of its maximum dry density, at a moisture content of 1 to 3% over the optimum moisture content for the soil.

b. Provide sufficient gradient to prevent ponding of water.

c. Use only quality materials of the type and thickness (minimum) specified. All aggregate base and subbase must meet Caltrans Standard Specifications for Class 2 materials and be angular in shape. All Class 2 aggregate base should be ¾ inch maximum in aggregate size.

d. Compact the base and subbase uniformly to a minimum of 95% of its maximum dry density.

e. Place the asphaltic concrete only during periods of fair weather when the free air temperature is within prescribed limits by Cal Trans Specifications.

f. Porous pavement systems which consist of porous paving blocks, asphaltic concrete or concrete are generally not recommended due to the potential for saturation of the subgrade soils and resulting increased potential for a shorter pavement life. At a minimum, porous pavement systems should include a layer of Mirafi HP370 geotextile fabric placed on the subgrade soil beneath the porous paving section. These pavement systems should only be used with the understanding by the Owner of the increased potential for pavement cracking, rutting, potholes, etc.

g. Maintenance should be undertaken on a routine basis.

SURFACE DRAINAGE

56. Surface water drainage is the responsibility of the project civil engineer. The following should be considered by the civil engineer in design of the project.

57. Surface water must not be allowed to pond or be trapped adjacent to improvements. Concentrations of surface runoff should be handled by providing structures, such as paved ditches, catch basins, etc.

58. Slope failures can occur where surface drainage is allowed to concentrate on unprotected slopes. Appropriate landscaping and surface drainage control around the project area is imperative in order to minimize the potential for shallow slope failures and erosion. Stormwater discharge locations should not be located at the top or on the face of any slope.

59. Following completion of the project we recommend that storm drainage provisions and performance of permanent erosion control measures be closely observed through the first season of significant rainfall, to determine if these systems are performing adequately and, if necessary, resolve any unforeseen issues.

EROSION CONTROL

60. The surface soils are classified as having a moderate potential for erosion. Therefore, the finished ground surface should be planted with ground cover and continually maintained to minimize surface erosion. For specific and detailed recommendations regarding erosion control on and surrounding the project site, the project civil engineer or an erosion control specialist should be consulted.

PLAN REVIEW

61. We respectfully request an opportunity to review the project plans and specifications during preparation and before bidding to ensure that the recommendations of this report have been included and to provide additional recommendations, if needed. These plan review services are also typically required by the reviewing agency. Misinterpretation of our recommendations or omission of our requirements from the project plans and specifications may result in changes to the project design during the construction phase, with the potential for additional costs and delays in order to bring the project into conformance with the requirements outlined within this report. Services performed for review of the project plans and specifications are considered “post-report” services and billed on a “time and materials” fee basis in accordance with our latest Standard Fee Schedule.

V. LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. This Geotechnical Investigation was prepared specifically for the Scotts Valley Water District and for the specific project and location described in the body of this report. This report and the recommendations included herein should be utilized for this specific project and location exclusively. This Geotechnical Investigation should not be applied to nor utilized on any other project or project site.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be provided.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes. This report should not be considered valid after a period of two (2) years without our review.

5. This report was prepared upon your request for our services in accordance with currently accepted standards of professional geotechnical engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.

6. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.

APPENDIX A

Pacific Crest Engineering Geologic Investigation Report

June 3, 2025 Job # 24069

Mr. Nate Gillespie
Scotts Valley Water District
2 Civic Center Drive
Scotts Valley, CA 95066

Subject: Bethany Tank Replacement
Geologic Investigation
Scotts Valley, California
County of Santa Cruz APN 023-39-101 & 102

Dear Nate,

This letter summarizes the results of our supplemental trenching and geological investigation program for proposed tank replacement for the Bethany Tank, located approximately 600 feet east of the northern terminus of Tabor Drive in Scotts Valley, California. The site is a graded pad situated on a very narrow bedrock ridge (see Figure 1 and Plate 1).

Our investigation was primarily focused on addressing the risk related to the potential hazard of coseismic ground cracking under the proposed tank footprint and landsliding off the flanks of the ridge that might encroach upon the tank pad and undermine the proposed tank.

In addition to pursuing a recent trenching program, we have also incorporated the past trenching findings from prior geological and geotechnical engineering investigations, such as the trenching investigation by Zinn Geology and geotechnical engineering investigation by Bauldry Engineering.

SCOPE OF INVESTIGATION

Work performed during this study included:

1. A review of published and unpublished literature relevant to the proposed tank replacement on the subject property.

2. Examination and interpretation of stereo-pair vertical aerial photographs, to assess the past effects of earthquakes and storms on the subject property.

3. Preparation of a geologic map, logs of trenches excavated and a ground displacement map for the property.

4. Excavation, shoring, inspection and detailed logging of two trenches. See Plate 1 for their location and Plate 2 for graphic logs of the sidewall.

5. Analysis and interpretation of the geologic data and preparation of this report.

REGIONAL GEOLOGIC SETTING

The subject property lies on a narrow bedrock ridge crest in the Santa Cruz Mountains in the central Coast Ranges physiographic province of California. This portion of the Coast Ranges is formed by a series of rugged, linear ridges and valleys following the pronounced northwest to southeast structural grain of central California geology. The Santa Cruz Mountains are mostly underlain by a large, elongate prism of granitic and metamorphic basement rocks, known collectively as the Salinian Block. These rocks are separated from contrasting basement rock types to the northeast by the San Andreas fault and to the southwest by the Sur-Nacimiento-San Gregorio fault system. Overlying the granitic basement rocks is a sequence of dominantly marine sedimentary rocks of Paleocene to Pliocene age and non-marine sediments of Pliocene to Pleistocene age (Figure 2).

Throughout the Cenozoic Era, this portion of California has been dominated by tectonic forces associated with lateral or "transform" motion between the North American and Pacific lithospheric plates, producing long, northwest-trending faults such as the San Andreas and San Gregorio, with horizontal displacements measured in tens to hundreds of miles (Figure 3). Accompanying the horizontal (strike-slip) movement of the plates have been episodes of compressive stress, reflected by repeated episodes of uplift, deformation, erosion and subsequent redeposition of sedimentary rocks. Near the crest of the Santa Cruz Mountains, this tectonic deformation is most evident in the sedimentary rocks older than the middle Miocene, and consists of steeply dipping folds, overturned bedding, faulting, jointing and fracturing. The Loma Prieta earthquake of 1989 and its aftershocks are the most recent reminders of the geologic unrest in the region.

The Quaternary history of the Santa Cruz Mountains has been dominated by landslide-related processes. Historical accounts and geologic studies on the San Andreas earthquake of 1906 and the Loma Prieta earthquake of 1989 indicate that there is a strong correlation between major earthquakes and the resulting landslides, earth flows, debris flows and ground cracking in this region.

REGIONAL SEISMIC SETTING

California's broad system of strike-slip faulting has had a long and complex history. Some of these faults present a seismic hazard to the subject property. The most important of these are the San Andreas and Zayante(-Vergeles) fault zones (Figures 2 and 3). These faults are either active or considered potentially active (Hall et al., 1974; Petersen et al., 1996; Working Group On Northern California Earthquake Potential [WGONCEP], 1996).

Topography

The subject property is located on a knife edged ridge crest with very steep flanks. The proposed tank area lies upon an entirely graded area consisting mostly of cut with fill side cast off the edge of the ridge crest where the top of the ridge transitions topographically to steep flanks.

Earth Materials

The local geologic map by McLaughlin et al. (2001; see Figure 4) shows the property as being underlain by Purisima Formation, located on the northern limb of the Scotts Valley syncline. There is no portrayal of bedding on the regional maps and we were unable to ascertain the strike and dip of bedding in the bedrock during our trenching program due to its massive appearance.

The Purisima Formation bedrock encountered in the trenches was a fine grained sandstone, yellow (10YR 7/6), dry, mottled with oxidation and lysengang banding, with undeformed fracture spacing 2-inches to massive, and where deformed ¼" to 12" fracture spacing. The fractures marking deformation were dilated and coated with pedogenic clays, sequioxides and sometimes rootlet mats. Overall the bedrock was slightly to moderately weathered, grading upward into severely weathered regolith.

The top of the ridge has been graded to create a flat pad, so the original native pedogenic soil was been scraped off and cast off the margins of the ridge. A thin blanket of fill that thickens to a wedge shape on the edges of the pad above the steep flanks from those past grading operations is present across the site.

Both the bedrock and the soil have been cut by extensional ground cracking, which is marked by soil filled fissures. Towards the edges of the ridge crest the cracking became more prevalent and presented as a zone of intense extensional dilation, which is an indication of incipient landsliding as the earth materials at the top of the slope begin to mobilize and move downslope.

GEOLOGIC HAZARDS

In our opinion, the primary geologic hazards that could potentially affect the proposed tank site 1) coseismic ground cracking due to ridge-top spreading, 2) landsliding off the steep flanks of the ridge and 3) intense seismic shaking.

Coseismic Ground Cracking

Ground cracks are commonly generated during earthquakes due to stresses from strong shaking. Structures may be detrimentally affected by development of significant ground cracks beneath foundations. Coseismic ground cracks are typically associated with youthful hummocks, swales, benches and closed depressions on ridgecrests (Beck, 1968; Tabor, 1971; Radbruch-Hall et al., 1977; Bovis, 1982; Savage and Varnes, 1987; Thorsen, 1989; Varnes et al., 1989). Detailed studies were performed on ground cracking observed in the nearby Summit area after the 1989 Loma Prieta earthquake (Griggs and Associates, 1990; Hart et al., 1990; Harp, 1998; Nolan and Weber, 1998).

The study by Nolan and Weber (1998) involved excavation, inspection and logging of two trenches through ground cracks observed at the ground surface after the 1989 Loma Prieta earthquake in the Summit area. They concluded that coseismic ground cracks observed after the 1989 Loma Prieta earthquake with surface expressions of at least 2 to 3 inches in width at the ground surface were recurrent phenomena and could be located according to recognizable preexisting surficial features. Trench logging is important in locating recurring ground cracks in the subsurface that do not display a discrete surface expression.

Nolan and Weber (1998) also noted, however, that their case study history of ground cracking and damage to residences resulting from the 1989 Loma Prieta Earthquake indicates that ground cracks exhibiting small earthquake-related displacements (less 2 inch vertical and 2 inches horizontal) were commonly but not necessarily associated with subsurface evidence of earlier episodes of movement. Therefore, the predictive reliability for small-scale cracks is lower.

The proposed tank site sits atop an a bedrock ridge that is clearly cut by coseismic ground cracking related to ridge-top spreading occurring from the center out (see Plate 2). In our opinion significant coseismic ground cracks, with at least 2 inches of horizontal and vertical displacement, are likely to affect the proposed tank site within its design life. This corresponds to a greater than lowest possible risk as outlined in the risk tables in Appendix B. The risk with respect to ground cracking should be mitigated and reduced to the lowest possible risk level through a combination of tank placement and foundation design to accommodate the anticipated coseismic ground cracking.

Landsliding

The flanks of the ridge crest appear to have failed in the past, both historically and in geological time. A portion of the southeastern ridge slid at some point in the past, based on prior trenching by Zinn Geology at the northeastern edge of the property, resulting in a cumulative ground displacement of slightly less than 4 feet vertical and 1 ½ feet horizontal.

The steep slope northwest of the existing tank appears to have failed in the 1980's, causing extensional cracks to appear under the existing tank foundation (M. Jacobs And Associates, 1985). The cracking appears to have been related to incipient landsliding and appears to have begun to undermine the foundation of the tank. A pin pier wall was subsequently installed to protect the tank foundation from continued undermining.

It is important to note that landsliding off of the steep flanks of the ridge is a separate process from the coseismic cracking hazard. Both processes can cause extensional cracking to occur under the tank foundation, but the landsliding hazard can be triggered by excessive rainfall as well as seismic shaking and may result in cumulative horizontal and vertical displacements of several feet.

We have depicted the landsliding hazard as red shaded zone on the attached Plate 3. As noted above, cumulative horizontal and vertical displacements of several feet may occur in this zone. The risk to the proposed tank foundation related to future landsliding is greater than a lowest possible risk as outlined in Appendix B if left unmitigated. This risk should be mitigated through tank placement, foundation design or a combination of the two schemes in order to lower the risk to lowest possible risk.

RECOMMENDATIONS

1. We recommend that the foundation for the proposed tank at be designed to accommodate the corresponding horizontal and vertical displacements stipulated on Plate 3. The zone shaded orange and labeled “Zone Of Soil Loss” is an area where the soil is loose and creeping, so the tank foundation should be designed for loss of that soil down to the bedrock in that area.

2. Seismic shaking values for any structures designed on the property should at least adhere to the minimum prescriptive design values outlined in the current California Building Code. The seismic shaking values should be developed by the Project Geotechnical Engineer of Record as part of their soils report for the design of structures.

3. Drainage plans should be developed for the tank site, with particular attention paid to guiding drainage away from the tank and the steep slopes that flank it. We request the opportunity to review any forthcoming drainage plans for consistency with our geologic findings and recommendations. The Project Civil Engineer Of Record should also consult the Santa Cruz County erosion control ordinances for additional requirements and restrictions. The control of storm water on the site is essential to prevent continued landsliding off of the steep flanks.

4. We recommend that our firm be provided the opportunity for a review of any forthcoming reports, designs and specifications by the project geotechnical engineer, structural engineer, architect and landscaper, in order that our recommendations may be properly interpreted and implemented in the design and specification. If our firm is not accorded the privilege of making the recommended review we can assume no responsibility for misinterpretation of our recommendations.

INVESTIGATIVE LIMITATIONS

1. This geological report was prepared specifically for for this specific project and location described in the body of this report. This report and the recommendations included herein should be utilized for this specific project and location exclusively. This Geological Investigation should not be applied to nor utilized on any other project or project site.

2. The recommendations of this report are based upon the assumption that the geological conditions do not deviate from those disclosed in this report. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be provided.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes. This report should not be considered valid after a period of two (2) years without our review.

5. This report was prepared upon your request for our services in accordance with currently accepted standards of professional geological practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.

6. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.

If you have any questions or comments regarding this report, please contact us at your earliest convenience.

Sincerely,

PACIFIC CREST ENGINEERING INC.

CERTIFIED ENGINEERING GEOLOGIST
ERIK N. ZINN
No. 2139
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
ERIK N. ZINN
No. 6854
STATE OF CALIFORNIA

Erik N. Zinn
Principal Geologist
P.G. #6854, C.E.G. #2139

Attachments: References
Appendix A – Figures
Appendix B - Scale of Acceptable Risks
Plate 1 – Geologic Site Map
Plate 2 – Trench Logs
Plate 3 – Design Ground Deformation Map

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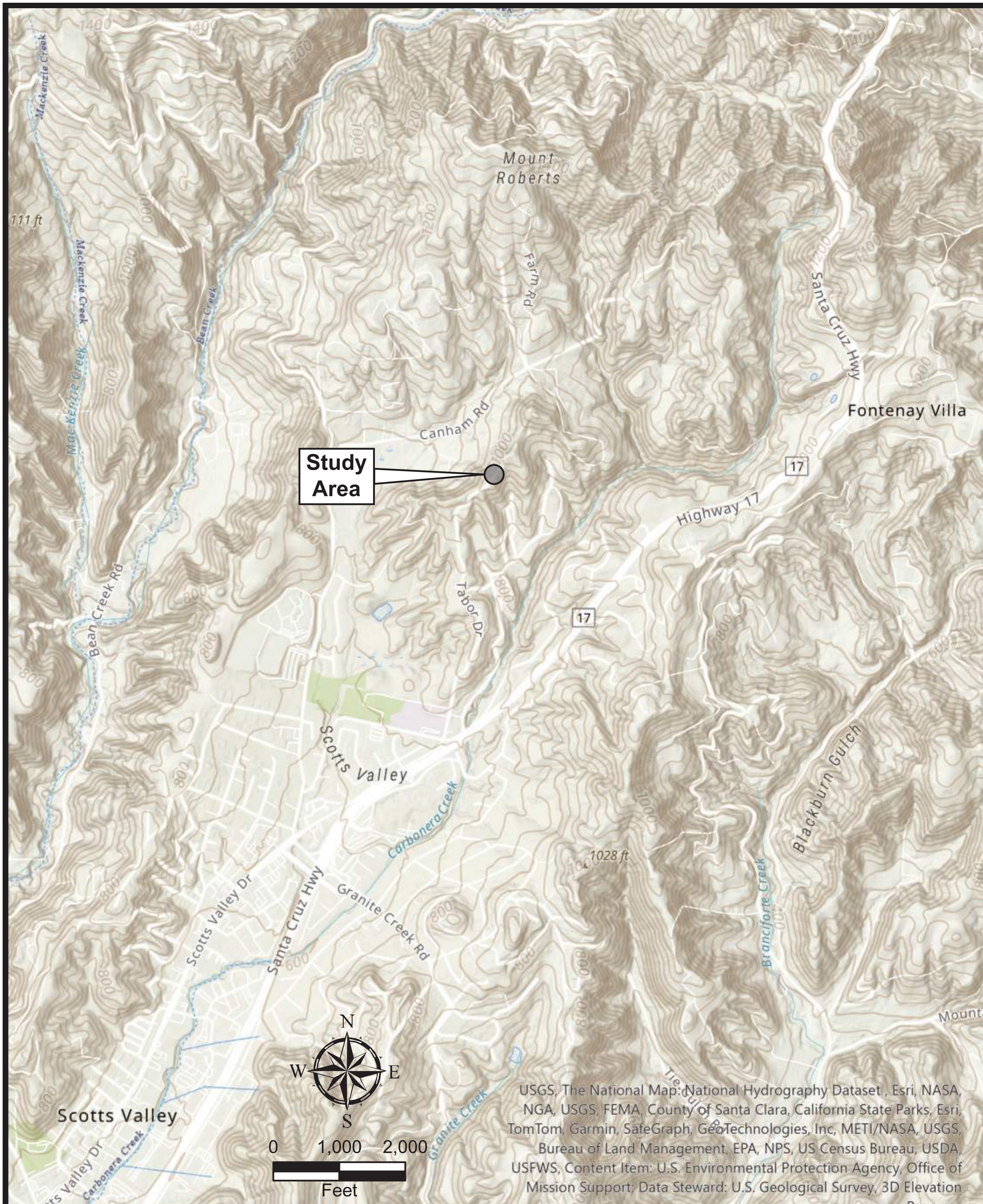
APPENDIX A

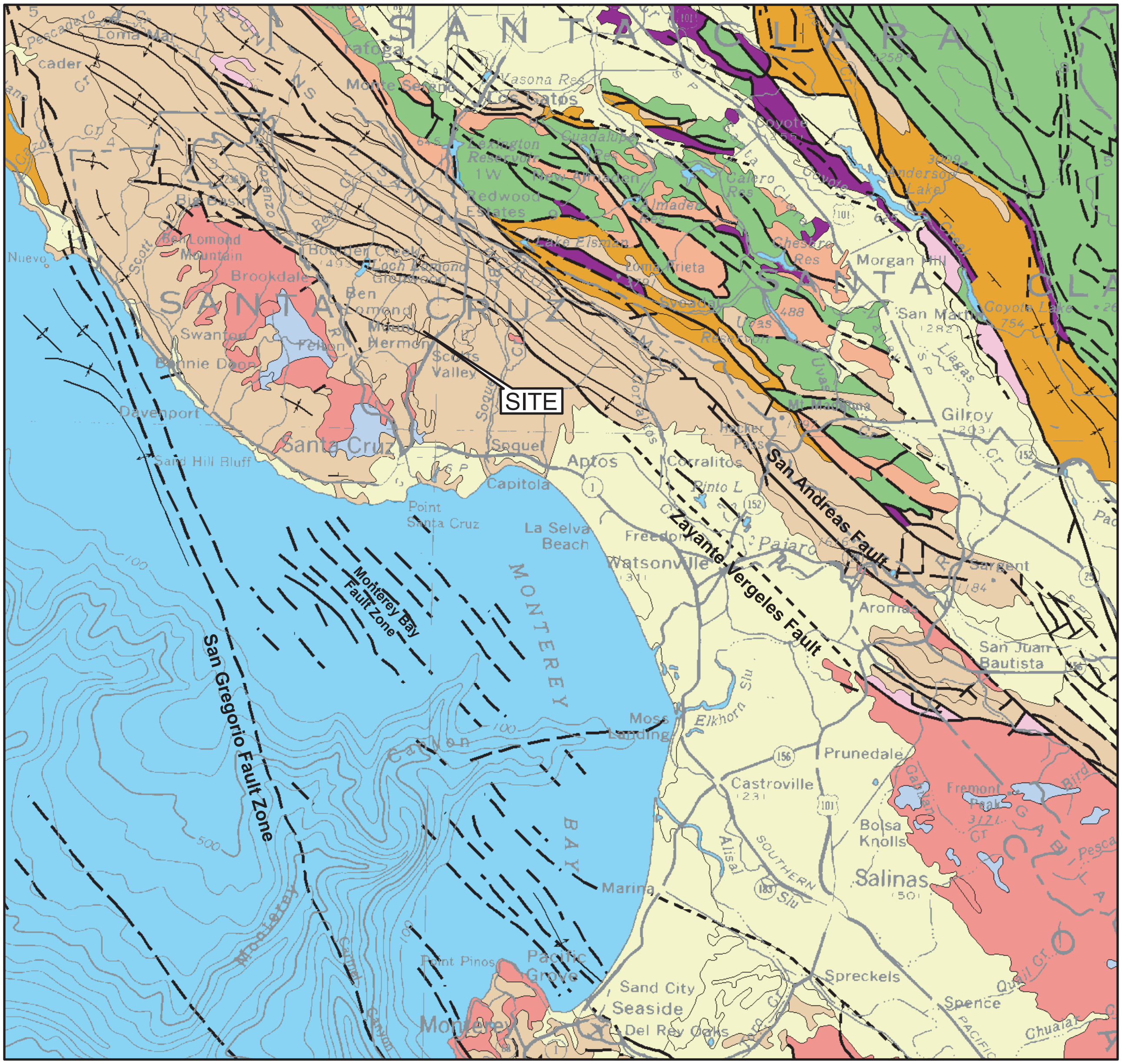
FIGURES



Pacific Crest ENGINEERING INC

Topographic Index Map
Bethany Tank Replacement
Scotts Valley
California

FIGURE #
1
JOB #
24069



Reference: Jennings, C.W., 1977, Geologic Map of California: California Department of Conservation, Division of Mines and Geology, scale 1:750,000.
Digital Data: Saucedo, G.J., Bedford, D.R., Raines, G.L., Miller, R.J., and Wentworth, C.M., 2000, GIS Data for the Geologic Map of California: California Department of Conservation, Division of Mines and Geology, CD-ROM 2000-007, ver. 2.0.

EXPLANATION

Geologic Units

- Quaternary Deposits
- Quaternary Volcanics
- Tertiary Sedimentary Rocks
- Tertiary Volcanic Rocks
- Pre-Tertiary Sedimentary Rocks
- Pre-Tertiary Volcanic Rocks
- Grantic Intrusive Rocks
- Franciscan Complex
- Ultramafic Rocks
- Pre-Tertiary Metamorphic Rock
- Pre-Cambrian Metamorphic and Igneous Rocks

Symbols

- contact
- fault, certain
- fault,approx. located
- fault, concealed or inferred
- anticline
- monocline
- syncline

N
W E
S

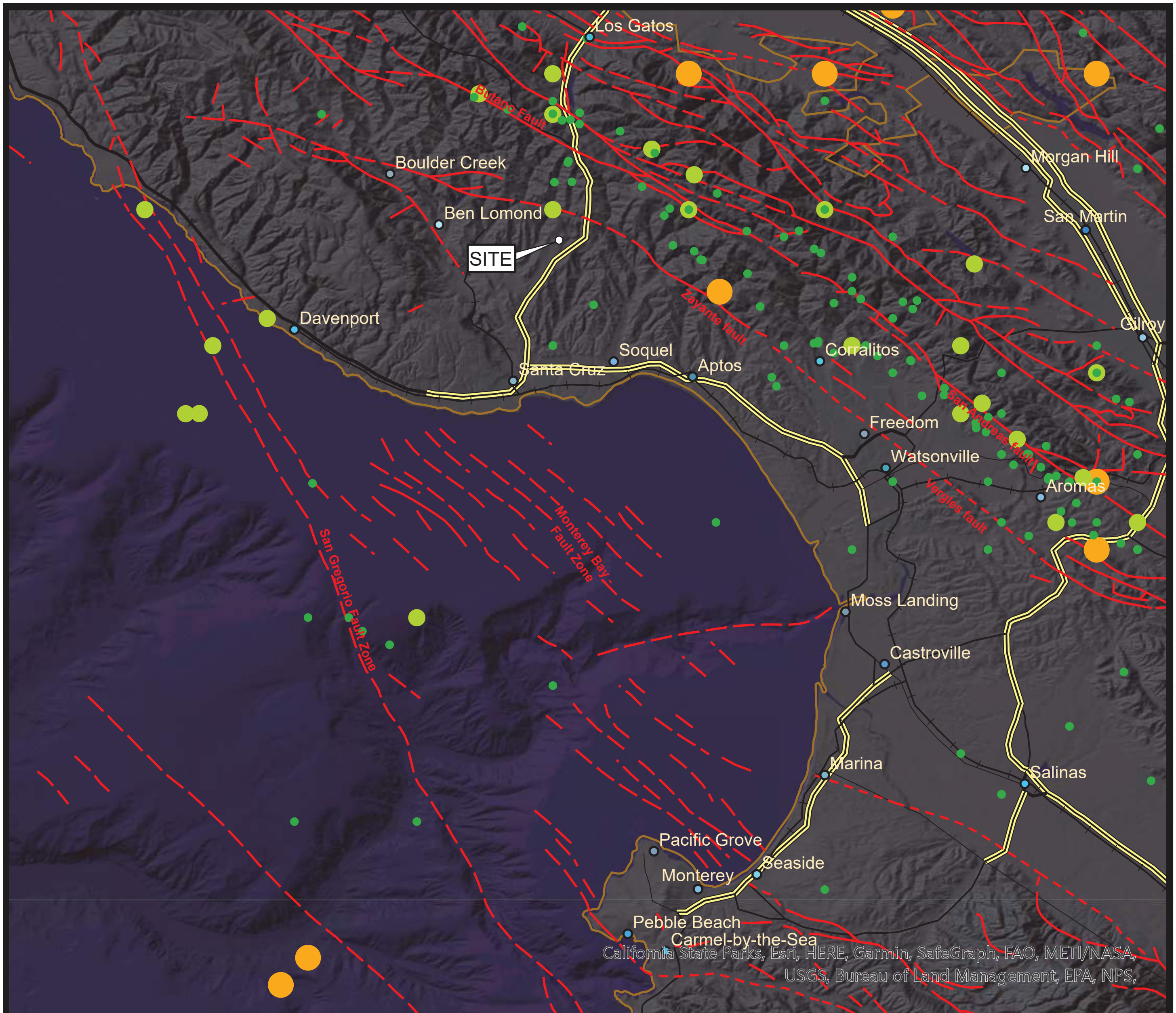
SCALE 1:500,000

10 Miles 0

Pacific Crest ENGINEERING INC

Regional Geologic Map
Bethany Tank Replacement
Scotts Valley
California

FIGURE #
2
JOB #
24069



Seismicity Information: Magnitude 4 and greater earthquakes, compiled from various sources, 1769 to 2000; available at www.consrv.cagov/CGS/rghm/quakes/cgs2000_fnl.txt

Fault Information: Jennings, C.W., 1977, Geologic map of California: California Department of Conservation, Division of Mines and Geology, scale 1:750,000

EXPLANATION

Symbols	**Earthquake Magnitude**
fault, certain	4.0 to 4.99
fault,approx. located	5.0 to 5.99
fault, concealed or inferred	6.0 to 6.99
	7.0 +

N W E S

SCALE 1:500,000

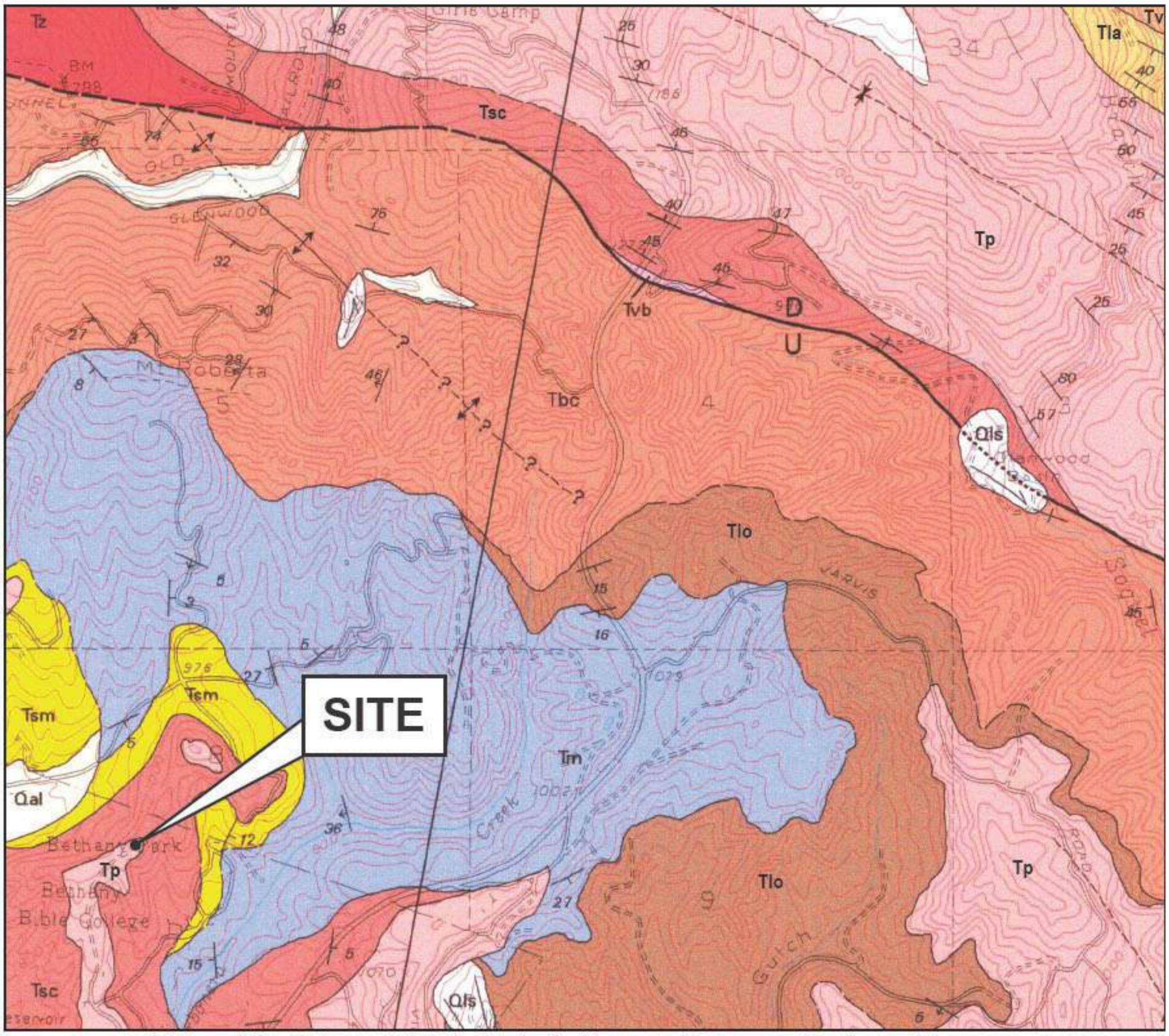
10 5 0 10

Miles

Pacific Crest ENGINEERING INC

Regional Seismicity Map
Bethany Tank Replacement
Scotts Valley
California

FIGURE #
3
JOB #
24069



BASE MAP: McLaughlin, R.J. et. al, 2001, Geologic Maps and Structure Sections of the Southwestern Santa Clara Valley and Southern Santa Cruz Mountains, Santa Clara and Santa Cruz Counties, California, Sheet 2: Laurel Quadrangle, U.S. Geological Survey, Miscellaneous Field Studies Map MF-2373, scale 1:24,000.

Explanation

UNITS

- Qls - Landslide deposits
- Qal - Alluvium
- Tp - Purisima formation
- Tsc - Santa Cruz mudstone
- Tsm - Santa Margarita sandstone
- Tm - Monterey formation
- Tlo - Lompico formation
- Tla - Lambert Shale
- Tv - Vaqueros formation
- Tvb - Basalt flows
- Tz - Zayante sandstone
- Tbc - Conglomerate

SYMBOLS

- contact, certain
- D / U fault, certain
  - D - downthrown block
  - U - upthrown block
- 25 bedding attitude
- 25 approximate bedding attitude
- horizontal bedding
- vertical bedding attitude
- syncline
- landslide deposit arrows show direction of movement

N E S W

SCALE 1:24,000

1000 0 1,000 2,000 feet

Pacific Crest ENGINEERING INC

Local Geologic Index Map
Bethany Tank Replacement
Scotts Valley
California

FIGURE #
4
JOB #
24069

APPENDIX B
SCALE OF ACCEPTABLE RISKS FROM GEOLOGIC HAZARDS

SCALE OF ACCEPTABLE RISKS FROM SEISMIC GEOLOGIC HAZARDS		
Risk Level	Structure Types	Extra Project Cost Probably Required to Reduce Risk to an Acceptable Level
Extremely low[1]	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	No set percentage (whatever is required for maximum attainable safety).
Slightly higher than under "Extremely low" level.[1]	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	5 to 25 percent of project cost.[2]
Lowest possible risk to occupants of the structure.[3]	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	5 to 15 percent of project cost.[4]
An "ordinary" level of risk to occupants of the structure.[3,5]	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1 to 2 percent of project cost, in most cases (2 to 10 percent of project cost in a minority of cases).[4]

Failure of a single structure may affect substantial populations.
These additional percentages are based on the assumptions that the base cost is the total cost of the building or other facility when ready for occupancy. In addition, it is assumed that the structure would have been designed and built in accordance with current California practice. Moreover, the estimated additional cost presumes that structures in this acceptable risk category are to embody sufficient safety to remain functional following an earthquake.
Failure of a single structure would affect primarily only the occupants.
These additional percentages are based on the assumption that the base cost is the total cost of the building or facility when ready for occupancy. In addition, it is assumed that the structures would have been designed and built in accordance with current California practice. Moreover the estimated additional cost presumes that structures in this acceptable-risk category are to be sufficiently safe to give reasonable assurance of preventing injury or loss of life during and following an earthquake, but otherwise not necessarily to remain functional.
"Ordinary risk": Resist minor earthquakes without damage: resist moderate earthquakes without structural damage, but with some non-structural damage; resist major earthquakes of the intensity or severity of the strongest experienced in California, without collapse, but with some structural damage as well as non-structural damage. In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. (Structural Engineers Association of California)

Source: *Meeting the Earthquake,* Joint Committee on Seismic Safety of the California Legislature, Jan. 1974, p.9.

SCALE OF ACCEPTABLE RISKS FROM NON-SEISMIC GEOLOGIC HAZARDS[6]		
Risk Level	Structure Type	Risk Characteristics
Extremely low risk	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	Failure affects substantial populations, risk nearly equals nearly zero.
Very low risk	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	Failure affects substantial populations. Risk slightly higher than 1 above.
Low risk	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	Failure of a single structure would affect primarily only the occupants.
"Ordinary" risk	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	Failure only affects owners /occupants of a structure rather than a substantial population. No significant potential for loss of life or serious physical injury. Risk level is similar or comparable to other ordinary risks (including seismic risks) to citizens of coastal California. No collapse of structures; structural damage limited to repairable damage in most cases. This degree of damage is unlikely as a result of storms with a repeat time of 50 years or less.
Moderate risk	Fences, driveways, non-habitable structures, detached retaining walls, sanitary landfills, recreation areas and open space.	Structure is not occupied or occupied infrequently. Low probability of physical injury. Moderate probability of collapse.

[6] Non-seismic geologic hazards include flooding, landslides, erosion, wave runup and sinkhole collapse

EXPLANATION

EARTH MATERIALS

af/Tp Artificial fill over Purisima Formation bedrock

Tp Purisima Formation bedrock

SYMBOLS

Earth materials contact

Base Map: Alpha Land Surveys Inc.; Topographic Map - Bethany Water Tank - Tabor Drive - Scotts Valley; Sheet 1; dated September 9, 2024

PROFESSIONAL GEOLOGIST ERIK N. ZINN No. 6854 STATE OF CALIFORNIA

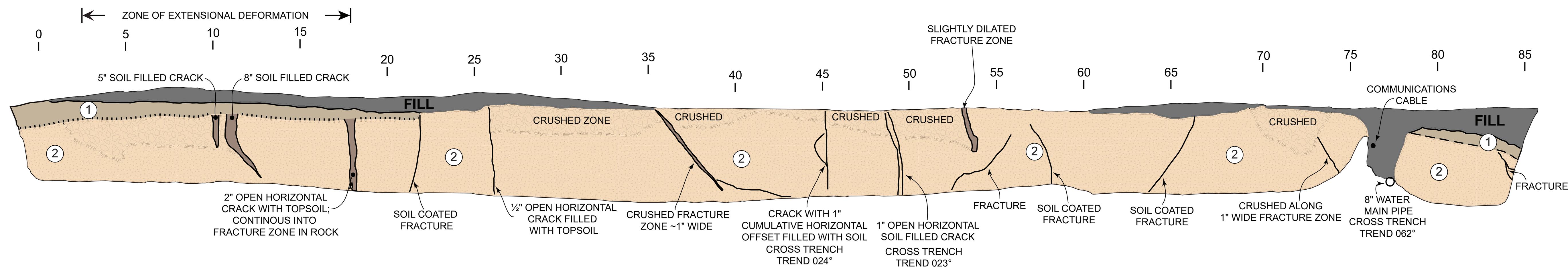
CERTIFIED ENGINEERING GEOLOGIST ERIK N. ZINN No. 2139 STATE OF CALIFORNIA

Pacific Crest
ENGINEERING INC

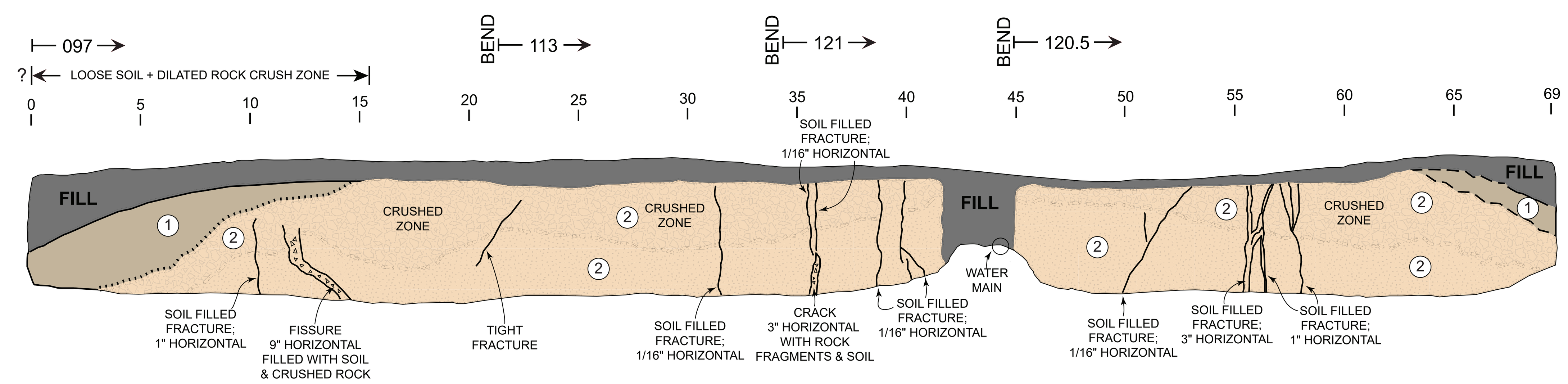
BETHANY TANK SITE PLAN
Bethany Tank Replacement
APN: 023-39-101 & 102
Scotts Valley, California

Date: 28 January 2025	Revised:
Job #24069	
Scale: 1"=10'	**Plate 1**
Drawn by: MP/enz	

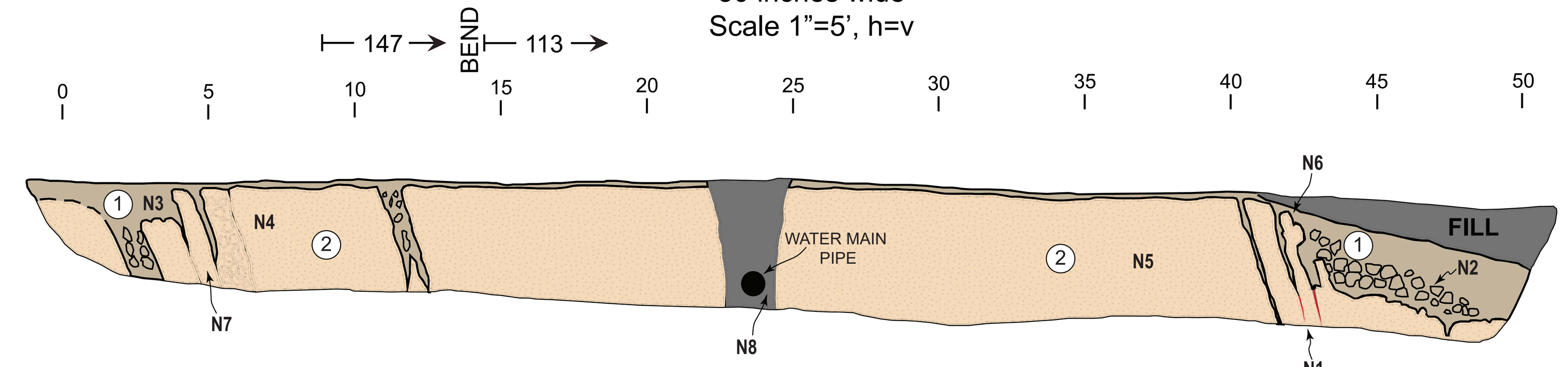
Trench 1
Northeast Wall Logged
30 inches wide
Scale 1"=5', h=v



Trench 2
North Wall Logged
36 inches wide
Scale 1"=5', h=v



Zinn Geology Trench
North Wall Logged
30 inches wide
Scale 1"=5', h=v



EXCAVATED AND LOGGED SEPTEMBER 2, 2010
BY ZINN GEOLOGY

EXPLANATION

2 Unit of earth materials (see "UNIT DESCRIPTIONS")

Sharp Contact

Diffuse Unit Contact

Gradational Unit Contact (over 0.5 feet)

Sandstone clasts

TRENCH UNIT DESCRIPTIONS:

UNIT 1: TOPSOIL - sandy SILT with gravel: grayish brown (10YR 5/2) to brown (10YR 4/3), dry, weak crumb structure, sand is fine grained, quartz-rich, gravel is composed of very fine grained sandstone, abundant roots and rootlets, sharp but gradational contact over 3 inches in extensional deformation zone, marked by change in color and composition

UNIT 2: Purisima formation - Sandstone - yellow (10YR 7/6), dry, mottled with oxidation and lysengang banding, very fine grained, undeformed fracture spacing 2-inches to massive, where deformed ¼" to 12" fracture spacing, where deformed fractures can be dilated and coated with pedogenic clays, sequioxides and sometimes rootlet mats slightly to moderately weathered grading upward into severely weathered regolith

NOTES

- N1 Landslide formed along extensional cracking, might have occurred in one event, at least 2½' to 3' of vertical seperation, cumulative horizontal offset is approximately 1' seperation in this zone
- N2 Stone line from collapsed rock face
- N3 Open fracture zone, approximately 1' of vertical offset with approximately 1' horizontal cumulative offset
- N4 Open fracture zone, no measurable vertical offset, approximately 1' horizontal seperation
- N5 Fracture: 150/77° SW
- N6 Escarpment cross-trench: 022/85-90°
- N7 Fracture: 208/78° E
- N8 190° is cross-trench trend for water pipe main

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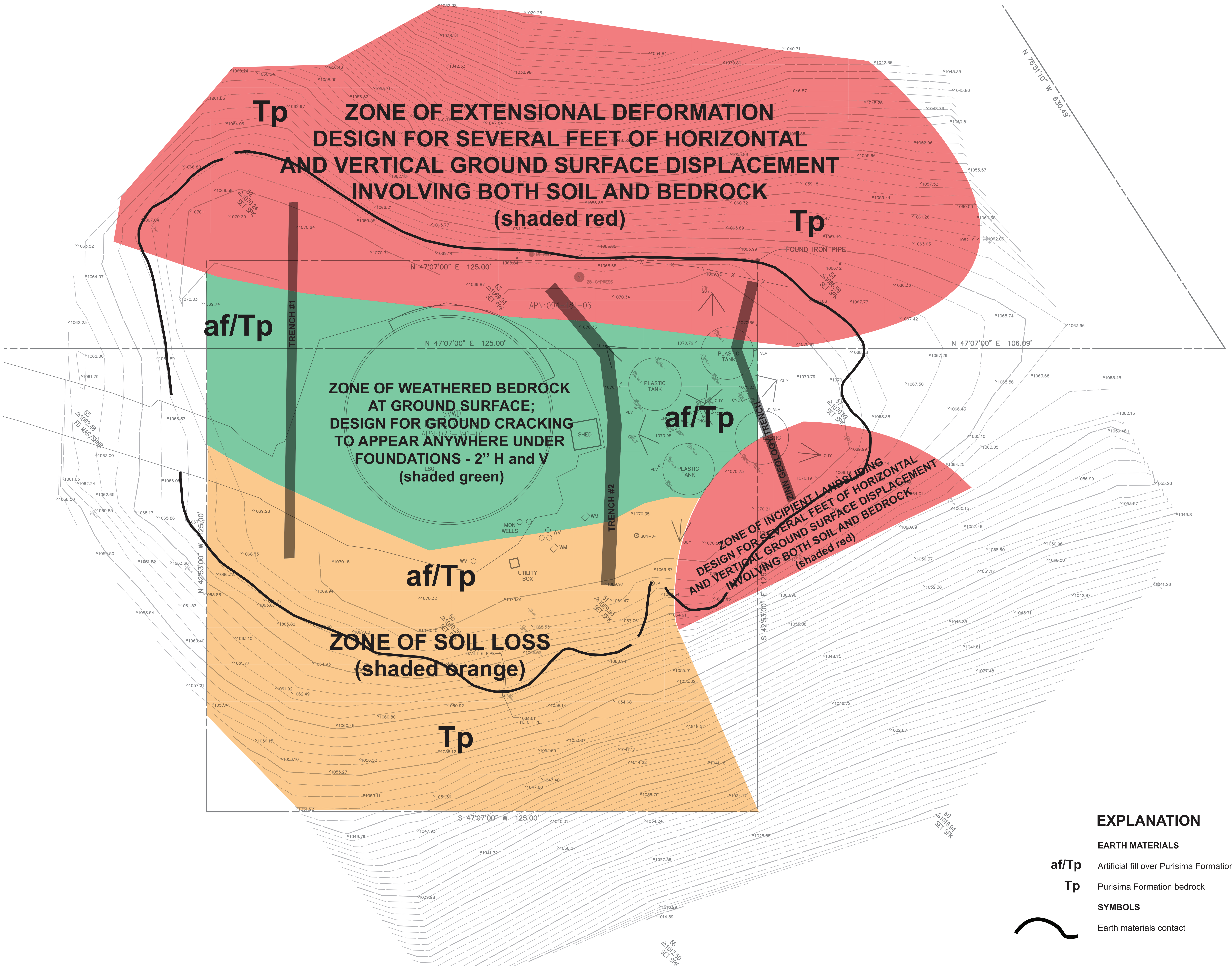
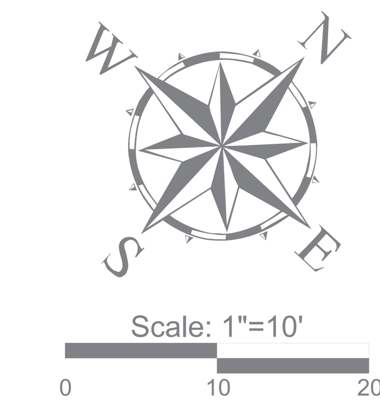
Pacific Crest
ENGINEERING INC

TRENCH LOGS
Bethany Tank Replacement
APN: 023-39-101 & 102
Scotts Valley, California

Date: 15 May 2025	Revised:
Job #24069	
Scale: 1"=5', H=V	**Plate 2**
Drawn by: ENZ/mp	





EXPLANATION

EARTH MATERIALS

af/Tp Artificial fill over Purisima Formation bedrock

Tp Purisima Formation bedrock

SYMBOLS

Earth materials contact

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Pacific Crest
ENGINEERING INC

DESIGN GROUND DEFORMATION MAP
Bethany Tank Replacement
APN: 023-39-101 & 102
Scotts Valley, California

Date: 15 May 2025 Revised:
Job #24069
Scale: 1"=10'
Drawn by: ENZ/enz

Plate 3

Base Map: Alpha Land Surveys Inc.; Topographic Map - Bethany Water Tank - Tabor Drive - Scotts Valley; Sheet 1; dated September 9, 2024

APPENDIX B

Bauldry Engineering 2010 Geotechnical Investigation Report

GEOTECHNICAL INVESTIGATION
FOR
BETHANY WATER TANK UPGRADE PROJECT
TABOR DRIVE
SCOTTS VALLEY, CALIFORNIA

FOR
SCOTTS VALLEY WATER DISTRICT
SCOTTS VALLEY, CALIFORNIA

BY
BAULDRY ENGINEERING, INC.
CONSULTING GEOTECHNICAL ENGINEERS
1020-SZ932-H73
JULY 2010

1020-SZ932-H73
July 30, 2010

Scotts Valley Water District
2 Civic Center Drive
P.O. Box 660006
Scotts Valley, CA 95067-0006

Attention: Colin Smith

Subject: Geotechnical Investigation
Bethany Water Tank Upgrade Project
Ridgecrest Drive
Scotts Valley, California

Dear Mr. Smith,

In accordance with your authorization, we have performed a geotechnical investigation for your proposed water tank upgrade project located in Scotts Valley, California.

The accompanying report presents our conclusions and recommendations as well as the results of the geotechnical investigation on which they are based. The conclusions and recommendations presented in this report are contingent upon our review of the plans during the design phase of the project, and our observation and testing during the construction phase of the project.

If you have any questions concerning the data, conclusions, or recommendations presented in this report, please call our office.

Very truly yours,

Bauldry Engineering, Inc.

Brian D. Bauldry
Principal Engineer
G. E. 2479
Exp. 12/31/10

C:\PubData\Projects\2010\1020-SZ932-H73 - Bethany Water Tank - Scotts Valley Water District\1020 gi.doc
Copies: 4 to Colin Smith – Scotts Valley Water District

1020-SZ932-H73
July 30, 2010

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July 30, 2010

GEOTECHNICAL INVESTIGATION

PURPOSE OF INVESTIGATION (ok)

The purpose of our investigation was to explore the subsurface conditions at the Bethany water tank site and based on our findings provide geotechnical engineering design and construction recommendations for the proposed upgrade to the existing water tank. Additionally, our investigation was directed to help establish potential geotechnical and geologic hazards that would be associated with replacing the existing tank with a larger diameter tank at a future date.

SCOPE OF SERVICES (OK)

This report describes the geotechnical investigation and presents results, including recommendations, for the proposed upgrade of the existing tank and provides a preliminary assessment of potential geotechnical and geologic hazards associated with replacing the existing tank with a larger diameter tank. If the proposed design and construction differ significantly from that planned at the time this report was written, the conclusions and recommendations provided in this report are null and void unless the changes are reviewed by our firm and the conclusions and recommendations presented in this report are modified, or verified, in writing.

Our scope of services for this project has consisted of:

1. Discussions with you and Erik Zinn of Zinn Geology.
2. Review of the following documents, maps and reports:
   a. The Topographic Survey Map of the Bethany Tank Site prepared by Atlas Land Survey, Inc. and dated 4/16/10.
   b. The geotechnical investigation report for stabilizing the slope adjacent to the existing water tank titled “Soil Investigation for Bethany Tank, Scotts Valley, California” prepared by M. Jacobs & Associates and dated November 4, 1985.
   c. The supplemental geotechnical investigation report for stabilizing the slope adjacent to the existing water tank prepared by M. Jacobs & Associates and dated January 2, 1986.
   d. The geotechnical aspects of the structural calculations for a “buried” pier retaining wall (soldier piers) prepared by Donald C. Urfer and Associates and dated January 2, 1986.
   e. The “Bethany Tank Slide Repair” plan prepared by Donald C. Urfer and Associates and dated February 4, 1986.
   f. Geologic Map of Santa Cruz County, California, Brabb, 1989.
   g. Preliminary Landslide Deposits in Santa Cruz County, California, Cooper-Clark, 1975.
   h. Map Showing Quaternary Geology and Liquefaction Potential of Santa Cruz County, California, Dupré, 1975.
   i. Map Showing Faults and Their Potential Hazards in Santa Cruz County, California; Hall, Sarna-Wojcicki, Dupré, 1974.
   j. Santa Cruz County’s online Geographic Information System “GISWEB Interactive Mapping Application” http://gis.co.santa-cruz.ca.us/internet/wwwgisweb/viewer.htm

3. The drilling and logging of 3 test borings.
4. Laboratory analysis of retrieved soil samples.
5. Engineering analysis of the field and laboratory results.
6. Preparation of this report documenting our investigation and presenting recommendations for the design of the project.

SITE DESCRIPTION

Location (OK)

The subject water tank is situated at the eastern terminus of Tabor Drive in Scotts Valley, California. The tank is located on APN 094-181-06 and APN 023-391-01.

Site Topography and Setting (OK)

The Bethany water storage tank is located on a relatively narrow ridge crest that is accessed from the end of Tabor Drive. The existing tank is set on a cut pad. The tank appears to be underlain entirely by cut. Spoils from the cut appear to have been loosely broadcast along both sides of the ridge crest. The flanks of the ridge descend to the northwest and southeast with a relatively steep gradient. It is our understanding that an unstable slope adjacent to and on the northwest side of the existing tank was stabilized with a buried soldier pier retaining wall.

Existing Tank

The existing water tank is a 400,000 gallon welded steel tank. The tank is reported to have a diameter of 46 feet 2 inches and a height of 32 feet 2 inches.

It is our understanding that as-built documentation is not available for either the existing tank or the existing slide repair. Therefore the following assumptions are based on visual observations, discussions with the Scotts Valley Water Department and available documentations and will need to be confirmed either prior to or during construction.

a. The existing documents provided our office indicate that the existing tank is supported by a shallow ring foundation in conjunction with a interior slab. It is our understanding that the ring foundation is most likely on the order of 18 inches wide and extends approximately 2 feet below the ground surface. We anticipate the base of the ring foundation is either shallowly or directly underlain by sandstone bedrock.

b. The bottom of the tank is assumed to be underlain by a layer of oil soaked sand and/or engineered fill with the sand/fill directly underlain by sandstone bedrock. The depth to bedrock beneath the tank is assumed to be no deeper than 5 feet below the base of the footing.

c. The existing tank is not bolted to the ring foundation.

The April 2010 survey of the existing tank by Atlas Land Surveying Inc. indicates that the ring foundation is essentially level. The survey indicates that the top of the foundation along the northwest side of the tank is approximately 0.11 inches lower than the east side. This may be due to either construction tolerance or differential settlement. We observed no significant foundation distress. The existing foundations appears to be performing adequately.

Existing Slide Repair – Northwest Slope (OK)

It is our understanding that as-built documentation is not available for the existing slide repair. Therefore the following assumptions are based on visual observations, discussions with the Scotts Valley Water Department and available slide repair documentations.

a. Seventeen 24-inch diameter drilled cast-in-place reinforced concrete piers were drilled between the northwest side of the existing tank and the top of the slope. The piers were designed to be a maximum of 6 inches apart (side to side) and run parallel with and 4 feet from the perimeter of the tank. The piers were designed to be connected by a 20-inch wide, 18-inch deep reinforced concrete capbeam.

b. The piers were designed as a "buried" retaining wall that stabilized the up to 7 feet of the upper slope. The piers were designed to be embedded a minimum of 17 feet into the hard massive siltstone that underlay the retained upper 7 feet of the slope.

c. Following the construction of the piers, the existing fill under the tank and between the tank and the piers was to be "strengthened and stabilized" using "lense reinforcing grouting".

d. The construction was to be monitored by the project geotechnical engineer including the pier drilling. The geotechnical engineer was to determine the depth to siltstone, the extent of the caisson placement and the adequacy of the lense reinforcing grouting.

e. Following completion of the above, the work area was to be covered with new asphalt pavement, which would conceal the "buried" pier retaining wall and capbeam.

Proposed Project
The proposed project consists of upgrading the existing water tank to current seismic code requirements. Additionally, a new roof is proposed. It is our understanding that the new roof will be supported by a new spread footings arranged radially around the interior of the tank approximately halfway between the center of the tank and the tank's existing perimeter footing.

Earth Materials
The project site is mapped on the USGS Geologic Map of Santa Cruz County (Brabb 1989) as being at the boundary of the Santa Cruz Mudstone (Tsc, Upper Miocene) and the Purisima Formation (Tp). The Purisima Formation typically consists of siltstone interbedded with fine grained sandstone. Santa Cruz Mudstone typically consists of medium to thickly bedded, faintly laminated, pale yellowish brown mudstone with blocky weathering. The bedrock encountered in our test borings appears to be consistent with the description for Santa Cruz Mudstone. The earth materials that overlaid the bedrock were consistent with earth materials derived from Santa Cruz Mudstone.

Undocumented artificial fill consisting of a mix of bedrock fragments and residual soils was encountered in Boring No. 2 to a depth of 6½ feet below existing grade. The existing fill is generally loose and considered to be an unstable and compressible non-engineered fill.
reported in the 2 borings drilled by Myron Jacobs & Associates on October 23, 1985 nor was groundwater reported in the 3 borings drilled by Myron Jacobs & Associates on December 20, 1985.

The groundwater conditions described in this report reflect the conditions encountered during the drilling investigations listed above at the specific locations drilled. It must be anticipated that the perched and regional groundwater tables may vary with location and will fluctuate with variations in rainfall, runoff, irrigation and other changes to the conditions existing at the time the drilling was performed.

GEOTECHNICAL HAZARDS (OK)

The Geologic Report by Zinn Geology should be consulted for a detailed discussion of the geologic setting, the seismicity, and the expected geologic and seismic hazards at the site.

Seismic Shaking and AWWA Site Class

For a detailed discussion and assessment of seismic shaking, including a site specific response spectrum, refer to the seismic shaking hazard analysis report prepared for the Bethany tank site by Zinn Geology.

The site class provided below is based on standard penetration values ("N" Values) obtained from our test borings and the procedures outlined in Section 13.2.4 of AWWA D100-05.

AWWA D100-05 - Table 25

Site Class	Soil Profile
C	Soft Rock

Liquefaction

Liquefaction tends to occur in loose, saturated fine grained sands and coarse silts. The site is underlain shallowly by bedrock, an earth material that is not susceptible to liquefaction. Liquefaction is not a hazard associated with the Bethany tank site.

Landsliding, Slope Stability and Co-seismic Ground Cracking

The Geologic Report by Zinn Geology should be consulted for a detailed discussion of the potential for landsliding, slope failure and co-seismic ground cracking to adversely impact the project site.

CONCLUSIONS AND RECOMMENDATIONS

PRIMARY GEOTECHNICAL ISSUES

1. Site Viability (OK)

The results of our investigation indicate that from a Geotechnical Engineering standpoint the existing water tank can be upgraded as proposed.

2. Primary Geotechnical Issues

Existing Tank

Based on our field and laboratory investigations, it is our opinion that the primary geotechnical issues associated with the existing tank site, the proposed seismic upgrade of the existing tank and the design and construction of a new roof for the existing water tank are the following:

a. **Unknown Conditions:** It is our understanding that as-built documentation is not available for the existing tank. Therefore the assumptions used in this report regarding the existing foundation and the conditions of the engineered fill and native earth materials directly beneath the tank are based on our borings outside the footprint of the existing tank, our visual observations, discussions with the Scotts Valley Water District, standard practices and available design documents. All assumptions must be confirmed either prior to or during construction.

b. **Compressible Fill:** The undocumented fill encountered in Boring No. 2 on the southeast side of the existing water tank consists of loose silty sand and clayey sand which is considered to be compressible. All fill, native soil or oil sand beneath the base of the existing tank should be considered compressible in comparison to the sandstone bedrock, which we assume directly underlies the existing ring foundation.

To mitigate the potential that differential settlement will occur between the roof and the existing foundation, we recommend the new roof foundation extend through all sand, fill and compressible native soil, if encountered beneath the tank. All new roof footings should be directly underlain by competent sandstone bedrock. Refer to the Foundations section of this report for details.

Future Larger Diameter Replacement Tank

Based on our field and laboratory investigations, it is our opinion that the primary geotechnical issues associated with the design and construction of a larger diameter replacement tank at the Bethany site are the following:

a. **Unknown Conditions:** It is our understanding that as-built documentation is not available for the existing tank. Therefore the assumptions used in this report regarding the existing foundation and the conditions of the engineered fill and native earth materials directly beneath the tank are based on our borings outside the footprint of the existing tank, our visual observations, discussions with the Scotts Valley Water District, standard practices and available design documents. All assumptions must be confirmed either prior to or during construction.

b. **Compressible Fill:** The undocumented fill encountered in Boring No. 2 on the southeast side of the existing water tank consists of loose silty sand and clayey sand which is considered to be compressible. All fill, native soil or oil sand beneath the base of the existing tank should be considered compressible in comparison to the sandstone bedrock, which we assume directly underlies the existing ring foundation.

To mitigate the potential that differential settlement will occur between the roof and the existing foundation, we recommend the new roof foundation extend through all sand, fill and compressible native soil, if encountered beneath the tank. All new roof footings should be directly underlain by competent sandstone bedrock. Refer to the Foundations section of this report for details.

POST REPORT SERVICES (OK)

3. Plan Review

Grading, foundation and drainage plans should be reviewed by the Geotechnical Engineer during their preparation and prior to contract bidding to insure that the recommendations of this report have been included and to provide additional recommendations, if needed.

4. Construction Observation and Testing

Field observation and testing must be provided during construction by a representative of Bauldry Engineering, Inc. to enable them to form an opinion regarding the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the foundation, drainage, and earthwork construction, including the degree of compaction, comply with the specification requirements. Any work related to foundation, drainage or earthwork construction performed without the full knowledge of, and not under the direct observation of Bauldry Engineering, Inc., the Geotechnical Engineer, will render the recommendations of this report null and void.

5. Notification and Preconstruction Meeting

The Geotechnical Engineer should be notified at least four (4) working days prior to any earthwork and foundation construction operations in order to observe the stripping and disposal of unsuitable materials, and to coordinate this work with the contractor. During this period, a pre-construction conference should be held on the site, with at least the owner's representative, the contractor and one of our engineers present. At this time, the project specifications and the testing and construction observation requirements will be outlined and discussed.

. EARTHWORK AND GRADING

6. Demolition and Initial Site Preparation

The initial preparation of the site will consist of the demolition of the existing water tank roof and cutting holes through the floor of the existing tank in preparation for the new roof footings. Any abandoned elements of the existing tank including foundations, underground utilities or other subsurface obstructions or remnant root mats found in the new footing areas must be completely removed. Soils contaminated with deleterious material should be removed from the site. The extent of this soil removal will be designated by the Geotechnical Engineer in the field.

All voids, including those created by the demolition of the structures, foundations, subsurface obstructions, utilities or remnant roots must be backfilled with properly compacted non-expansive native soils that are free of organic and other deleterious materials or with approved import fill.

7. Compaction Requirements

All disturbed soil on the project should be compacted to a minimum of 90% of its maximum dry density. The maximum dry density will be obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557. This test will also establish the optimum moisture content of the material. Field density testing will be in accordance with ASTM Test #D2922.

8. Moisture Conditioning

The moisture conditioning procedure should result in soil with a relatively uniform moisture content of 1 to 3 percent over optimum at the time of compaction. If the soil is dry water may need to be added. If the soil is wet, it will need to be dried back. The native soil may require a diligent and active drying and/or mixing operation to reduce or raise the moisture content to the levels required to obtain adequate compaction.

9. Engineered Fill Material

Native soil and/or import fill may be used as engineered fill for the project as indicated below.

Re-use of the native soil will require the following:

a. Segregation of all expansive soil encountered during the excavation operation under the observation of the Geotechnical Engineer. All excavated expansive soil should be removed from the construction area.
b. Removal of organics, deleterious material, and cobbles larger than 3 inches.
c. Thorough mixing and moisture conditioning of approved native soil.

All imported engineered fill material should meet the criteria outlined below:

a. Granular, well graded, with sufficient binder to allow utility trenches to stand open.
b. Minimum Sand Equivalent of 20 and Resistance "R" Value of 30.
c. Free of deleterious material, organics and rocks larger than 2 inches in size.
d. Non-expansive with a Plasticity Index below 12.

Samples of any proposed imported fill planned for use on this project should be submitted to the Geotechnical Engineer for appropriate testing and approval not less than 4 working days before the anticipated jobsite delivery.

FOUNDATIONS - SPREAD FOOTINGS

10. General

It is our opinion that reinforced concrete spread footings constructed in conjunction with the site preparation procedures outlined in this report are an appropriate foundation system to support the new roof of the water tank. All footings should contain steel reinforcement as determined by the Project Structural Engineer.

To mitigate the potential that differential settlement will occur between the roof and the existing foundation, the new roof foundations should extend through all sand, fill and compressible native soil, if encountered beneath the tank. All new roof footings should be directly underlain by competent sandstone bedrock.

The footing excavations should be adequately moisture conditioned prior to placing concrete. Requirements for moisture conditioning the footing subgrade will depend on the soil type and seasonal moisture conditions, and will be determined by the Geotechnical Engineer at the time of construction.

All footing excavations must be observed by a representative of Bauldry Engineering, Inc. before steel is placed and concrete is poured to insure bedding into proper material.

11. Minimum Footing Dimensions
Footing widths should be based on allowable bearing values but not less than 18 inches. Minimum embedment depth for all new footings should be 24 inches below lowest adjacent grade or 6 inches into competent sandstone bedrock, whichever is deeper. We anticipate actual footing depths for the roof support to be on the order of 24 to 30 inches.

12. Allowable Bearing Capacity
Footings constructed in accordance with the criteria listed above may be designed for the following allowable bearing capacities.

Allowable Bearing Capacities

Footing Width	Footing Depth	Allowable Bearing Capacity
18 inches	24 inches	2,000 psf
24 inches	24 inches	2,200 psf
30 inches	24 inches	2,400 psf

The allowable bearing capacity listed above may be increased by 1/3rd for short duration loads such as those imposed by wind and seismic forces.

UTILITIES
13. Set Backs
New utility trenches that are parallel to the sides of the water tank should be placed so that they do not extend below a line with a 2:1 (H:V) gradient extending from the bottom outside edge of all footings.

14. Utility Trench Backfill
Trenches may be backfilled with the native materials or approved import granular material. The backfill soil should be compacted in thin lifts to a minimum of 95% of its maximum dry density in driveway areas and 90% in all other areas. Jetting of the trench backfill is not recommended.

15. Shoring
Trenches must be shored as required by the local agency and the State of California Division of Industrial Safety construction safety orders.

SURFACE DRAINAGE
16. Stability of Slopes
Controlling surface drainage and landscape irrigation is critical to the long-term stability of the slopes at the subject site. It is imperative that irrigation activities and all concentrated

surface water be effectively controlled. Uncontrolled surface drainage could cause slope instability.

17. Surface Grades and Storm Water Runoff

Water must not be allowed to pond adjacent to the water tank foundation. Final grades should slope away from foundations such that water is rapidly transported to drainage facilities.

18. Drain Pipes

Subsurface pipes used in storm water runoff systems must be robust rigid solid pipes capable of supporting the overburden loads. Flexible corrugated pipes must not be used.

19. Maintenance

Surface drainage facilities must not be altered, and there should be no modifications of the finished grades at the project site without first consulting Bauldry Engineering, Inc.

The building and surface drainage facilities must be inspected and maintained on a routine basis. Repairs, whenever necessary, must be made in a timely manner. We recommend that the property owner inspect the drainage systems prior to each rainy season, following the first significant rain, and throughout each rainy season. The project civil and geotechnical engineers should be consulted if significant erosion or other drainage problems occur so that the conditions can be observed and supplemental recommendations can be provided, as necessary.

FIELD AND LABORATORY METHODS

Field Investigation

Three 4 inch diameter test borings were drilled on the site on July , 2006. The drilling method used was hydraulically operated continuous flight augers. An engineer from Bauldry Engineering, Inc. was present during the drilling operations to log the soils encountered and to choose soil sampling type and locations.

Relatively undisturbed soil samples were obtained at various depths by driving a split spoon sampler 18 inches into the ground. This was achieved by freely dropping a 140 pound down hole safety hammer through a vertical height of 30 inches. The number of blows needed to drive the sampler for each 6 inch portion is recorded and the total number of blows needed to drive the last 12 inches is reported as the Standard Penetration Test value. The outside diameter of the samplers used in this investigation was either 3 inches, 2½ inches, or 2 inches as denoted by “L”, “M”, or “T” respectively in the boring logs. All standard penetration test data has been normalized to a 2 inch O.D. sampler so as to be the standard "N" value.

Appendix A contains the Site Plan showing the locations of the test borings and the Log of Test Borings presenting the soil profile explored in each boring, the sample locations, and the SPT "N" values for each sample. Contacts between soil types and the different earth materials shown on the boring logs are approximate as the actual transition may be gradual.

Laboratory Investigation

The laboratory testing program was developed to help in evaluating the bearing capacity, settlement characteristics, swell potential, lateral earth characteristics, liquefaction potential, pavement design parameters, and the stable slope gradients for the soil on the site. Laboratory tests performed include:

1. Moisture Density relationships in accordance with ASTM test D2937.
2. Direct Shear tests in accordance with ASTM test D3080.
3. Unconfined Compression tests in accordance with ASTM test D2166.
4. Atterberg Limits tests in accordance with ASTM test D4318.
5. Expansion Index tests in accordance with ASTM test D4829
6. "R" Value tests in accordance with California test 301.
7. Swell Pressure tests in accordance with ASTM test D4546.
8. Penetrometer tests to determine unconfined compressive strength.
9. Gradation tests in accordance with ASTM test D422.

The results of the laboratory tests are presented on the boring logs opposite the sample tested.

1020-SZ932-H73
July 30, 2010

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time our report was written, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to insure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to insure that the Contractors and Subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes.

4. This report was prepared upon your request for our services in accordance with currently accepted standards of professional Geotechnical Engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.

5. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.

6. Bauldry Engineering, Inc. is not a mold prevention consultant; none of our services performed in connection with the proposed project are for the purpose of mold prevention. Proper implementation of the recommendations conveyed in our reports will not of itself be sufficient to prevent mold from growing in or on the structures involved. Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. Your project Architect or a mold prevention specialist should be consulted regarding mold prevention.

7. Determination or verification of locations and grades during the investigation, planning or construction phases of the project is outside our scope of services. This work will not be performed by our firm. Determination of locations and grades is the responsibility of others.

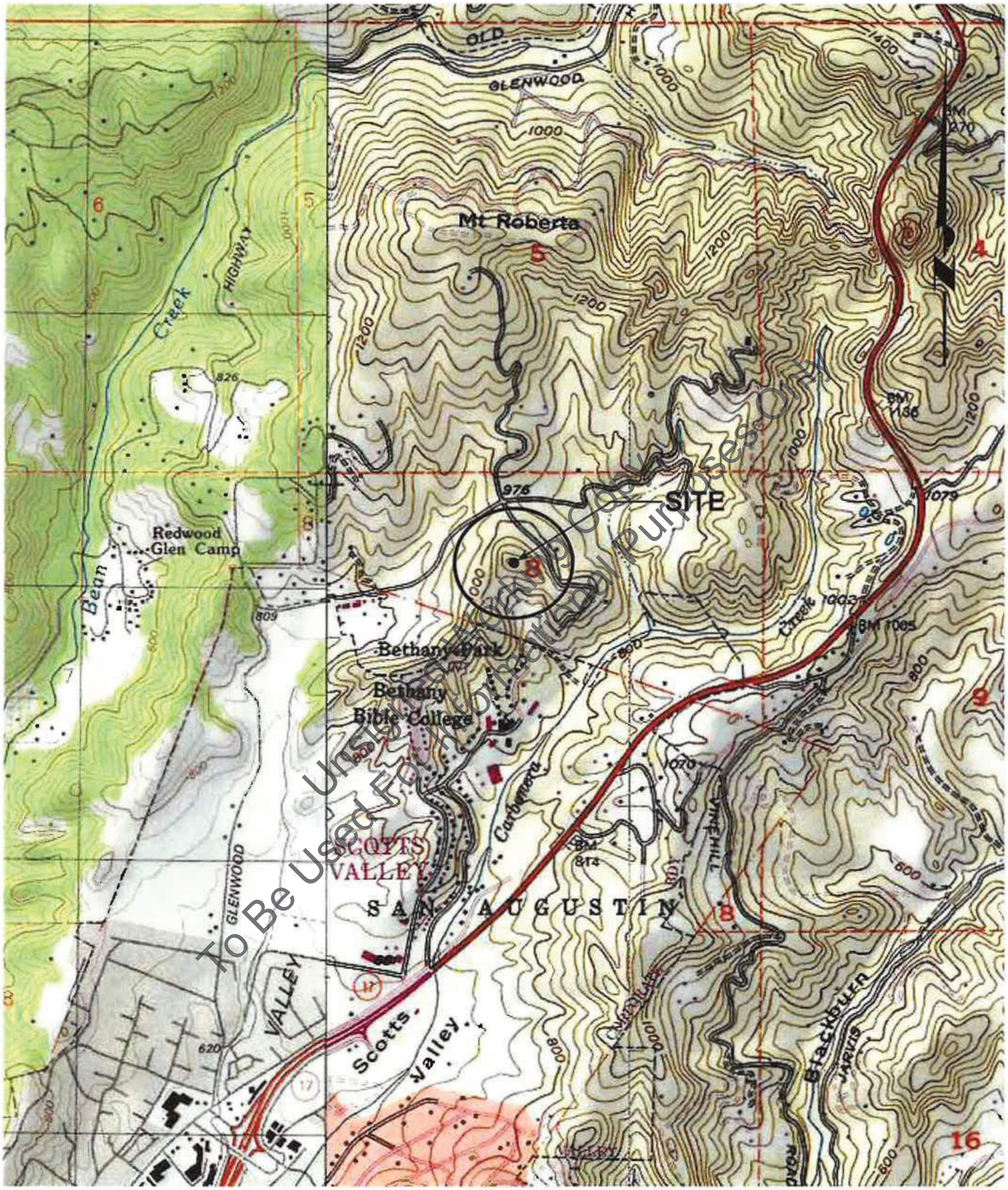
1020-SZ932-H73
July 30, 2010

ASFE – IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

1020-SZ932-H73
July 30, 2010

APPENDIX A

PROJECT NO. 1020-SZ932-H73	REPORT DATE:



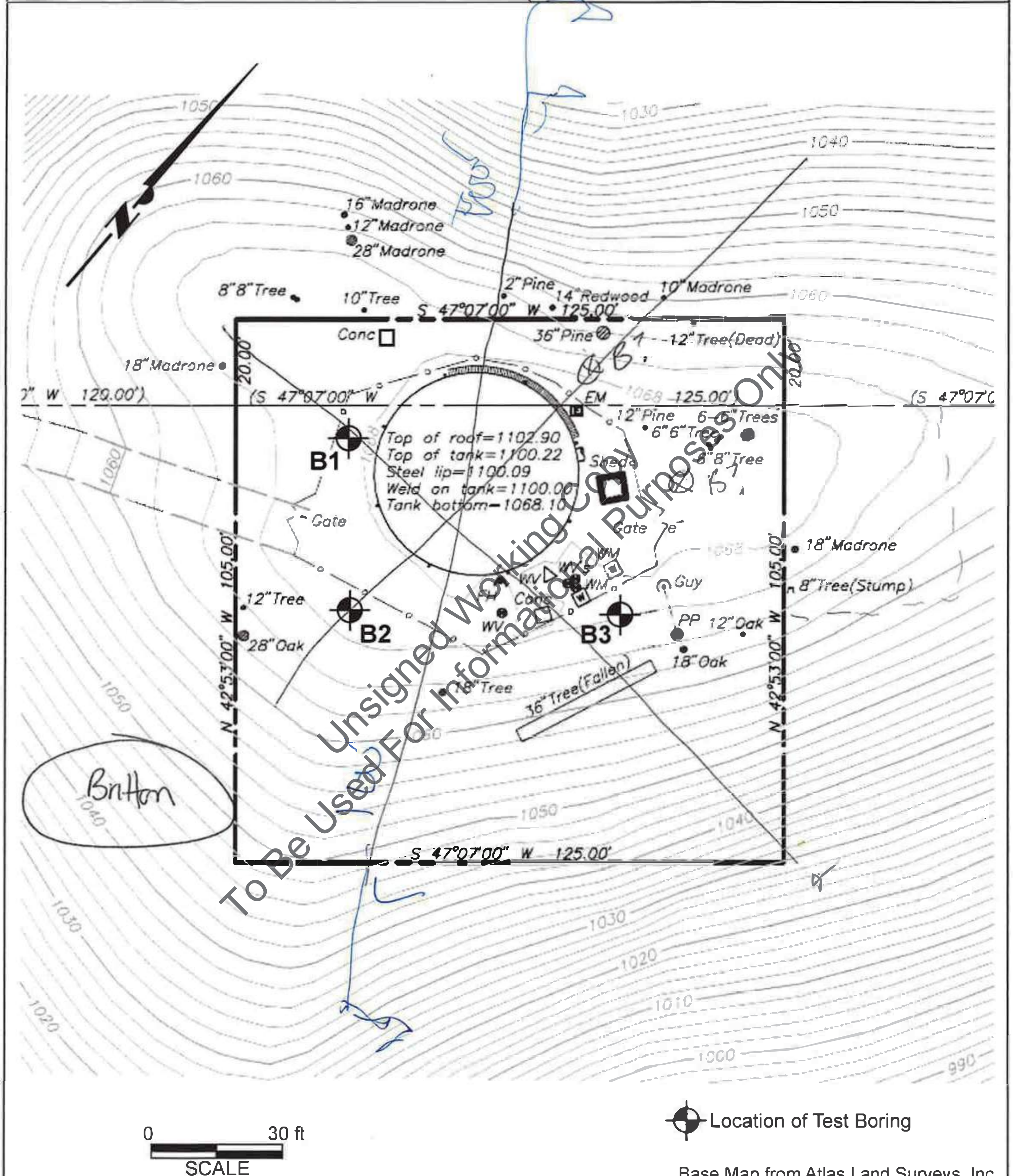
0 2000 ft
SCALE

Base Map from U.S.G.S 7.5 Minute Topographic Map; Felton Quadrangle

Bauldry Engineering, Inc.	Figure No. 1	Regional Site Plan

PROJECT NO. 1020-SZ932-H73

REPORT DATE:



Base Map from Atlas Land Surveys, Inc.

Bauldry Engineering, Inc.

Figure No. 2 Site Plan Showing Test Borings

PROJECT NO. 1020-SZ932-H73 REPORT DATE:

BORING LOG EXPLANATION

Logged By ____ Date Drilled ________ Boring Diameter ___ Boring No. ___

Depth, ft.	Sample No.	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density, p.c.f.	Moisture % of Dry Wt.	MISC. LAB RESULTS
			▼ ← Ground water elevation	Note: All blows/foot are normalized to 2" outside diameter sampler size					
	1-1 L		← Soil Sample Number ← Soil Sampler Size/Type L = 3" Outside Diameter M = 2.5" Outside Diameter T = 2" Outside Diameter ST = Shelby Tube BAG = Bag Sample						

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488-84

PRIMARY DIVISIONS			GROUP SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	**GRAVELS** MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	**CLEAN GRAVELS** (LESS THAN 5% FINES)	GW	Well graded gravels, gravel sand mixture, little or no fines.
			GP	Poorly graded gravels or gravels-sand mixtures, little or no fines.
		GRAVELS (MORE THAN 12% FINES)	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	**CLEAN SANDS** (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS (MORE THAN 12% FINES)	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine clayey sand or silty sand, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, silty or lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Organic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT
VERY SOFT	0 - 2
SOFT	2 - 4
FIRM	4 - 8
STIFF	8 - 16
VERY STIFF	16 - 32
HARD	OVER 32

Bauldry Engineering, Inc. Figure No. 3 Boring Log Explanation

PROJECT NO. 1020-SZ932-H73 REPORT DATE:

Logged By BDB Date Drilled 6/22/10 Boring Diameter 4"SS Boring No. 1

Depth, ft.	Sample No.	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	MISC. LAB RESULTS
0–1			BEDROCK:						
1–2	1-1 L		Mottled reddish brown/gray Sandy SILTSTONE, near vertical infilled fracture, dark brown infill, slightly weathered, soft		41	119.5	23.8	96.6	
3	1-2 T				42/5"		21.6		
7–8	1-3 T		Reddish brown/grayish brown Sandy SILTSTONE, slightly weathered, soft to moderately hard		50/3"		24.1		
15–16	1-4 T		Reddish brown/grayish brown Sandy SILTSTONE, slightly weathered, soft to moderately hard		50/ 4½"		24.2		
			Boring terminated at 15¾'						

PROJECT NO. 1020-SZ932-H73 REPORT DATE:

Logged By BDB Date Drilled 6/22/10 Boring Diameter 4"SS Boring No. 2

Depth, ft.	Sample No.	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	MISC. LAB RESULTS
0–1			FILL: Brown, reddish brown Silty SAND, mudstone clasts, damp, loose	SM					
1–2	2-1 L				12	93.0	11.1	83.7	EI = 19 37% Passing #200 Sieve
3	2-2 T		Silty Sandstone floater Brown, reddish brown Silty SAND, mudstone clasts, damp, loose		26		28.1		
6–7	2-3 L		Brown Silty SAND to Sandy SILT, damp, medium dense		23	104.1	29.0	80.7	
7			BEDROCK: Reddish brown, gray Sandy SILTSTONE, near vertical fracture infilled with dark brown soil, moderately weathered, soft						
8–9	2-4 T				50/5"		24.5		
15–16	2-5 T		Reddish brown, olive brown Sandy SILTSTONE, moderately to slightly weathered, soft		69		25.0		
22	2-6 T		No sample recovery Very slow drilling		100/5"				

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

PROJECT NO. 1020-SZ932-H73			REPORT DATE:	
Logged By BDB	Date Drilled 6/22/10		Boring Diameter 4"SS	Boring No. 2 (cont)

Depth, ft.	Sample No.	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	MISC. LAB RESULTS
24	2-7 T		No sample recovery Silstone fragments on auger		50/ 2½"				
			Boring terminated at 24¼'						
25									
26									
27									
28									
29									
30									
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									

PROJECT NO. 1020-SZ932-H73 REPORT DATE:

Logged By BDB Date Drilled 6/22/10 Boring Diameter 4"SS Boring No. 3

Depth, ft.	Sample No.	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Wet Density (pcf)	Moisture Content (%)	Dry Density (pcf)	MISC. LAB RESULTS
			Sandy SILT	ML					
1	3-1 L		BEDROCK: Yellowish brown/reddish brown Silty SANDSTONE, near vertical fracture with 1" wide at 1' bgs to hairline at 2½' bgs infilled with dark brown soil, slightly weathered, soft						
2					39	108.6	21.6	89.3	
3	3-2 T				52		25.1		
4									
5			Increase in drilling resistance Slow drilling						
6	3-3 T		Reddish brown/grayish brown Sandy SILTSTONE, slightly weathered, soft		50/5½"		24.8		
7									
8									
9									
10	3-4 T		Reddish brown/grayish brown Sandy SILTSTONE, slightly weathered, soft		50/6"		27.0		
11			Boring terminated at 11'						
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									

APPENDIX B

Laboratory Results