GEOTECHNICAL INVESTIGATION
FOR
NEW ELECTICAL & CHEMICAL
FEED BUILDING
SEASIDE, CALIFORNIA

FOR
PUEBLO WATER RESOURCES
VENTURA, CALIFORNIA

BY
PACIFIC CREST ENGINEERING INC.
CONSULTING GEOTECHNICAL ENGINEERS
0922-M242-E12
APRIL 2009
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April 30, 2009

Project No. 0922-M242-E12

Pueblo Water Resources
4478 Market Street, Suite 705
Ventura, CA 93003

Attention: Mr. Steve Tanner, PE

Subject: Geotechnical Investigation
New Electrical & Chemical Feed Building
Santa Margarita Aquifer Storage and Recovery Project
1110 General Jim Moore Boulevard
Seaside, California

Dear Mr. Tanner,

In accordance with your authorization, we have performed a geotechnical investigation for the above referenced project located at 1110 General Jim Moore Boulevard, in Seaside, California.

The accompanying report presents our conclusions and recommendations as well as the results of the geotechnical investigation on which they are based. If you have any questions concerning the data, conclusions or recommendations presented in this report, please call our office.

Very truly yours,

PACIFIC CREST ENGINEERING INC.

Cara L Russo

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Staff Geologist

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Copies: 4 to Client

GEOTECHNICAL INVESTIGATION

PURPOSE AND SCOPE

This report describes the geotechnical investigation and presents results, including recommendations, for your new electrical and chemical feed building project located at 1110 General Jim Moore Boulevard in Seaside, California. Our scope of services for this project has consisted of:

1. Discussions with you and the members of the design team including Mr. Joe Oliver of the Monterey Peninsula Water District.
2. Review of the pertinent published material concerning the site including County planning maps, preliminary site plans, geologic and topographic maps, and other available literature.
3. The drilling and logging of 2 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.

LOCATION AND DESCRIPTION

The project site is located at 1110 General Jim Moore Boulevard on the east side of the road. Please refer to Figure No. 1, Regional Site Map, for the general vicinity of the project site. The project site is just south of the intersection with Eucalyptus Road and is located at the following coordinates:

Latitude =	36.620227 degrees
Longitude =	-121.816631 degrees

At the time of our site visits, the vicinity of the proposed new electrical and chemical feed building was vacant. The plot was graded, stepped cut, and sloped to the west. A few native plants were scattered about, but the Older Dune Deposits were visible at the ground surface. The site of the proposed project was completely surrounded by a gravel loop. An existing trailer, wood shed on a concrete pad, a metal shed, and wells were present within the same parcel as the proposed new building.

It is our understanding that the project involves the construction of a one-story utility building with a total floor area of approximately 1,200 square feet. The existing building pad consists of a graded and stepped cut pad. The pad at the higher elevation will be excavated an additional 12 to 18 inches from its present location in order to bring the pad to one elevation. The southwestern portion of the building will have a 4 to 5 foot deep basement in the lower section of the building for double containment of fluids and spill control in the storage room. It is our understanding that the basement will be a concrete structure with a concrete slab-on-grade floor.

FIELD INVESTIGATION

Soil Borings

Two 6 inch diameter test borings were drilled on the site on April 17, 2009. The location of the test borings are shown on Figure No. 2, Site Map Showing Test Borings. The drilling method used was hydraulically operated continuous flight augers. A geologist from Pacific Crest Engineering Inc., was present during the drilling operations to log the soil 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 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 (SPT) value. The outside diameter of the samplers used in this investigation was 3 inches and is noted respectively as “L” on the boring logs. All standard penetration test data has been normalized to a 2 inch O.D. sampler so as to be the SPT "N" value.

The soils encountered in the borings were continuously logged in the field and visually described in accordance with the Unified Soil Classification System (ASTM D2488 (Modified), Figure No. 3). The soil classification was verified and or modified upon completion of laboratory testing.

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. Stratification lines on the boring logs are approximate as the actual transition between soil types may be gradual.

LABORATORY INVESTIGATION

The laboratory testing program was developed to help in evaluating the engineering properties of the materials encountered on the site. Laboratory tests performed include:

a. Moisture Density relationships in accordance with ASTM test D2937.

b. Gradation tests in accordance with ASTM test D1140.

c. Corrosivity testing including pH, resistivity, chloride concentration, and sulfate concentration.

The results of the laboratory tests are presented on the boring logs opposite the sample tested or within Appendix A.

SOIL CONDITIONS

Regional Geologic Maps

The surficial geology in the area of the project site is mapped as Older Coastal Dunes (Clark, Dupre', and Rosenberg, 1997). The Older Coastal Dunes are described as weakly consolidated, poorly graded fine to medium grained sand deposits. Some of these deposits are covered with a thin lens of eolian deposits. The native soils encountered in the test borings are consistent with this description.

Soil Borings

Our borings encountered a variety of soil including silty sand, sand with silt, and sand. Both test borings were drilled within the footprint of the proposed new electrical and chemical feed building. The following describes the soil conditions encountered within each test boring.

Boring No. 1 encountered brown silty sand in the upper 24 feet. The sand was fine to medium grained, sub-angular to sub-rounded shaped, and poorly graded. Mica flakes were scattered throughout the obtained samples and the samples tended to coarsen with depth. Trace rounded chert pebbles were noted near 6 ½ feet. The surface soils within the cut were fairly well compacted as the density near 3 ½ feet was described as hard. Overall, the density ranged from medium dense to hard. From 24 feet to the maximum explored depth of 36 feet the soil was described as yellowish tan sand. The sand was fine to medium grained with trace coarse grains, sub-angular to sub-rounded shaped, and poorly graded. Mica flakes were scattered throughout the collected samples. The density ranged from medium dense to very dense.

Boring No. 2 encountered dark brown sand with silt in the upper 5 feet. The sand was very fine to medium grained, sub-angular to sub-rounded shaped, and poorly graded. Mica flakes and trace rounded chert pebbles were scattered throughout the obtained sample. Trace granitic gravel was noted near 3 ½ feet. At this depth the density was described as medium dense. From 5 feet to the maximum depth explored of 16 ½ feet the boring encountered dark reddish brown sand with silt. The sand was fine to medium grained with trace coarse grains, sub-angular to sub-rounded shaped, and poorly graded. Mica flakes were scattered throughout the collected samples. Trace rounded chert pebbles were noted from 11 to 11 ½ feet. At these depths the density was described as medium dense.

Groundwater was not encountered in any of the test borings to a maximum explored depth of 36 feet.

REGIONAL SEISMIC SETTING

The seismic setting of the site is one in which it is reasonable to assume that the site will experience significant seismic shaking during the lifetime of the project.

Based upon our review of the fault maps for the for the Monterey area (Clark, Dupre', and Rosenberg, 1997), and the Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada (CDMG, 1998), active or potentially active faults which may significantly affect the site include those listed in the Table No. 1, below.

TABLE No. 1, Faults in the Monterey Bay Area

Fault Name	Distance (miles)	Distance (km.)	Direction	Slip Rate* (mm/yr.)	M_W Max*
San Andreas – 1906 Segment	21.7	35.0	Northeast	24	7.9
Palo Colorado – Sur	12.0	19.3	Southwest	3	7.0
Rinconada	5.0	8.1	Northeast	1	7.5
Monterey Bay – Tularcitos	3.6	5.8	Southwest	0.5	7.3

*Source: CDMG, February, 1998

SEISMIC HAZARDS

A detailed investigation of seismic hazards is beyond our scope of services for this project. In general however, seismic hazards which may affect project sites in the Monterey Bay area include ground shaking, ground surface fault rupture, liquefaction and lateral spreading, and seismically induced slope instabilities. Geotechnical aspects of these issues are discussed below:

Ground Shaking

Ground shaking will be felt on the site. 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. 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. The seismic design of the project should be based on the 2007 California Building Code (CBC) as it has incorporated the most recent seismic design parameters. The following values for the seismic design of the project site were derived or taken from the 2007 CBC:

TABLE No. 2, The 2007 CBC Seismic Design Parameters

Design Parameter	**Specific to Site**	**Reference (See Note 1)**
Site Class	D, Stiff Soil	Table 1613.5.2
Mapped Spectral Acceleration for Short Periods	$S_s = 1.302$ g	Fig. 22-3, ASCE 7-05
Mapped Spectral Acceleration for 1-second Period	$S_1 = 0.558$ g	Fig. 22-4, ASCE 7-05
Short Period Site Coefficient	Fa = 1.0	Table 1613.5.3(1)
1-Second Period Site Coefficient	Fv = 1.5	Table 1613.5.3(2)
MCE Spectral Response Acceleration for Short Period	$S_{MS} = 1.302$ g	Section 1613.5.3
MCE Spectral Response Acceleration for 1-Second Period	$S_{M1} = 0.837$ g	Section 1613.5.3
5% Damped Spectral Response Acceleration for Short Period	$S_{DS} = 0.868$ g	Section 1613.5.4
5% Damped Spectral Response Acceleration for 1-Second Period	$S_{D1} = 0.558$ g	Section 1613.5.4
Seismic Design Category (**See Note 2**)	D	Section 1613.5.6

Note 1: Design values may also have been obtained by using the Ground Motion Parameter Calculator available on the USGS website at http://earthquake.usgs.gov/research/hazmaps/design/index.php. Refer to the "Liquefaction" section for further information on how the Site Class may have been derived.

Note 2: Seismic Design Category assumes Class II occupancy per 2007 CBC Table 1604.5. Pacific Crest Engineering Inc. should be contacted for revised Table 2 seismic design parameters if the building has a different occupancy rating from the one assumed.

Ground Surface Fault Rupture

Ground surface fault rupture occurs along the surficial trace(s) of active faults during significant seismic events. Pacific Crest Engineering Inc. has not performed a specific investigation for the presence of active faults on the project site. Since the nearest known active or potentially active fault is mapped approximately 3.6 miles (approximately 5.8 km) from the site (Clark, Dupre', Rosenberg, 1997, and CDMG, 1998), the potential for ground surface fault rupture at this site is low.

Liquefaction

Liquefaction tends to occur in loose, saturated fine grained sands, coarse silts or clays with a low plasticity. Based upon our review of the regional liquefaction maps (Dupre' and Tinsley, 1980; Rosenberg, 2001) the site is located in an area classified as having a low potential for liquefaction. We did encounter loose, cohesionless clean sands within our test borings, however, we did not encounter groundwater in the upper 36 feet. Neither did we encounter clays with a Plasticity Index of 7 or lower (refer to the paper "Liquefaction Susceptibility Criteria for Silts and Clays" by Boulanger and Idriss, 2006). The soils encountered in are test borings were generally silty or poorly graded sands that were loose to medium dense near the surface and became very dense with depth.

Generally, we would not expect a significant amount of liquefaction to occur at this site, given the lack of groundwater in the upper 36 feet and the increasing density of the soils with depth. Our site specific investigation of this project site, including the nature of the subsurface soil, the location of the ground water table, and the estimated ground accelerations, leads to the conclusion that the liquefaction potential is low.

Liquefaction Induced Lateral Spreading

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 of the project site indicates that the potential for liquefaction to occur is low, and consequently the potential for lateral spreading is also low.

Landsliding

Seismically induced landsliding is a hazard with low potential for affecting your site since the site is relatively flat.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

GENERAL

1. The results of our investigation indicate that from a geotechnical engineering standpoint the property may be developed as proposed provided these recommendations are included in the design and construction.

2. Our laboratory testing indicates that the near surface soils possess low expansive properties. This analysis was based on several sieve analyses and our visual classification of the soils by a Staff Geologist based on the Unified Soil Classification System.

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

4. 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 you or your representative, the grading contractor, a City or County representative and one of our engineers present. At this meeting, the project specifications and the testing and inspection responsibilities will be outlined and discussed.

5. Field observation and testing must be provided by a representative of Pacific Crest Engineering Inc., to enable them to form an opinion as to the degree of conformance of the exposed site conditions to those foreseen in this report, the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. **Any work related to grading or foundation excavation that is performed without the full knowledge and direct observation of Pacific Crest Engineering Inc., the Geotechnical Engineer of Record, will render the recommendations of this report invalid, unless the Client hires a new Geotechnical Engineer who agrees to take over complete responsibility for this report's findings, conclusions and recommendations.** The new Geotechnical Engineer must agree to prepare a Transfer of Responsibility letter. This may require additional test borings and laboratory analysis if the new Geotechnical Engineer does not completely agree with our prior findings, conclusions and recommendations.

PRIMARY GEOTECHNICAL CONSIDERATIONS

6. 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 architectural damage and the need for post-earthquake repairs.

SITE PREPARATION

7. The initial preparation of the site will consist of the removal of any existing on-site debris. Septic tanks and leaching lines, if found, must be completely removed. The extent of this soil removal will be designated by a representative of Pacific Crest Engineering Inc. in the field. This material must be removed from the site.

8. Any voids created by removal of tree and root balls, septic tanks, and leach lines must be backfilled with properly compacted native soils that are free of organic and other deleterious materials or with approved imported fill.

9. Any wells encountered shall be capped in accordance with the requirements and approval of the County Health Department. The strength of the cap shall be equal to the adjacent soil and shall not be located within 5 feet of a structural footing.

10. Surface vegetation, tree roots and organically contaminated topsoil should then be removed ("stripped") from the area 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.

11. It is anticipated that the depth of stripping may be 2 to 4 inches, however the required depth of stripping must be based upon visual observations of a representative of Pacific Crest Engineering Inc., in the field. The depth of stripping will vary upon the type and density of vegetation across the project site and with the time of year. Areas with dense vegetation or groves of trees may require an increased depth of stripping.

12. It is possible that there are areas of man-made fill on the project site that our field investigation did not detect. Areas of man-made fill, if encountered on the project site 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 approved native soils that are free of organic and other deleterious materials, or with approved imported fill.

13. Following the stripping and backfilling of voids, the area should be excavated to the design soil subgrade elevation. The exposed soils in the building and paving areas should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as an engineered fill except for any contaminated material noted by a representative of Pacific Crest Engineering Inc. in the field. The moisture conditioning procedure will depend on the

time of year that the work is done, but should result in the soils being 1 to 3 percent over their optimum moisture content at the time of compaction. Compaction of the exposed subgrade soils should extend 5 feet horizontally beyond all slabs, footings and pavement areas.

Note: If this work is done during or soon after the rainy season, the on-site soils and other materials 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.

14. The soil on the project site should be compacted as follows:

    a. In pavement areas the upper 8 inches of subgrade, and all aggregate subbase and aggregate base, should be compacted to a minimum of 95% of its maximum dry density,
    b. In pavement areas all utility trench backfill should be compacted to 95% of its maximum dry density,
    c. All remaining soil on the project site should be compacted to a minimum of 90% of its maximum dry density.

15. 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 #D2922 (nuclear method).

16. Native or imported soil used as engineered fill on this project should meet the following requirements:

    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 utility trenches to stand open,
    d. free of rocks in excess of 2 inches in size.

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.

17. All native and import 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.

18. We recommend field density testing be performed in maximum 2 foot elevation differences. In general terms, we would recommend at least one compaction test per 200 linear feet of utility trench or retaining wall backfill, and at least one compaction test per

2,000 square feet of building or structure area. 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.

19. 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 ten (10) working days before the anticipated jobsite delivery. 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.

CUT AND FILL SLOPES

20. All fill slopes should be constructed with engineered fill meeting the minimum density requirements of this report and have a gradient no steeper than 3:1 (horizontal to vertical). Fill slopes should not exceed 15 feet in vertical height unless specifically reviewed by Pacific Crest Engineering Inc. Where the vertical height exceeds 15 feet, intermediate benches must be provided. These benches should be at least 6 feet wide and sloped to control surface drainage. A lined ditch should be used on the bench.

21. Fill slopes should be keyed into the native slopes by providing a 10 foot wide base keyway sloped negatively at least 2% into the bank. The depth of the keyways will vary, depending on the materials encountered. It is anticipated that the depth of the keyways may be 3 to 6 feet, but at all locations shall be at least 2 feet into firm material.

22. Subsequent keys may be required as the fill section progress upslope. Keys will be designated in the field by a representative of Pacific Crest Engineering Inc. See Figure No. 8 for general details.

23. Cut slopes shall not exceed a 3:1 (horizontal to vertical) gradient and a 15 foot vertical height unless specifically reviewed by a representative of Pacific Crest Engineering Inc. Where the vertical height exceeds 15 feet, intermediate benches must be provided. These benches should be at least 6 feet wide and sloped to control surface drainage. A lined ditch should be used on the bench.

24. The above slope gradients are based on the strength characteristics of the materials under conditions of normal moisture content that would result from rainfall falling directly on the slope, and do not take into account the additional activating forces applied by seepage from spring areas. Therefore, in order to maintain stable slopes at the recommended gradients, it is important that any seepage forces and accompanying hydrostatic pressure encountered be relieved by adequate drainage. Drainage facilities may include subdrains, gravel blankets, rock fill surface trenches or horizontally drilled drains. Configurations and type of drainage will be determined by a representative of Pacific Crest Engineering Inc. during the grading operations.

25. The surfaces of all cut and fill slopes should be prepared and maintained to reduce erosion. This work, at a minimum, should include track rolling of the slope and effective planting. The protection of the slopes should be installed as soon as practicable so that a sufficient growth will be established prior to inclement weather conditions. It is vital that no slope be left standing through a winter season without the erosion control measures having been provided.

26. The above recommended gradients do not preclude periodic maintenance of the slopes, as minor sloughing and erosion may take place.

27. If a fill slope is to be placed above a cut slope, the toe of the fill slope should be set back at least 8 feet horizontally from the top of the cut slope. A lateral surface drain should be placed in the area between the cut and fill slopes.

EROSION CONTROL

28. The surface soils are classified as having a high 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, you should consult your civil engineer or an erosion control specialist.

FOUNDATIONS - SPREAD FOOTINGS

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

30. Considering the soil characteristics and site preparation recommendations, it is our opinion that an appropriate foundation system to support the proposed structures will consist of reinforced concrete spread footings bedded into firm native soil. This system could consist of continuous exterior footings, in conjunction with interior isolated spread footings or additional continuous footings or concrete slabs.

31. Footing widths and depths should be based upon the allowable bearing value but not less than the minimum widths and depths as shown in the table below. The footing excavations must be free of loose material prior to placing concrete. The footing excavations should be thoroughly saturated prior to placing concrete.

TABLE No. 3, Minimum Footing Widths and Depths

Number of Stories	Footing Width	Footing Depth
1	12 inches	12 inches
2	15 inches	18 inches
3	18 inches	24 inches
Multi-story	24 inches	24 inches

Please note: The minimum footing embedment is measured from the lowest existing and adjacent soil grade and should not include any concrete slab-on-grade, capillary break and sand cushion in the total depth of embedment.

32. Footings constructed to the given criteria may be designed for the following allowable bearing capacities:

  a. 2,000 psf for Dead plus Live Load
  b. a 1/3rd increase for Seismic or Wind Load

Please note: In computing the pressures transmitted to the soil by the footings, the embedded weight of the footing may be neglected.

33. Expected total settlement due to applied dead and live loads is not expected to exceed 1 inch across the length of the structure, with differential settlement of about 0.5 to 0. 6 inches.

34. No footing should be placed closer than 8 feet to the top of a fill slope nor 6 feet from the base of a cut slope.

35. No footing shall be placed on slopes steeper than 4:1 (h:v). If the intent is to place the foundation on sloping ground which exceeds 4:1 (h:v), Pacific Crest Engineering Inc. should be contacted for an alternative pier and grade beam foundation design.

36. All footings should be excavated into firm native soil. No footings shall be constructed with the intent of placing engineered fill against the footing after the footing is poured, and counting that engineered fill as part of the embedment depth of the footing.

37. Footings may be assumed to have a resistance to lateral sliding of 0.35.

38. Footings may be assumed to have a lateral bearing pressure resistance value of 250 psf/ft.

39. All grade beams, thickened slab edges and other foundation elements which impart structure loads to the soil (from dead, live, wind or seismic loads) should be considered "footings" and constructed according to the recommendations of this section, including required depths below lowest adjacent soil grade.

40. Footing excavations must be observed by a representative of Pacific Crest Engineering Inc. before placement of formwork, steel and concrete to ensure bedding into proper material.

41. The footings should contain steel reinforcement as determined by the Project Civil or Structural Engineer in accordance with applicable CBC or ACI Standards.

SLAB-ON-GRADE CONSTRUCTION

42. Concrete slab-on-grade floors may be used for ground level construction on native soil or engineered fill. The upper 8 inches of slab subgrade should be processed and compacted to a minimum of 95% relative dry density.

43. Slabs may be structurally integrated with the footings. If the slabs are constructed as "free floating" slabs, they should be provided with a minimum ¼ inch felt separation between the slab and footing. The slabs should be separated into approximately 15' x 15' square sections with dummy joints or similar type crack control devices.

44. All concrete slabs-on-grade should be underlain by a minimum 4 inch thick capillary break of ¾ inch clean crushed rock (no fines). It is recommended that neither Class II baserock nor sand be employed as the capillary break material.

45. Where floor coverings are anticipated or vapor transmission may be a problem, a vapor/waterproof membrane should be placed between the capillary break layer and the floor slab in order to reduce the potential for moisture condensation under floor coverings. We recommend a high quality vapor retarder at least 10 mil thick and puncture resistant (Stego Wrap or equivalent). The vapor barrier must be a least 10 mil in thickness and meet the specifications for ASTM E 1745, Standard Specification For Water Vapor Retarder A 2-inch layer of moist sand on top of the membrane will help protect the membrane and will assist in equalizing the curing rate of the concrete.

Please Note: Recommendations given above for the reduction of moisture transmission through the slab are general in nature and present good construction practice. Moisture protection measures for concrete slabs-on-grade should meet applicable ACI and ASTM standards. Pacific Crest Engineering Inc. are not waterproofing experts. For a more complete and specific discussion of moisture protection within the structure, a waterproofing expert should be consulted.

46. 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.

47. Slab thickness, reinforcement, and doweling should be determined by the Project Civil or Structural Engineer. The use of welded wire mesh is not recommended for slab reinforcement.

UTILITY TRENCHES

48. Utility trenches that are parallel to the sides of the building 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.

49. 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.

50. 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.

51. Unless concrete bedding is required around utility pipes, free-draining clean sand should be used as bedding. Sand bedding should be compacted to at least 95 percent relative compaction.

52. Approved imported clean sand or native soil 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, patios, and other hardscape areas. Each layer of trench backfill should be water conditioned and 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.

53. **All utility trenches beneath perimeter footing or grade beams should be backfilled with controlled density fill (such as 2-sack sand\cement slurry) to help minimize potential moisture intrusion below interior floors. The width of the plug should be at least <u>three times</u> the width of the footing or grade beam at the building perimeter, but not less than 36 inches. A representative from Pacific Crest Engineering Inc. should be contacted to observe the placement of slurry plugs. In addition, all utility pipes which penetrate through the footings, stemwalls or grade beams (below the exterior soil grade) should also be sealed water-tight, as determined by the Project Engineer or Architect.**

54. 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.

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

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

LATERAL PRESSURES

57. Retaining walls with full drainage should be designed using the following criteria:

   a. The following lateral earth pressure values should be used for design:

TABLE No. 4, Active and At-Rest Earth Pressure Values

Backfill Slope (H:V)	Active Earth Pressure (psf/ft of depth)	At-rest Earth Pressure (psf/ft of depth)
Level	30	40
3:1	35	45
2:1	45	55

58. Active earth pressure values may be used when walls are free to yield an amount sufficient to develop the active earth pressure condition (about ½% of height). The effect of wall rotation should be considered for areas behind the planned retaining wall (pavements, foundations, slabs, etc.). **When walls are restrained at the top or to design for minimal wall rotation, use the at-rest earth pressure values.**

   a. For resisting passive earth pressure use 250 psf/ft of depth.

   b. A "coefficient of friction" between base of foundation and soil of 0.35.

   c. Exterior or interior wall footings may be designed for an allowable bearing capacity of 2,000 psf for Dead plus Live Load, with a 1/3rd increase for short term loads.

   d. To develop the resisting passive earth pressure, the retaining wall footings should be embedded a minimum of 18 inches below the lowest adjacent grade. There should be a minimum of 5 feet of horizontal cover as measured from the outside edge of the footing.

   e. Any live or dead loads which will transmit a force to the wall, refer to Figure No. 9.

f. For flexible (yielding) retaining walls, the resultant seismic force on the wall is $8H^2$ and acts at a point 0.6H up from the base of the wall. This force has been estimated using the Mononobe-Okabe method of analysis as modified by Whitman (1990), and assumes a yielding wall condition.

g. For rigid (non-yielding) retaining walls, the resultant seismic force on the wall is $12H^2$ and acts at a point 0.6H up from the base of the wall.

Please note: Should the slope behind the retaining walls be other than shown in Table No.4, supplemental design criteria will be provided for the active earth or at rest pressures for the particular slope angle.

59. The above criteria are based on **fully drained conditions**. Therefore, we recommend that permeable material meeting the State of California Standard Specification Section 68-1.025, Class 1, Type A, be placed behind the wall, with a minimum width of 12 inches and extending for the full height of the wall to within 1 foot of the ground surface. The permeable material should be covered with Mirafi 140N filter fabric or equivalent and then compacted native soil placed to the ground surface. A 4 inch diameter perforated rigid plastic drain pipe should be installed within 3 inches of the bottom of the permeable material and be discharged to a suitable, approved location such as the project storm drain system. The perforations should be located and oriented on the lower half of the pipe. Neither the pipe nor the permeable material should be wrapped in filter fabric. Please refer to Figure No. 10, Typical Retaining Wall Drain Detail.

60. The area behind the wall and beyond the permeable material should be compacted with approved material to a minimum relative dry density of 90%.

SURFACE DRAINAGE

61. 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.

62. Surface water must not be allowed to pond or be trapped adjacent to the building foundations nor on the building pad nor in the parking areas.

63. All roof eaves should be guttered, with the outlets from the downspouts provided with adequate capacity to carry the storm water from the structures to reduce the possibility of soil saturation and erosion. The connection should be in a closed conduit which discharges at an approved location away from the structures and the graded area. The discharge location should not be located at the top of, or on the face of any topographic slopes. We would recommend a discharge point which is at least 10 feet down slope of any foundation or fill areas.

64. Final grades should be provided with a positive gradient away from all foundations in order to provide for rapid removal of the surface water from the foundations to an adequate discharge point. Soil grades should slope away from foundation areas at least 5 percent for the first 10 feet. Impervious surface areas should slope away from foundations at least 2 percent for the first 10 feet. The Project Civil Engineer, Architect or Building Designer should refer to 2007 CBC Section 1803.3 for further information. Concentrations of surface water runoff should be handled by providing necessary structures, such as paved ditches, catch basins, etc.

65. Cut and fill slopes shall be constructed so that surface water will not be allowed to drain over the top of the slope face. This may require berms along the top of fill slopes and surface drainage ditches above cut slopes. All cut, fill and disturbed native slope areas should be hydro-seeded or other means of erosion control provided, as determined by the Project Civil Engineer.

66. Irrigation activities at the site should not be done in an uncontrolled or unreasonable manner.

67. The building and surface drainage facilities must not be altered nor any filling or excavation work performed in the area without first consulting Pacific Crest Engineering Inc. Surface drainage improvements developed by the project civil engineer must be maintained by the property owner at all times, as improper drainage provisions can produce undesirable affects.

PAVEMENT DESIGN

68. 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 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. The use of "recycled" materials, such as asphaltic concrete for aggregate base or subbase is not recommended.

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

f. Use ½ inch maximum, Type "A" medium graded asphaltic concrete. Place the asphaltic concrete only during periods of fair weather when the free air temperature is within prescribed limits by Cal Trans Specifications.

g. Place ¼ gallon per square yard of SG-70 prime coat over the aggregate base section, prior to placement of the asphaltic concrete.

h. **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.**

i. Maintenance should be undertaken on a routine basis.

SOIL CORROSIVITY

69. Corrosivity tests were run on one representative surface soil sample collected on the project site. These results are summarized as follows:

TABLE No.5, Corrosivity Test Summary

Sample	Soil Resistivity	Chloride	Sulfate (water soluble)	pH
	Ohm-cm	mg/kg	mg/kg	
2-1-1	3737	8	<5	7.6

70. Cal Trans considers a site to be corrosive to foundation elements if one or more of the following conditions exist at the site:

a. The soil resistivity is less than 1,000 ohm-cm
b. Chloride concentration is greater than or equal to 500 mg/Kg (ppm)
c. Sulfate concentration is greater than or equal to 2000 mg/Kg (ppm)
d. The soil pH is 5.5 or less

Refer to Cal Trans Corrosion Guidelines, version 1.0 (September, 2003) for additional information.

71. Based on the results of the chloride, sulfate and pH, it appears that the conditions in the shallow existing soil should be assumed to be non-corrosive based on Cal Trans guidelines. The corrosion potential for any imported select fill should also be checked for corrosivity.

72. Please refer to Appendix A for the specific results of the corrosivity testing by the analytical laboratory.

PLAN REVIEW

73. 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.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. This Geotechnical Investigation was prepared specifically for you 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. Please refer to the ASFE "Important Information about Your Geotechnical Engineering Report" attached with this report.

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.

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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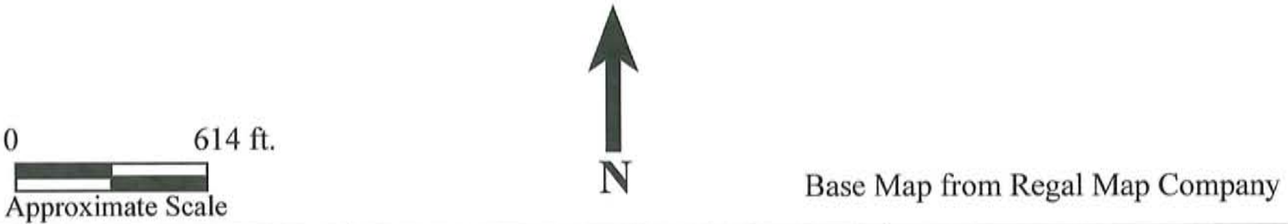
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

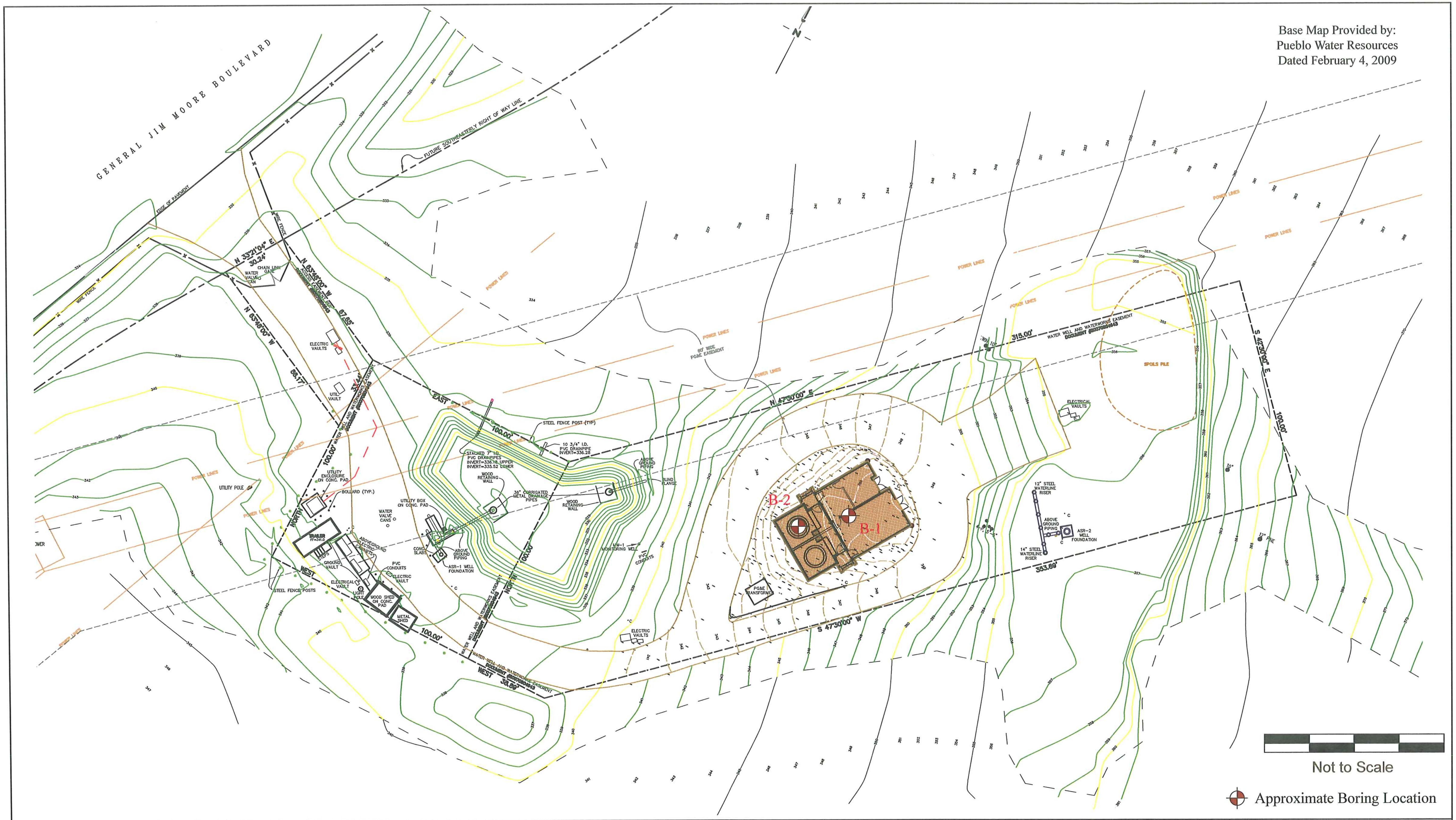
APPENDIX A

Regional Site Map
Site Map Showing Test Borings
Boring Log Explanation
Log of Test Borings
Caltrans Corrosivity Test Summary
Keyway Detail
Surcharge Pressure Diagram
Typical Retaining Wall Drain Detail



Base Map from Regal Map Company

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Regional Site Map** Electrical & Chemical Feed Building Seaside, California	Figure No. 1 Project No. 0922 Date: 4/30/09



Base Map Provided by:
Pueblo Water Resources
Dated February 4, 2009

Not to Scale

Approximate Boring Location

Pacific Crest Engineering Inc.
444 Airport Blvd., Suite 106
Watsonville, CA 95076

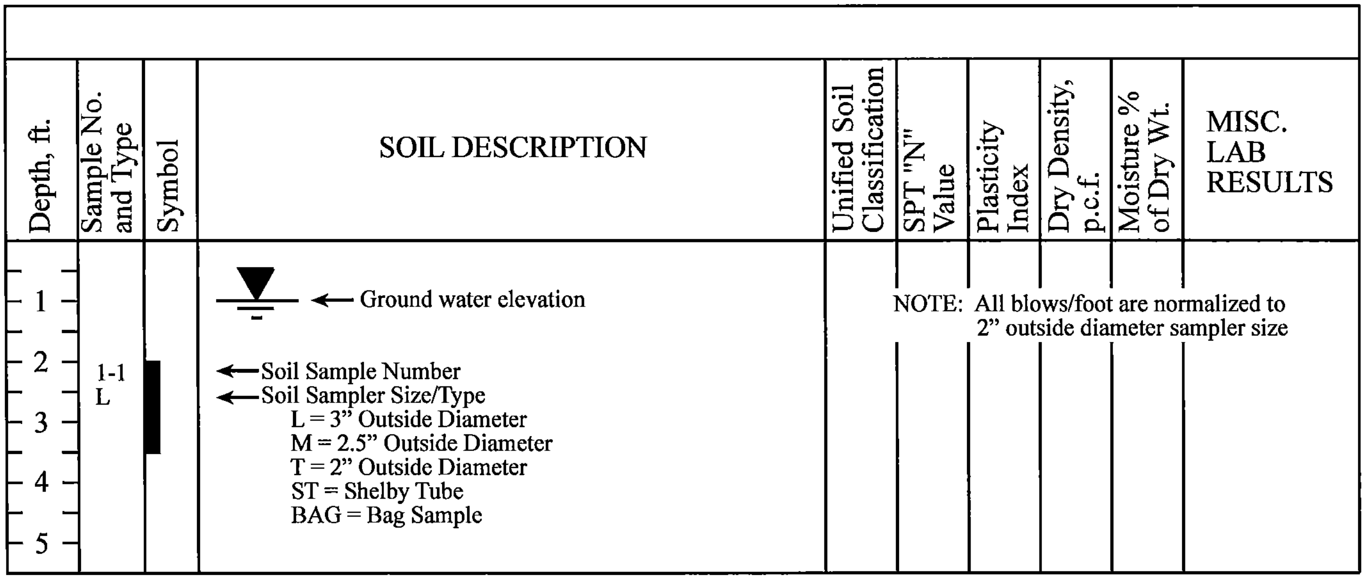
Site Map Showing Test Boring Locations
Electrical and Chemical Feed Building
Seaside, California

Figure No. 2
Project No. 0922
Date: 4/30/09

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488 (Modified)

PRIMARY DIVISIONS			GROUP SYMBOL	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 mixtures, 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 35%		ML	Inorganic silts and very fine clayey sand silty sands, with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly, sand, silty or lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS BETWEEN 35% AND 50%		MI	Inorganic silts, clayey silts and silty fine sands of intermediate plasticity
			CI	Inorganic clays, gravelly/sandy clays and silty clays of intermediate plasticity
			OI	Organic clays and silty clays of intermediate 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

BORING LOG EXPLANATION



NOTE: All blows/foot are normalized to 2” outside diameter sampler size

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

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Boring Log Explanation** Electrical & Chemical Feed Building Seaside, California	Figure No. 3 Project No. 0922 Date: 4/30/09

LOGGED BY CLR DATE DRILLED 4/17/09 BORING DIAMETER 6" BORING NO. 1

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1			Brown Silty SAND, fine to medium grained, sub-angular to sub-rounded shaped, poorly graded, mica flakes scattered throughout the sample, trace coarse grains scattered throughout the sample, damp, hard, (Older Dune Deposits)	SM					
2	1-1 L								
3					35		121.3	8.9	17.1% Passing #200 Sieve
4									
5	1-2 L		Color changed to dark reddish brown, trace rounded chert pebbles scattered throughout the sample, damp, medium dense						
6					17		108.7	5.4	
7									
8									
9									
10	1-3 L		Color change to yellowish tan, slight decrease in coarseness of sand, very fine to medium grained, slightly damp, medium dense						
11					18		108.4	2.0	
12									
13									
14									
15	1-4 L		Slight reddish tan mottling scattered throughout the sample, slight increase in coarseness of sand, damp, medium dense						
16					19		104.9	4.4	
17									
18									
19									
20	1-5 L		Lack of mottling, slight decrease in coarseness of sand, damp, medium dense						
21					30		100.2	3.5	
22									
23									
24									

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Log of Test Borings
Electrical & Chemical Feed Building
Seaside, California

Figure No. 4
Project No. 0922
Date: 4/30/09

LOGGED BY CLR DATE DRILLED 4/17/09 BORING DIAMETER 6" BORING NO. 1

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	1-6 L		Yellowish tan SAND, fine to medum grained, trace coarse grains, sub-angular to sub-rounded shaped, mica flakes scattered throughout the sample, poorly graded, damp, medium dense, (Older Dune Deposits)	SP	29		94.0	3.7	1.6% Passing #200 Sieve
26									
27									
28									
29									
30	1-7 L		Lack of coarse grains, slighlty damp, very dense		50/6"		103.6	4.0	
31									
32									
33									
34									
35	1-8 L		Slightly damp, very dense		50/5"		103.6	4.2	1.8% Passing #200 Sieve
36			Boring terminated at 36 feet. No groundwater encountered.						
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** Electrical & Chemical Feed Building Seaside, California	Figure No. 5 Project No. 0922 Date: 4/30/09

LOGGED BY CLR DATE DRILLED 4/17/09 BORING DIAMETER 6" BORING NO. 2

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1 2 3 4	2-1 L		Dark brown SAND with Silt, very fine to medium grained, sub-angular to sub-rounded shaped, poorly graded, trace rounded chert pebbles scattered throughout the sample, trace granitic gravel near 3 1/2 feet, mica flakes scattered throughout the sample, damp, medium dense	SP-SM	17				
5 6 7 8 9	2-2 L		Dark reddish brown SAND with Silt, fine to medium grained, trace coarse grains, sub-angular to sub-rounded shaped, poorly graded, mica flakes scattered throughout the sample, damp, loose, (Older Dune Deposits)	SP-SM	7		110.8	6.4	10.0% Passing #200 Sieve
10 11 12 13 14	2-3 L		Color change to yellowish tan, trace rounded chert pebbles scattered throughout the sample,, slightly damp, medium dense		19		99.4	2.8	
15 16	2-4 L		Color change to tan, lack of rounded chert pebbles, slightly damp, medium dense		23		99.2	3.2	
17 18 19 20 21 22 23 24			Boring terminated at 16 1/2 feet. No groundwater encountered.						

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Watsonville, CA 95076

Log of Test Borings
Electrical & Chemical Feed Building
Seaside, California

Figure No. 6
Project No. 0922
Date: 4/30/09

Corrosivity Test Summary

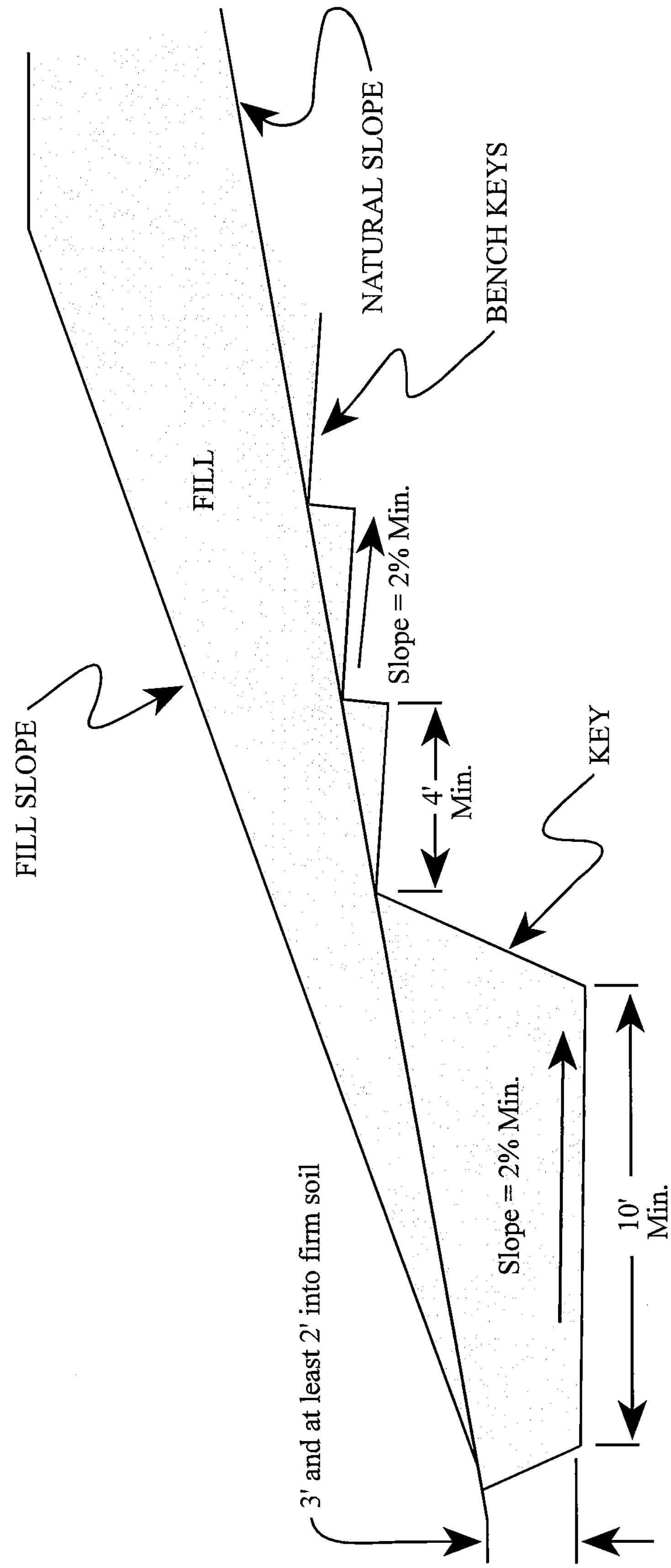
CTL # 416-391 **Date:** 4/30/2009 **Tested By:** PJ **Checked:** PJ

Client: Pacific Crest Engineering **Project:** New Electric & Chemical Feed Building **Proj. No:** 0922

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	As Received	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
-	2-1-1	-	-	3737	-	8	<5	<0.0005	7.6	-	9.2	Dark Brown Silty SAND

Figure No. 7
Project No. 0922
Date: 4/30/09



TYPICAL KEY AND BENCHES

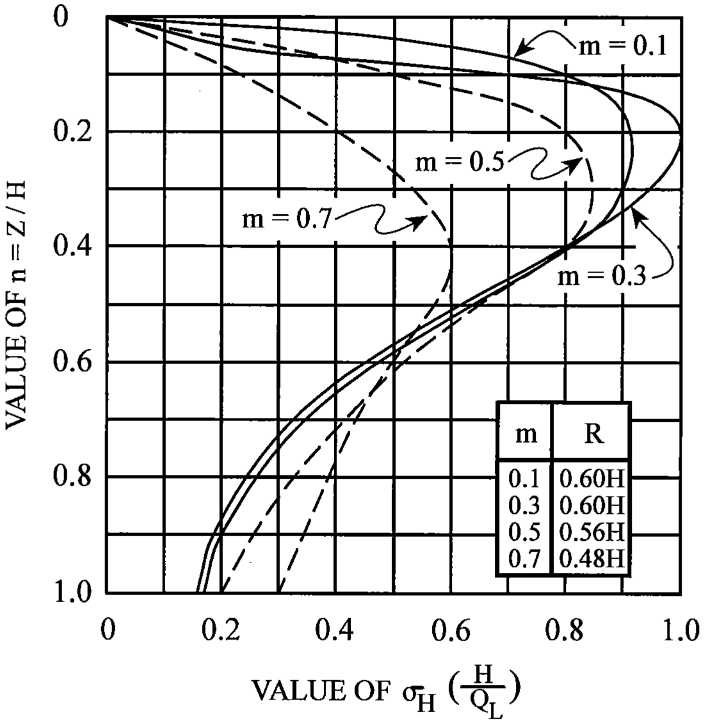
not to scale

Pacific Crest Engineering Inc.
444 Airport Blvd., Suite 106
Watsonville, CA 95076

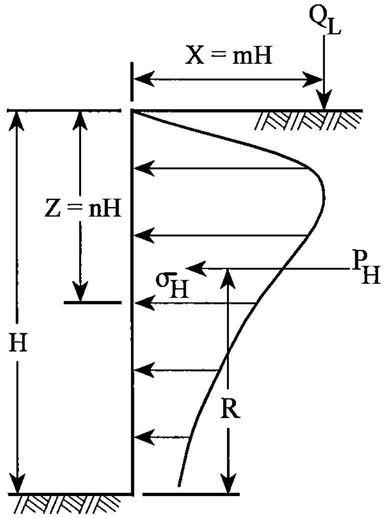
Keyway Detail
Electrical & Chemical Feed Building
Seaside, California

Figure No. 8
Project No. 0922
Date: 4/30/09

LINE LOAD



m	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H



FOR m ≦ 0.4:

$$\sigma_H \left(\frac{H}{Q_L}\right) = \frac{0.20\,n}{(0.16+n^2)^2}$$

$$P_H = 0.55\,Q_L$$

FOR m > 0.4:

$$\sigma_H \left(\frac{H}{Q_L}\right) = \frac{1.28\,m^2 n}{(m^2+n^2)^2}$$

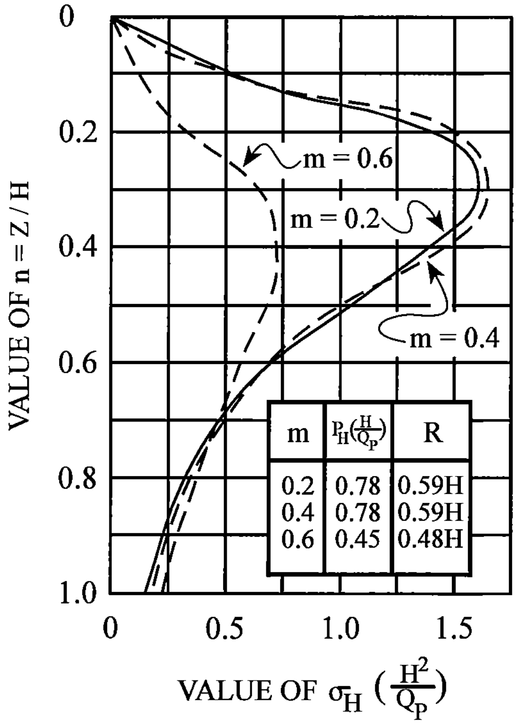
$$\text{RESULTANT } P_H = \frac{0.64\,Q_L}{(m^2+1)}$$

PRESSURES FROM LINE LOAD Q_L

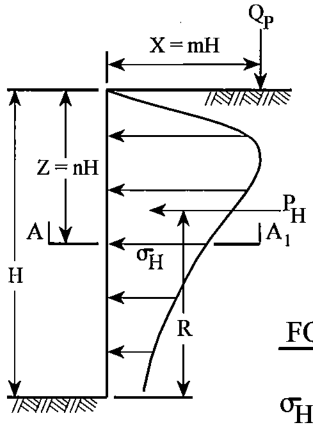
(BOISSINESQ EQUATION MODIFIED BY EXPERMENT)

REFERENCE: Design Manual
NAVFAC DM-7.02
Figure 11
Page 7.2-74

POINT LOAD



m	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



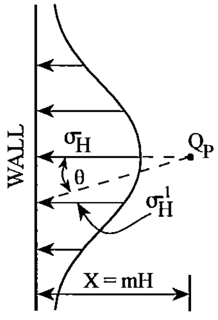
FOR m ≦ 0.4:

$$\sigma_H \left(\frac{H^2}{Q_P}\right) = \frac{0.28\,n^2}{(0.16+n^2)^3}$$

FOR m > 0.4:

$$\sigma_H \left(\frac{H^2}{Q_P}\right) = \frac{1.77\,m^2 n^2}{(m^2+n^2)^3}$$

$$\sigma_H^1 = \sigma_H \cos^2(1.1\,\theta)$$

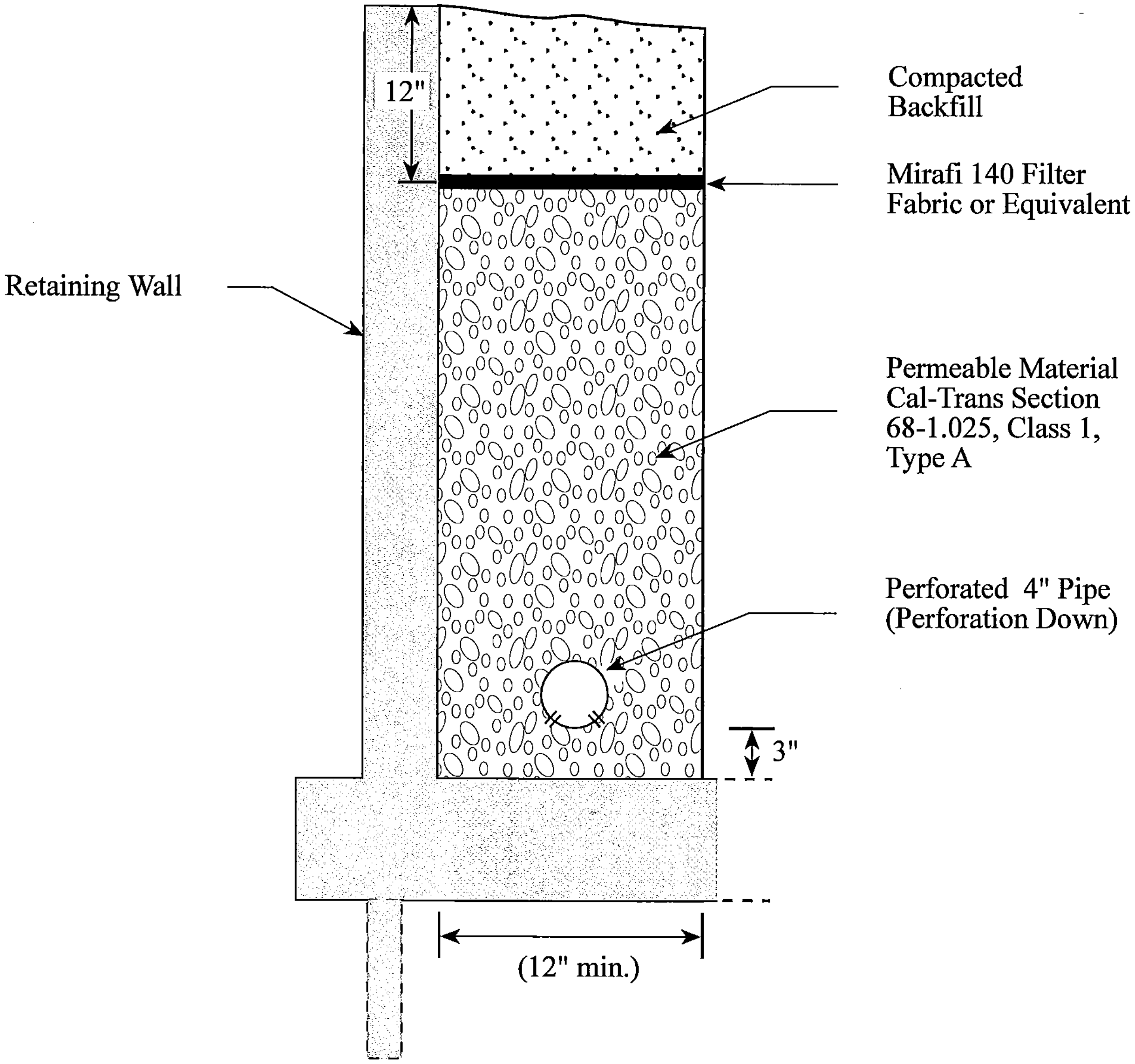


SECTION A-A_1

PRESSURES FROM POINT LOAD Q_P

(BOISSINESQ EQUATION MODIFIED
BY EXPERMENT)

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Surcharge Pressure Diagram-1** Electrical & Chemical Feed Building Seaside, California	Figure No. 9 Project No. 0901 Date: 4/30/09



Not to Scale

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Typical Retaining Wall Drain Detail** Electrical & Chemical Feed Building Seaside, California	Figure No. 10 Project No. 0901 Date: 4/30/09