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GEOTECHNICAL INVESTIGATION
For
INCLUSIONARY HOUSING
Area D
Pebble Beach, California

Prepared For
Pebble Beach Company
Pebble Beach Company, California

Prepared By
HARO, KASUNICH AND ASSOCIATES, INC.
Geotechnical & Coastal Engineers
Project No. M10473
April 2013

Project No. M10473
30 April 2013

PEBBLE BEACH COMPANY
c/o Ms. Cheryl Burrell
P.O. Box 1767
Pebble Beach, California 93953

Subject: Geotechnical Investigation

Reference: Inclusionary Housing Development
Area D Del Monte Forest Plan
Congress Road
Pebble Beach, California

Dear Ms. Burrell:

In accordance with your authorization, we have performed a Geotechnical Investigation for the referenced project in Pebble Beach, 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 or conclusions or recommendations presented in this report, please call our office.

Respectfully Submitted,

HARO, KASUNICH & ASSOCIATES, INC.

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GEOTECHNICAL INVESTIGATION

Introduction

This report summarizes our findings, conclusions and recommendations from our geotechnical investigation for the inclusionary housing development located at Area D – Del Monte Forest Plan, Pebble Beach, California.

Our Geologic Report for this project site is dated 29 April 2013. A septic feasibility report is not part of this study. We presume the development will be connected to a municipal sewer line.

A topographic survey map for the referenced project site, dated 26 February 2013, was prepared by L & S Engineering And Surveying, Inc. This was used as a base map for our Test Bore Hole Location Map (see Figure 2 in Appendix A). The topographic survey map shows the footprints of the design concept that includes four groups of multi-unit family dwellings, carports, access driveway, parking areas, and a common facility.

As the project plans have not been finalized, some of the recommendations presented in this report are general in nature. We should be provided an opportunity to review project plans once they have been developed to verify that the intent of our geotechnical recommendations have been met.

Purpose and Scope

The purpose of our investigation was to explore and evaluate subsurface conditions at the site and to provide geotechnical criteria and recommendations for design and construction of the proposed project. The specific scope of our services was as follows:

1. Review data in our files pertinent to the site.
2. Explore the subsurface conditions at the site with nine (9) test bore holes drilled from 6.5 feet to 25 feet deep.
3. Field and laboratory testing of selected soil samples to determine their pertinent engineering properties.
5. Perform non-standard falling head tests within select test bore holes between 2.5 feet to 8.7 feet deep.
4. Analyze the resulting data to develop geotechnical recommendations for building foundations, retaining walls, slabs-on-grade, pavements, and site grading.
5. Present the results of our investigation in this report.

Site Location and Conditions

The project site is located on an undeveloped parcel on the north side of the intersection of S.F. B. Morse Drive and Congress Road. The site is bound by an existing residential development to the northwest, Congress Road to the southwest and northeast, and S.F.B. Morse Drive to the south. The site is thickly forested with oak trees, pine trees, stumps, shrubs, grass, bushes, and a blanket of organic forest debris that covers the ground surface. There are several walking paths that meander around the site. See Figure No. 1 for site vicinity.

Project Description

Based on the preliminary plan sheet provided to us, we understand the project consists of the development of inclusionary housing. Four clusters of multi-unit single family dwellings are shown equally spaced apart. Six units are shown per cluster each with three car ports per cluster. A driveway is shown with frontage at two locations on Congress Road. Parking areas are shown along the northwest side of the driveway. A common area facility is shown centrally located on the northwest side of the driveway.

Field Exploration

Subsurface conditions were explored on 26 March 2013 by drilling nine (9) test borings to depths of 6.5 feet to 25 feet. The borings were advanced with 6-inch diameter continuous flight auger equipment mounted on a tractor.

Representative soil samples were obtained from the exploratory borings at selected depths, or at major strata changes. These samples were recovered using a 3.0 inch O.D. Modified California Sampler (L), or by a Standard Terzaghi Sampler (T). The soils encountered in the borings were continuously logged in the field and described in accordance with the Unified Soil Classification System (ASTM D2488, Visual-Manual Proceeding). The Logs of Test Borings are included in the Appendix of this report. The logs depict subsurface conditions at the approximate locations shown on the Site Plan. The subsurface conditions at other locations may differ from those encountered at the explored locations. Stratification lines shown on the logs represent the approximate boundaries between soil types. The actual transitions may be gradual.

The penetration blow counts noted on the boring logs were obtained by driving a sampler into the soil with a 140-pound hammer dropping through a 30-inch fall. The sampler was driven up to 18 inches into the soil and the number of blows counted for each 6 inch penetration interval. The numbers indicated on the logs are the total number of blows that were recorded for the second and third 6-inch intervals, or the blows that were required to drive the penetration depth shown if high resistance was encountered.

Laboratory Testing

Soil samples obtained from the borings at selected depths were taken to our laboratory for further examination and laboratory testing. The laboratory testing program was directed toward determining pertinent engineering properties of soil underlying the project site.

Natural moisture contents and dry densities were determined on selected samples and are recorded on the boring logs at the appropriate depths. Since water has a significant influence on soil, the natural moisture content provides a rough indicator of the soil's compressibility, strength, and potential expansion characteristics.

Atterberg Limits tests were performed on a clayey soil samples to aid in classification as well as to help estimate the degree of expansion.

The strength parameters of the underlying earth materials were determined from field penetration resistance of the in-situ soil, saturated direct shear tests, and an unconfined compression test performed in the laboratory.

The results of the laboratory testing appear on the "Logs of Test Boring" opposite the sample tested.

Subsurface Conditions

The general soil profile at the site consisted of loose or stiff soil to depths of 1 foot to 3 feet over medium dense or very stiff soil to depths of around 5 feet to 15 feet. Dense soil was encountered at depths of 5 to 15 feet on the northwest half of the site and 3 to 5 feet on the southeast half. Very dense soil was encountered at the project site at depths of 17 feet on the southeast half, 21 feet in the approximate center, and 25 feet on the northwest half.

In general, the loose and stiff soil was comprised of silty sands, sands, and occasional lenses of sandy lean clay. The loose soil zone was contaminated with roots and organics in the top 12 inches to 18 inches and could extend deeper in some areas. The loose soil stratum was typically moist with moisture contents from 7.4 percent to 16.4 percent in the granular soil and 22.5 percent in the clay soil. Based on the measured Atterberg Limits (P.I. = 19 & 29), the swell potential of the sandy lean clay within this stratum is moderate. Where foundation excavations expose sandy lean clay, it should be moisture conditioned 3 to 5 percent over the laboratory determined optimum moisture content within 24 hours of concrete placement.

The medium dense or very stiff soil was comprised of silty sand, sand, clayey sand, and occasional lenses of sandy lean clay. The medium dense soil stratum was typically moist to wet and occasionally saturated. Moisture contents ranged from 9.9 percent to 17.5 percent. Based on the measured Atterberg Limits (P.I. = 12), the swell potential of the sandy lean clay within this stratum is low.

The dense soil was comprised of sand and clayey sand. In general the clayey sands were oxidized and lightly cemented in some locations. We interpret these stratums as weathered sandstone. The dense soils were moist to wet and saturated below the groundwater level. Moisture contents ranged from 15.7 percent to 18.0 percent.

The very dense sand encountered in Test Bore Hole 4 was a weathered granitic bedrock that required a minimum 100 blows to drive the sampler 1 foot. Moisture content was damp to dry. In Test Holes 5 and 7 the depth to the bedrock was determined by advancing the solid flight auger until refusal was encountered.

Root Zone and Disturbed Soils

The project site is thickly forested with pine and oak trees. A thick layer of forest debris blankets the site. Top soil with roots was encountered in our test bore holes to a depth of 18 inches below the ground surface. The actual depth of the root zone is unknown at this time and beyond the scope of this report. Removal of dense clusters of tree roots may disturb the top 2 feet to 3 feet or more of soil below the ground surface. The disturbed soil layer is un-suitable for foundation support. The organic content of the soil within the root zone must be removed if the soil is to be re-used as engineered fill.

Groundwater

Perched groundwater was encountered within Test Bore Holes B1, B2, B4, B5, B7, and B8 at depths of 5 feet to 13 feet below the ground surface. The groundwater was perched upon the granitic bedrock formation. Groundwater was also observed seeping out of the cut bank along the road shoulder of Congress Road on the northwest side of the site. It should be noted that groundwater levels may fluctuate due to variations in rainfall or other factors not evident during our investigation. Proposed improvements and earthwork construction should be planned with consideration that perched groundwater could rise and daylight into crawlspaces and excavations.

Erosion

The loose soil stratum at this site is highly erodible and erosion can be severe where there are steep slopes and uncontrolled runoff, particularly where the natural drainage is modified by the works of man and not properly controlled. Typically, once the upper surface of the fill material is breached by a rill or a gully, erosion proceeds at an accelerated rate, and the rills and gullies deepen and migrate headward (upslope). This process may contribute to the initiation of shallow debris flows if rills and gullies are not mitigated or maintained and if surface drainage controls are not adequately designed and constructed. Due to the gentle grades at this site, a shallow debris flow would be a landscape maintenance issue, but none the less would need to be repaired.

Percolation Testing

Percolation testing of the subsurface soil within Area D was performed. Percolation test holes were filled with a vertical 3 inch diameter ADS pipe with the bottom 18 inches slotted. The bottom 24 inches of annulus space between the walls of the test holes and the ADS pipe were filled with drain rock and remaining space filled with native soils up to the ground surface. After installation, the bottom 12 to 18 inches of each test hole was pre-saturated 24 hours prior to the percolation tests.

The percolation testing commenced by adding or maintaining water levels until the level reach 12 inches from the bottom of the pipe. A non standard falling head test was then performed by taking a water level reading every 30 minutes for four hours. Test holes that ran dry during the four hour test period were refilled as needed and testing continued (B-3). Test holes that ran dry before the first 30 minute reading were read and refilled every 10 minutes for an hour (B-1).

Two percolation test holes were set up at locations B-1, B-3, B-6, and B-8. The test holes had percolation zones set at different elevations in each location. Typically percolation zones ranged from 2 feet to 5 feet deep for the shallower of the two holes and 5 feet to 8 feet deep for the other hole. The purpose of this test was to determine a falling head percolation rate at select depths below the ground surface. The percolation zones were selected based on the depth of perch ground water in each area. The bottoms of the deeper percolation test holes were a minimum of 2 feet above the perched groundwater. The deep percolation test hole at B-1 and the shallow percolation test hole at B-8 were filled in with soil by vandals.

In general, percolation zones between 1.5 feet deep to 4 feet deep had percolation rates from 12 inches per hour to 36 inches per hour. The same can be assumed for the shallow percolation test hole at location B-8. Water level was observed dropping quickly during its pre-saturation prior to it being vandalized. Percolation zones greater than 4 feet deep had percolation rates of 0 inches per hour. The poor percolation rates of the deeper test holes can be attributed to perched ground water and localized lenses of clay. The final percolation rate presented in Figures 17 to 22 should be used by the civil engineer of record in their drainage retention (soil infiltration) design.

Seismicity

The following is a general discussion of seismic considerations affecting the project area. Detailed studies of seismicity, faulting and other geologic hazards are beyond the scope of our geotechnical investigation for this project. Refer to our Geological Report for this project site dated 29 April 2013. It is highly probable that a major earthquake will occur in northern California during the next 50 years. During a major earthquake epicentered nearby, there is a potential for severe ground shaking at this site. Structures designed in accordance with the most current California Building Code (CBC) should react well to seismic shaking.

2010 CBC Geotechnical Related Seismicity

The new earth retention structures should be designed in conformance with the most current California Building Code (2010 CBC). For seismic design, the soil properties at the site are classified as **Site Class "C"** based on definitions presented in Table 1613.5.2 in the 2010 CBC. The longitude and latitude were determined using a satellite image generated by Google Earth. These coordinates were taken from the middle of the area of the proposed improvements:

Longitude = -121.935931, Latitude = 36.599316

The coordinates listed above were used as input in the Java Ground Motion Parameter Calculator created by the USGS to determine the ground motion associated with the maximum considered earthquake (MCE) SM and the reduced ground motion for design SD. The results are as follows:

<u>Site Class C</u>
SM_s= 1.528
SM_1= 0.820
SD_s= 1.019
SD_1= 0.547

Using $SD_s/2.5$ a PGA of 0.41g is determined. This results in a seismic surcharge as a lateral force equivalent to $13H^2$ lbs/ft located at 0.6H above the ground surface.

Liquefaction

During an earthquake seismic waves travel through the earth and vibrate the ground. In cohesionless, granular materials having low relative density (e.g., loose sands), this vibration can disturb the particle framework leading to increased compaction of the material and reduction of pore space between the framework grains.

If the sediment is saturated, water occupying the pore spaces resists this compaction and exerts pore pressure that reduces the contact stress between the sediment grains. With continued shaking, transfer of inter-granular stress to pore water can generate pore pressures great enough to cause the sediment to loose strength and change from a solid state to a liquid state (i.e. liquefaction).

Liquefaction can lead to several types of ground failure, depending on slope conditions and the geologic and hydrologic setting (Seed, 1968; Youd, 1973; Tinsley et al, 1985). The four most common types of ground failure are: lateral spreading; flow failures; ground oscillation; and loss of bearing strength.

Within our test bore holes for this project, we encountered loose sandy soils within the top 1 foot to 3 feet below the ground surface. Perched groundwater was encountered between 5 feet to 11 feet below the ground surface. If the groundwater were to rise to the surface the loose sandy soils have potential to liquefy. However, we have recommended that foundation elements penetrate loose soils and be embedded into firm native soil or the loose soils must be removed and replaced as organic free engineered fill. If buildings are constructed in accordance with the recommendations of this report the potential that liquefaction will impact the proposed improvements is low to nil.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, the proposed development is feasible from a geotechnical standpoint, provided the design criteria and recommendations presented in this report are incorporated into the design and construction of the project.

Geotechnical considerations at the proposed site include providing firm, uniform bearing support for foundations, concrete slabs and pavements, moisture conditioning of clay soil within foundation trenches, uniform support of concrete slab-on-grade floors, perched groundwater, the potential for seismic shaking, and liquefaction potential (lose of bearing capacity) of loose near surface soils.

To provide a uniform bearing zone for foundation support, we recommend a conventional reinforced concrete spread foundation system embedded into either un-disturbed firm native soil or a mat of recompacted engineered fill. Buildings should not be supported by a combination of firm native soil and engineered fill.

During removal of dense clusters of trees and their roots large voids and disturbance of soil is estimated from 1 foot to 3 feet deep. Building foundations supported by firm native soil should be deepened in these areas to achieve the required embedment into un-disturbed native soil. To re-establish finished grade to support concrete slabs, flatwork, and promote positive drainage away from improvements the voids and disturbed soil areas can be replaced with a mat of engineered fill prepared in accordance with recommendations of the section titled “Site Grading”.

Foundation trenches that expose clay soil should be moisture conditioned in accordance with the recommendations in this report. As an alternative the foundation trenches can be deepened to penetrate the clay soil into the medium dense or dense sandy type soils below.

As an alternative to deepened foundations a mat of engineered fill can be constructed and conventional foundations embedded into it to the design depths. The mat of engineered fill should be constructed in accordance with the recommendations of the section titled “Site Grading”.

To reduce the potential of perched groundwater damaging improvements curtain drains should be constructed on the northeast (uphill side) of buildings, on the downhill side of crawlspaces, and other moisture sensitive improvements.

The following recommendations should be used as guidelines for preparing project plans and specifications, and assume that **Haro, Kasunich & Associates** will be commissioned to review project grading and foundation plans before construction and to observe, test and advise during earthwork and foundation construction. This additional opportunity to examine the site will allow us to compare subsurface conditions exposed during construction with those inferred from this investigation. Unusual or unforeseen soil conditions may require supplemental evaluation by the geotechnical engineer.

Site Grading

1. The geotechnical engineer should be notified **at least four (4) working days prior to any grading or foundation excavating** so the work in the field can be coordinated with the grading contractor and arrangements for testing and observation can be made. The recommendations of this report are based on the assumption that the geotechnical engineer will perform the required testing and observation during grading and construction. It is the owner's responsibility to make the necessary arrangements for these required services.

2. Where referenced in this report, Percent Relative Compaction and Optimum Moisture Content shall be based on ASTM Test Designation D1557.

3. Areas to be graded should be cleared of all obstructions, including trees not designated to remain and other unsuitable material. Existing depressions or voids created during site clearing should be backfilled with engineered fill.

4. Cleared areas should then be stripped of organic-laden topsoil. Stripping depth is anticipated to be from 6 to 12 inches. Actual depth of stripping should be determined in the field by the geotechnical engineer. Strippings should be wasted off-site or stockpiled for use in landscaped areas if desired.

5. Building foundations and concrete slab-on-grade floors should be uniformly supported either by firm native soil or by a mat of engineered fill. Building foundations and interior concrete slab floors should not be supported by a combination of native soils and engineered fill.

6. The mat of engineered fill should be a minimum 3 feet thick and extend a minimum 18 inches below the bottom of the foundation element and 5 horizontal feet beyond foundation edges in all directions. For support of concrete slabs, pavements, and flatwork the mat of engineered fill should be a minimum 18 inches thick and extend 3 horizontal feet beyond improvements in all directions.

7. The mat of engineered fill should have a maximum change in thickness of 4 feet across the pad see Figures 25 and 26.

8. Engineered fill should be placed in thin lifts not to exceed 8 inches in loose thickness, moisture conditioned, and compacted to a minimum of 90 percent relative

compaction. The upper 8 inches of pavement and concrete slab subgrades should be compacted to at least 95 percent relative compaction. The aggregate base below pavements should likewise be compacted to 95 percent relative compaction see Figure 28.

9. We estimate shrinkage factors of about 15 percent for the on-site materials when used in engineered fills.

10. Any imported fill should meet the following criteria:

   a. Be free of wood, brush, roots, grass, debris and other deleterious materials.
   b. Not contain rocks or clods greater than 2.5 inches in diameter.
   c. Not more than 20 percent passing the #200 sieve.
   d. Have a plasticity index less than 15.
   e. Be evaluated for conformance to the aforementioned requirements by the geotechnical engineer. Submit to the geotechnical engineer samples of import material or utility trench backfill for compliance testing a minimum of 4 days before it is delivered to the job site.

11. Perched groundwater was encountered at the project site between 5 feet and 13 feet below the ground surface. As a result the grading contractor may encounter compaction difficulty (i.e. pumping action and/or the bringing of free water to the sur-

face) in subgrade preparation, especially if grading is performed during or shortly after the inclement weather season. If compaction cannot be achieved after adjusting the soil moisture content, it may be necessary to stabilize the subgrade soil with angular crushed rock. The bridging material should be a coarse granular mixture of rock having a maximum size of about 3 inches. It is anticipated that quarry run or crusher run materials will be satisfactory. The material should be well graded between the largest and smallest particle size, with no more than 12 percent passing the # 200 sieve.

Curtain Drains

12. Curtain drains should be constructed on the uphill side of buildings and other moisture sensitive improvements see Figure 23 and 24.

13. Curtain drains should be constructed within crawlspaces on the downstream side of buildings see Figure 24A.

14. Curtain drain trenches should be a minimum 12 inches wide and extend 12 inches below the bottom of downstream improvements that are to be drained. A 4 inch diameter perforated pipe should be placed on a thin bed of approved gravel at the bottom of the trench. The trench should be backfilled to within 12 inches of the ground surface with Class 1, Type A permeable gravels. The top 12 inches of the trench should

be capped with native soils compacted in accordance with the recommendations of our geotechnical investigation for this project see Figures 23, 24, and 24A.

15. Curtain drains, bench drains, and retaining wall back drains can be combined. The drain pipes for each subsurface drain should be discharged independently or connected significantly down slope from improvement see Figure 25 and 26.

Fill Slopes (See Figure 27)

16. Compacted fill slopes should be constructed at a slope inclination not steeper than 2:1 horizontal to vertical. Fill slopes with these recommended gradients may require periodic maintenance to remove minor soil sloughing.

17. All fill slopes placed on slopes with gradients in excess of 5:1 (horizontal to vertical) must be adequately keyed and benched into competent material. The toe key should be at least 8 feet wide and should extend at least 18 inches into firm native soil. The bottom of the toe key should be sloped downward at about 2 percent toward the back of the key.

18. The bottom of toe keys and subsequent bench keys should be scarified to a depth of 8 inches; moisture conditioned, and compacted to a relative compaction of 90 percent.

19. There should be a minimum of 10 feet horizontal separation between the bottom of footing elements and the top of a fill slope or the base of a cut slope.

20. In order to maintain stable slopes at the recommended gradients, it is important that seepage forces and accompanying hydrostatic pressure be relieved by adequate drainage. Adequate backdrains in keyways and benches should be provided. The locations of backdrains and outlets will be determined by the geotechnical engineer in the field during grading.

21. Permanent cut slopes up to 10 feet in height should be inclined no steeper than 2:1 (H:V) into medium dense soil and 1:1 (H:V) into dense soils. Where steeper cut slopes are proposed they should be approved in the field by the Geotechnical Engineer. Excavations should be properly shored and braced during construction to prevent sloughing and caving at sidewalls. The contractor should be aware of all CAL OSHA and local safety requirements and codes dealing with excavations and trenches.

22. Following grading, exposed soil should be planted as soon as possible with erosion resistant vegetation.

23. After the earthwork operations have been completed and the geotechnical engineer has finished his observation of the work, no further earthwork operations shall be performed except with the approval of and under the observation of the geotechnical engineer.

Building Foundations - Spread Footing Foundation System

24. The proposed residence may be supported on conventional or deepened spread footings embedded into firm native soils or a mat of engineered fill prepared in accordance with the section of this report titled "Site Grading".

25. Where clay soil or elastic silts are exposed the foundation trenches should be moisture conditioned to 3 to 5 percent over the optimum moisture content within 24 hours of concrete placement. The Geotechnical Engineer should verify this has been done. As an alternative the foundation elements could be deepened to penetrate clay and elastic silt soil or clay soils can be removed and replaced with a non-expansive engineered fill.

26. Foundation elements supporting one story and two buildings should be embedded a minimum of 12 inches and 18 inches respectively into firm native soil of engineered fill. Foundation elements should be sized and reinforced in accordance with

applicable building codes and standards. Foundations should also be designed with respect to live and dead loads determined by the structural engineer.

27. Interior load-bearing walls and concentrated loads should be supported on continuous reinforced concrete foundations that are structurally connected at each end to the continuous perimeter foundations to create a grid system. Spacing of interior continuous foundations for the grid pattern will depend on the specific structure; however spacing of 20 feet could be used as an initial guideline. Non-bearing interior continuous foundations should be embedded a minimum of 12 inches into firm native soil or engineered fill.

28. The foundation trenches should be kept moist and be thoroughly cleaned of all slough or loose materials prior to pouring concrete. In addition, all footings located adjacent to other footings or utility trenches should have their bearing surfaces founded below a 1.5:1 line projected upward from the bottom edge of the adjacent footings or utility trenches.

29. Foundations designed in accordance with the above may be designed for an allowable soil bearing pressure of 2,000 psf for dead plus live loads. This value may be increased by one-third to include short-term seismic and wind loads.

30. Lateral load resistance for structures supported on spread footings may be developed in friction between the foundation bottom and the supporting subgrade. A friction coefficient of 0.35 is considered applicable.

31. All footings should be reinforced in accordance with applicable CBC and/or ACI standards; however, we recommend the footings contain a minimum steel reinforcement of four (4) No. 4 bars; i.e., two near the top and two near the bottom of the footing.

32. The footing excavations should be thoroughly cleaned and observed by the geotechnical engineer prior to placing forms and steel, to verify subsurface soil conditions are consistent with the anticipated soil conditions and the footings are in accordance with our recommendations.

Retaining Wall Lateral Pressures

33. Retaining walls should be designed to resist both lateral earth pressures and any additional surcharge loads. For design of retaining walls up to 10 feet high and fully **drained**, the following design criteria may be used. **For undrained loading conditions double the recommended lateral earth pressures:**

A. Active earth pressure for walls allowed to yield is that exerted by an equivalent fluid weighing 35 pcf for a level backslope gradient; and 50 pcf for a 2:1 (horizontal to vertical) backslope gradient. This assumes a fully drained condition.

B. Where walls are restrained from moving at the top, design for a uniform rectangular distribution equivalent to 24 H psf per foot for a level backslope, and 35 H psf per foot for a 2:1 backslope, where H is the height of the wall.

C. Use a coefficient of friction between base of foundation and native soil of 0.35. Alternatively, where retaining wall footings are poured neat against firm native soil, a passive resistance of 300 pcf (EFW) may be used. The top 12 inches and all topsoil or other loose materials should be neglected when computing passive resistance.

D. In addition, the walls should be designed for any adjacent live or dead loads which will exert a force on the wall (garage and/or auto traffic) see Figure 29.

E. Retaining walls that act as interior house walls should be thoroughly waterproofed. Consult with a water proofing expert.

34. To account for seismic loading, a horizontal line load surcharge equal to $13H^2$ pounds per linear foot of wall may be assumed to act at 0.6H above the base of the wall (where H is the height of the wall), if the retaining wall is part of or supporting a critical structure.

35. The above lateral pressure values assume that the walls are fully drained to prevent hydrostatic pressure behind the walls. Drainage materials behind the wall should consist of either Class 1; Type A permeable material complying with Section 68 of Caltrans Standard Specifications, latest edition or an approved equivalent.

36. The drainage material should be at least 12 inches thick. The drains should extend from the base of the walls to within 12 inches of the top of the backfill. A perforated pipe should be placed (holes down) about 4 inches above the bottom of the wall and be tied to a suitable drain outlet. Wall backdrains should be capped at the surface with clayey material to prevent infiltration of surface runoff into the backdrains. Filter fabric (Mirafi 140N or equivalent) should separate the subdrain material from the overlying soil cap see Figure 23.

37. For basement walls the base of the gravel drainage column should be made impermeable so as to allow collected water to reach the drainpipe rather than seep beneath the wall foundation.

Concrete Slabs-on-Grade

38. Building floor slabs and exterior slabs should be constructed on properly water conditioned and compacted soil subgrades. Soil subgrades should be prepared and compacted as recommended in the section entitled "Site Grading".

39. Concrete slabs-on-grade planned for the site should be constructed on firm native soil or engineered fill as outlined in the grading section of this report. To provide uniform support the subgrade should be scarified, moisture conditioned, and compacted to minimum 95 percent relative compaction. Prior to construction of the slab, the subgrade surface should be proof-rolled to provide a smooth, firm, uniform surface for slab support.

40. The change in thickness of the mat of engineered fill supporting the slab should not exceed 4 feet across the footprint of the slab. See Figure 25 and 26.

41. To reduce the potential for cracking and curling as well as other undesirable defects the concrete slab-on-grade design, placement, and curing should be done in accordance with ACI 302.1R-04.

42. In areas where floor wetness would be undesirable, a blanket drain comprised of 4 inches of Class I Type A permeable should be placed beneath the floor slab to act as

a capillary break. Whether or not a vapor barrier should be used and the location of its placement below the slab should be decided following the criteria set forth is ACI 302.1R-04. The granular material in ACI 302 should be 4 inches of aggregate baserock compacted to a minimum 95 percent relative compaction.

43. Whether to pour directly onto a vapor barrier or place a layer of aggregate baserock over the vapor barrier should be determined following ACI 302 Figure 3. Other factors influence this decision such as the installation of roof membranes and slab curing procedures. Refer to ACI 302 for more information. A 4 inch layer of compacted aggregate baserock (not natural sand) is recommended as the granular material.

44. Vapor barriers should have permanence (water vapor transmission rating) of 0.00 perms when tested in accordance with ASTM E 96. The laps or seams should be overlapped 6 inches or as instructed by manufacturer.

45. Concrete slabs without vapor barriers should not be poured in direct contact with the capillary break (high friction). This will increase the potential for cracking related to volume change of the slab and is not recommended. A 4 inch thick layer of compacted aggregate baserock should be placed on top of the capillary break.

46. The exterior slab reinforcement should not be tied to the building foundations. These exterior slabs can be expected to suffer some cracking and movement. However, thickened exterior edges, a well-prepared subgrade including pre-moistening prior to pouring concrete, adequately spaced expansion joints, and good workmanship should minimize cracking and movement.

Site Drainage

47. Surface drainage should include provisions for positive gradients so that surface runoff is not permitted to pond adjacent to foundations and pavements. Surface drainage should be directed away from the building foundations.

48. Rain gutters should be placed around roof eaves. Discharge from the roof gutters should be conveyed away from the downspouts by solid pipe and dispersed into energy dissipaters located a minimum of 20 feet downslope of the home site.

49. The migration of water or spread of extensive root systems below foundations, slabs, or pavements may cause undesirable differential movements and subsequent damage to these structures. Landscaping should be planned accordingly.

50. ***Due to the potential for perched water to develop on the dense decomposed granite, subdrains should be constructed on the uphill side of improvements.*** The need for subdrains should be evaluated by the geotechnical engineer once the structure locations, pad grades and foundation plans have been finalized.

Erosion Control

51. The loose top soil at the project site has significant potential for erosion where unvegetated. We recommend the following provisions be incorporated into the project plans.

A. All grading and soil disturbance shall be kept to a minimum.

B. No eroded soil will be allowed to leave the site.

C. All bare soil and cut and fill slopes should be seeded and mulched immediately after grading and planted with barley, rye, grass and crimson clover.

Utility Trenches

52. Utility trenches adjacent to foundations should be located above an imaginary line with a slope gradient of 2:1 (H:V) projected from the bottom outer edge of the element.

53. Utility trenches should be backfilled above the pipe and sand zone with engineered fill placed in accordance with the recommendations outlined in this report within the section entitled “Site Grading”.

Pavements

54. Asphaltic concrete, aggregate base and subbase, and preparation of the subgrade should conform to and be placed in accordance with the Caltrans Standard Specifications latest edition, except that the test methods for compaction should be determined by ASTM D1557-78.

55. To have the selected pavement sections perform to their greatest efficiency, it is very important that the following items be considered see Figure 28:

a. Scarify and moisture condition the top 8 inches of subgrade and compact to a minimum relative compaction of 95 percent, at a moisture content which is about 4 percent above laboratory optimum value.

b. Provide sufficient gradient to prevent ponding of water.

c. Use only quality materials of the type and thickness (minimum) specified. All baserock (R=78 minimum) must meet CALTRANS Standard Specifications for Class 2 Untreated Aggregate Base

(Section 26). All subbase (R=50 minimum) must meet CALTRANS Standard Specifications for Class 2 Untreated Aggregate Subbase, (Section 25).

d. Compact the baserock and subbase uniformly to a minimum relative compaction of 95 percent.

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

f. Maintenance should be undertaken on a routine basis.

Plan Review, Construction Observation and Testing

56. Haro, Kasunich and Associates should be provided an opportunity to review project plans prior to construction to evaluate if our recommendations have been properly interpreted and implemented. It also provides us an opportunity to provide comments and additional recommendations. We should also provide foundation excavation observations and earthwork observations and testing during construction. This allows us to confirm anticipated soil conditions and evaluate conformance with our recommendations and project plans. If we do not review the plans and provide observation and testing services during the earthwork phase of the project, we assume no responsibility for misinterpretation of our recommendations.

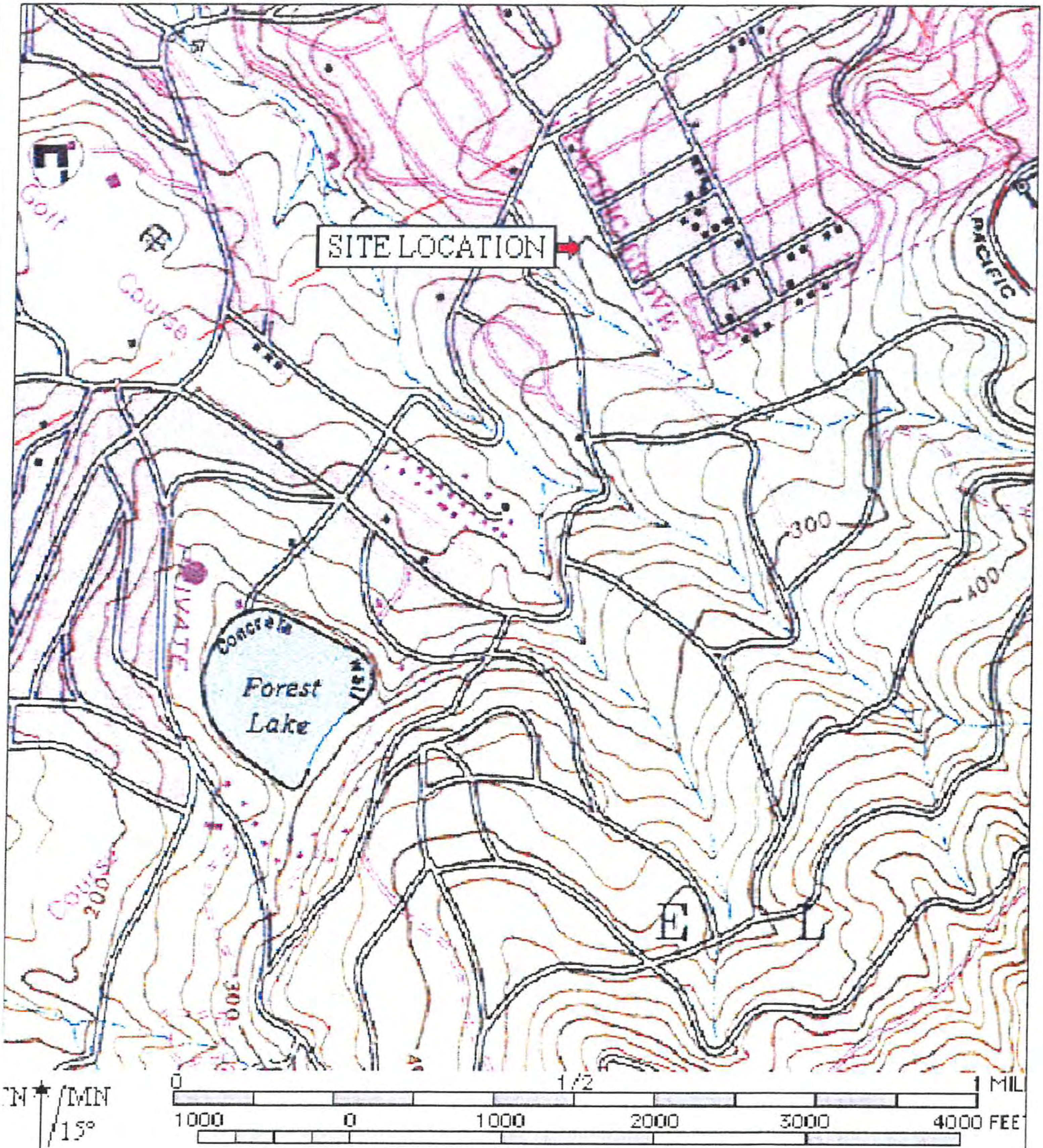
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 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 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. The conclusions and recommendations contained herein are professional opinions derived in accordance with current standards of professional practice. No other warranty expressed or implied is made.

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 be due to natural processes or to 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 our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by a geotechnical engineer.

APPENDIX A



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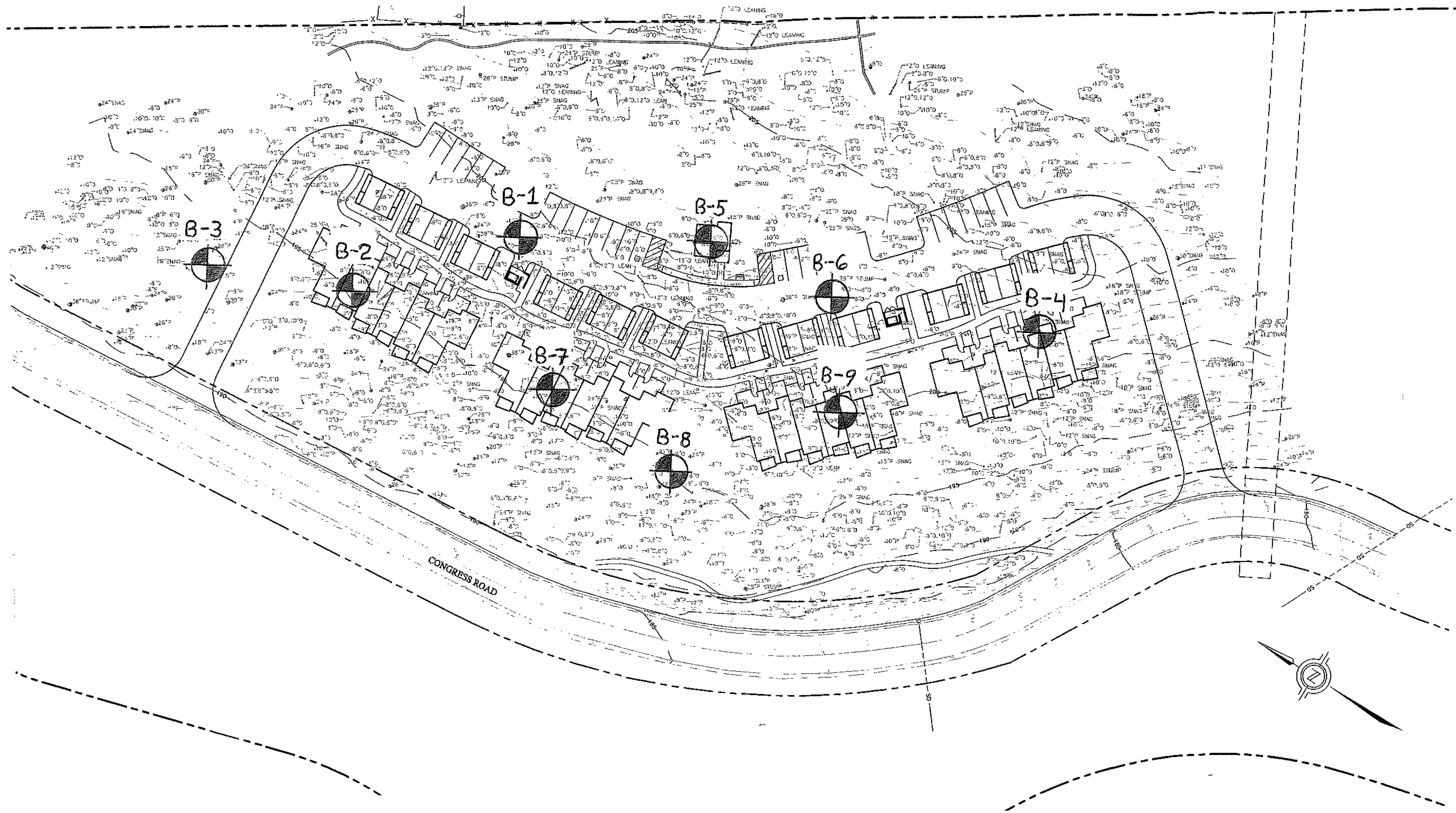
SITE LOCATION MAP
Area D - Del Monte Forest Plan
Congress Road
Pebble Beach, Monterey County, California

SCALE AS SHOWN	APN 008-041-009
DRAWN BY BILL S.	
DATE 4-3-13	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
REVISED	
JOB NO M10473	

FIGURE NO. 34



NOTES:

B-1 EXPLORATORY BORING LOCATION

BASE MAP BY L&S ENGINEERING AND SURVEYING INC. DATED 2-26-13

BORING SITE PLAN
Area D - Del Monte Forest Plan
Congress Road
Pebble Beach, Monterey County, California

SCALE: 1"=70' (approximate)	APN 008-041-009
DRAWN BY: BILL S.	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
DATE: 4-3-13	
REVISED:	
JOB NO. M10473	
FIGURE NO. 2	35

COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH 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 NO. 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 WITH 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 NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, 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	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200 40 10 4 3/4" 3" 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

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

CONSISTENCY

SILTS AND CLAYS	STRENGTH (TSF)**	BLOWS PER FOOT*
VERY SOFT	0 - ¼	0 - 2
SOFT	¼ - ½	2 - 4
FIRM	½ - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

SAMPLING METHOD

STANDARD PENETRATION TEST	T	
MODIFIED CALIFORNIA	L or M	
PITCHER BARREL	P	
SHELBY TUBE	S	
BULK	B	

H_2O

Final	▽
Initial	▼

Water level designation

*Number of blows of 140 lb hammer falling 30 inches to drive a 2" O.D. (1⅜" I.D.) split spoon sampler (ASTM D-1586)

**Unconfined compressive strength in tons/ft² as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO LOGS
Area D - Del Monte Forest Plan
Congress Road
Pebble Beach, Monterey County, California

SCALE: NTS	APN 008-041-009
DRAWN BY: BILL S.	
DATE: 4-3-13	
REVISED:	
JOB NO. M10473	

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 3

Haro, Kasunich & Associates, Inc.
Geotechnical and Coastal Engineers

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC | DATE DRILLED March 26, 2013 | BORING DIAMETER 6" | BORING NO. B-1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0	1-1-1 (L)		Brown Silty SAND, fine to medium grain, damp, loose	SM	17		102	7.4	
5	1-2-1 (L)		Tan Silty SAND, fine to medium grain, saturated, medium dense		27				
			Boring terminated at 9 feet						
10									
15									
20									
25									
30									
35									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\M10473 Area D Subdivision.log Date: 4/29/2013

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 4

Area D Subdivision

PROJECT NO. M10473

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

LOGGED BY MC | DATE DRILLED March 26, 2013 | BORING DIAMETER 6" | BORING NO. B-2

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Brown Silty SAND, fine to medium grain, roots, moist, loose	SM					
				24				
	2-1-1 (L)	Color change to tan and medium dense		20				
	2-2 (T)	Interbedded brown orange tan Silty SAND, fine to medium grain, wet, medium dense						
5	2-3 (T)	Tan orange rust SAND, fine to medium grain, oxidation, wet, medium dense	SW	26			13.1	Atterberg Limits PI = 12 LL = 22.1%
		Black Sandy CLAY, organic odor, fine grain, moist, very stiff	CL					
10	2-4 (T)	Grey Clayey SAND, fine to medium grain, wet, medium dense (weathered sandstone)	SC	21			16.8	
15	2-5 (T)	Grey SAND, fine to medium grain, wet to saturated, dense (weathered granite)	SW	49			18.7	
		Pulled out auger and hole caved in to 6.5 feet						
20		Boring terminated at 20 feet						
25								
30								
35								

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HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 5

Haro, Kasunich & Associates, Inc.

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC | DATE DRILLED March 26, 2013 | BORING DIAMETER 6" | BORING NO. B-3

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Brown Silty SAND, fine to medium grain, roots, moist, loose	SM					
	3-1-1 (L)		Tan and orange SAND, fine to medium grain, moist, medium dense	SW	30		117	8.5	
5	3-2 (T)		Color change to orange grey SAND, fine to medium grain, very moist to wet, dense (weathered sandstone)		30				
			Boring terminated at 8 feet						
10									
15									
20									
25									
30									
35									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\M10473 Area D Subdivision.log Date: 4/29/2013

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 6

Haro, Kasunich & Associates, Inc.

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC **DATE DRILLED** March 26, 2013 **BORING DIAMETER** 6" **BORING NO.** B-4

Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
	Brown Silty SAND, fine to medium grain, damp, roots	SM					
4-1-1 (L)	Grey orange brown Sandy CLAY, fine grain, moist, stiff	CL	20		104	22.5	Unconfined Compression Test Qu = 2,983 psf Atterberg Limits (4-1-1) PI = 29 LL = 45.9%
4-2 (T)	Grey with orange mottling Clayey SAND, fine to coarse grain, moist, dense (weathered sandstone)	SC	34				
4-3 (T)	Grey SAND, fine to medium grain, moist to damp, trace of binder, dense	SW	36				
	Orange brown Clayey SAND, fine to medium grain, moist	SC					
4-4 (T)	Grey orange Clayey SAND, fine to coarse grain saturated, dense		33			17.0	
4-5 (T)	Grey white SAND fine to medium grain, damp, very dense (weathered granite)	SW	50/5"				

Boring terminated at 19.5 feet

Depth, ft.: 0, 5, 10, 15, 20, 25, 30, 35

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 7

Haro, Kasunich & Associates, Inc.

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC | DATE DRILLED March 26, 2013 | BORING DIAMETER 6" | BORING NO. B-5

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft. - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Brown grey Silty SAND, fine to medium grain, damp, roots, loose	SM					
	5-1-1 (L)		Tan SAND fine to medium grain, moist to damp, medium dense	SW	39		117	9.9	
5	5-2 (T)		Grey orange tan mottled Clayey SAND, fine to medium grain, moist, dense (weathered sandstone)	SC	45				
10	5-3- 1 (L)		Grey orange mottled SAND fine to medium grain, wet, medium dense	SW	53			15.5	
			Orange brown SAND fine to medium grain, saturated (weathered sandstone)						
15			Grey SAND fine to medium grain, saturated, (weathered granite)						
			Failed sample attempt hole caved in						
20			Very dense at 21 feet (weathered granite)	SW					
			Boring terminated at 22 feet						
25									
30									
35									

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HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 8

Haro, Kasunich & Associates, Inc.

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC DATE DRILLED March 26, 2013 BORING DIAMETER 6" BORING NO. B-6

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Dark brown Silty SAND, fine to medium grain, roots, organics, very loose	SM					
	6-1-1 (L)		Grey brown Silty SAND, fine to medium grain, damp		10				
5	6-2 (T)		Mottled tan and orange Clayey SAND, fine to medium grain, moist, dense (weathered sandstone)	SC	42				
			Boring terminated at 8.5 feet						
10									
15									
20									
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 9

Haro, Kasunich & Associates, Inc.

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC DATE DRILLED March 26, 2013 BORING DIAMETER 6" BORING NO. B-7

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Brown Silty SAND, fine to medium grain, damp, roots, loose	SM					
	7-1-1 (L)		Tan SAND, fine to medium, grain, moist, very loose	SW	10		92.1	16.4	Saturated Direct Shear $\phi = 38^{\circ}$ C = 4.0 psf
	7-2 (T)		Mottled tan orange Clayey SAND, fine to medium grain, moist, medium dense (weathered sandstone)	SC	25				
5									
10	7-3 (T)		Grey Clayey SAND, fine to coarse grain, moist, medium dense (weathered granite)	SW	24			15.5	
			Grey SAND, fine to coarse grain, wet to saturated, medium dense (weathered granite)						
15	7-4 (T)		Same as above but dense		30				
20									
			Very dense (weathered granite)	SW					
25			Boring terminated at 25 feet						
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr FIGURE NO. 10

Haro, Kasunich & Associates, Inc.

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC | DATE DRILLED March 26, 2013 | BORING DIAMETER 6" | BORING NO. B-8

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Brown Silty SAND, roots, damp	SM					
			Tan SAND, fine to medium grain	SW					
5			Orange grey Clayey SAND	SC					
	8-1 (T)				22				
10			Boring terminated at 9.5 feet						
15									
20									
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 11

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Area D Subdivision

PROJECT NO. M10473

LOGGED BY MC | DATE DRILLED March 26, 2013 | BORING DIAMETER 6" | BORING NO. B-9

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Brown tan SAND fine to medium, grain, damp, loose	SW					
	9-1 (T)		Grey orange mottled Clayey SAND, fine to coarse grain, moist, roots, medium dense (weathered sandstone)	SC	15			17.5	Atterberg Limits PI = 19 LL = 30.3%
5	9-2 (T)		Grey Clayey SAND, fine to coarse grain, moist, dense (weathered granite)		51				
			Boring terminated at 6.5 feet						
10									
15									
20									
25									
30									
35									

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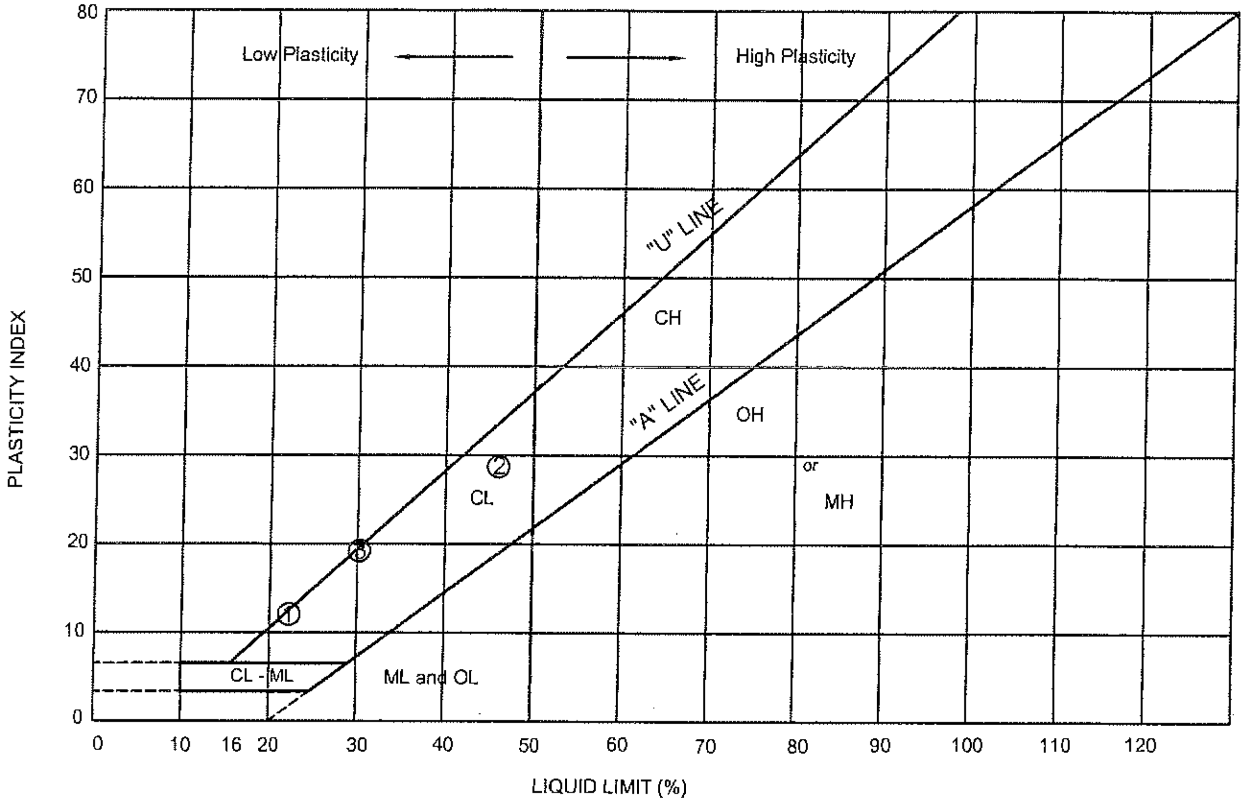
HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 12

PLASTICITY CHART



PLASTICITY DATA

Key Symbol	Sample Number	Depth (feet)	Natural Water Content W(%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index	Liquidity Index W - PL / LL -PL	Unified Soil Classification Symbol
①	2-3	5.0	13.1	10.7	22.1	12	+0.200	CL
②	4-1-1	1.0	22.5	17.1	45.9	29	+0.1862	CL
③	9-1	1.0	17.5	11.9	30.3	19	+0.2947	*

ATTERBERG LIMITS TEST RESULTS
Pebble Beach Company Area D
Pebble Beach, California

SCALE	No Scale
DRAWN BY	MC
DATE	April 2013
REVISED	
JOB NO.	M10473

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-1475

FIGURE NO. 13

Direct Shear

Project:	Pebble Beach Subdivision D		Date	4/5/2013
Sample #	7-1-1		Tested By:	MA
Description	Tan Sand			

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	13.4	28.1	53.1	
Shear Stress (PSF)	394.6	826.6	1562.4	

Equation of Trendline	
Intercept	Slope
3.5271	0.7724

C (PSF)	PHI
4	38

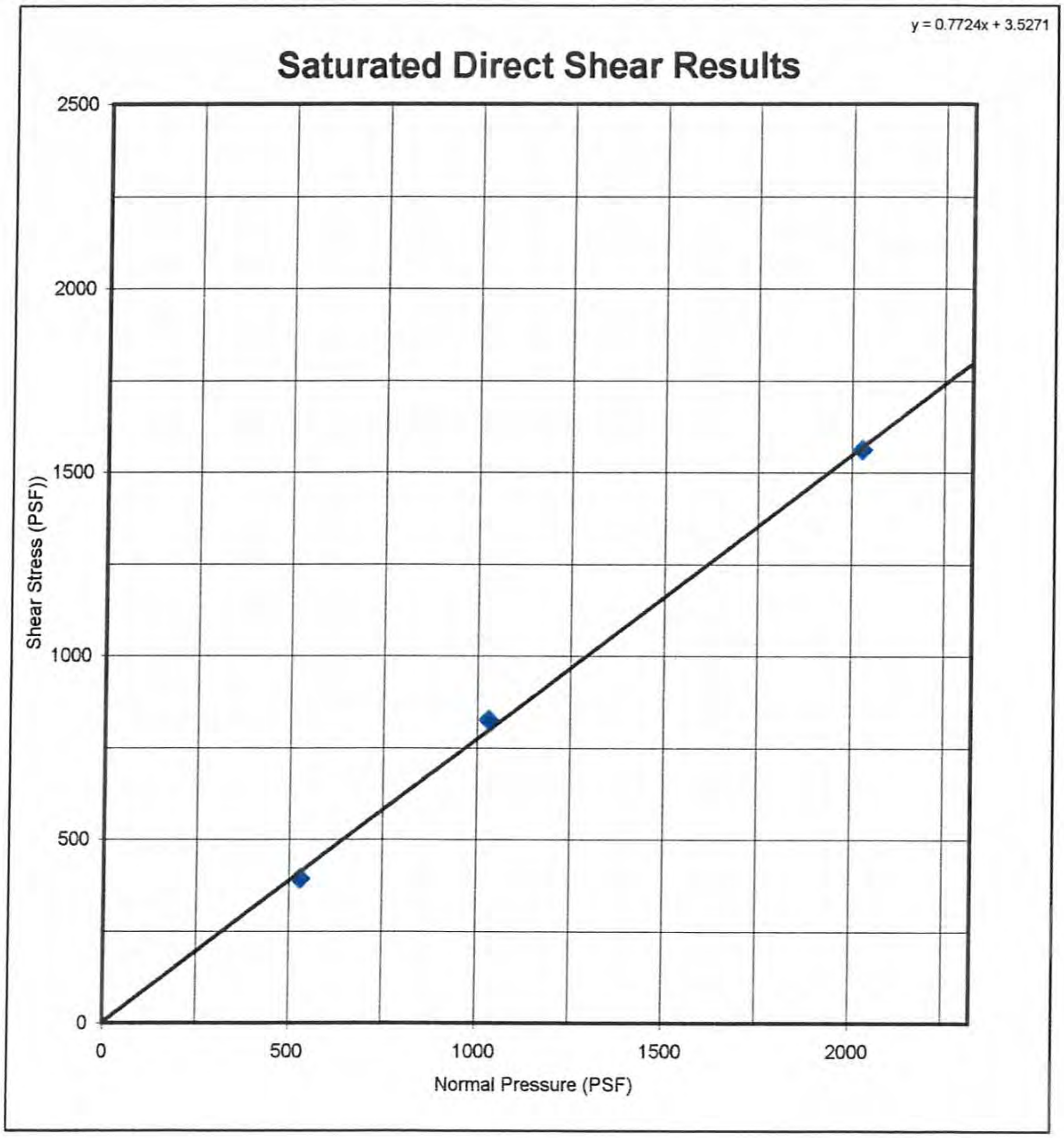


Figure No. 14

R-value Test Report (Caltrans 301)

Job No.:	032-409	Date:	04/04/13	Initial Moisture,	10.4%
Client:	Haro, Kasunich and Associates, Inc.	Tested	MD	R-value by Stabilometer	66
Project:	Pebbel Beach Subdivision D - M10473	Reduced	RU		
Sample	B-3	Checked	DC	Expansion Pressure	0 psf
Soil Type:	Dark Brown Silty SAND/SAND w/ Silt				

Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	106	165	585		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	71	49	37		
Weight of Soil & Mold, grams	3085	3125	3063		
Weight of Mold, grams	2099	2166	2102		
Height After Compaction, in.	2.53	2.53	2.51		
Moisture Content, %	16.9	14.9	13.8		
Dry Density, pcf	100.9	99.9	101.9		
Expansion Pressure, psf	0.0	0.0	21.5		
Stabilometer @ 1000					
Stabilometer @ 2000	48	38	36		
Turns Displacement	4.56	4.42	4.29		
R-value	56	64	67		

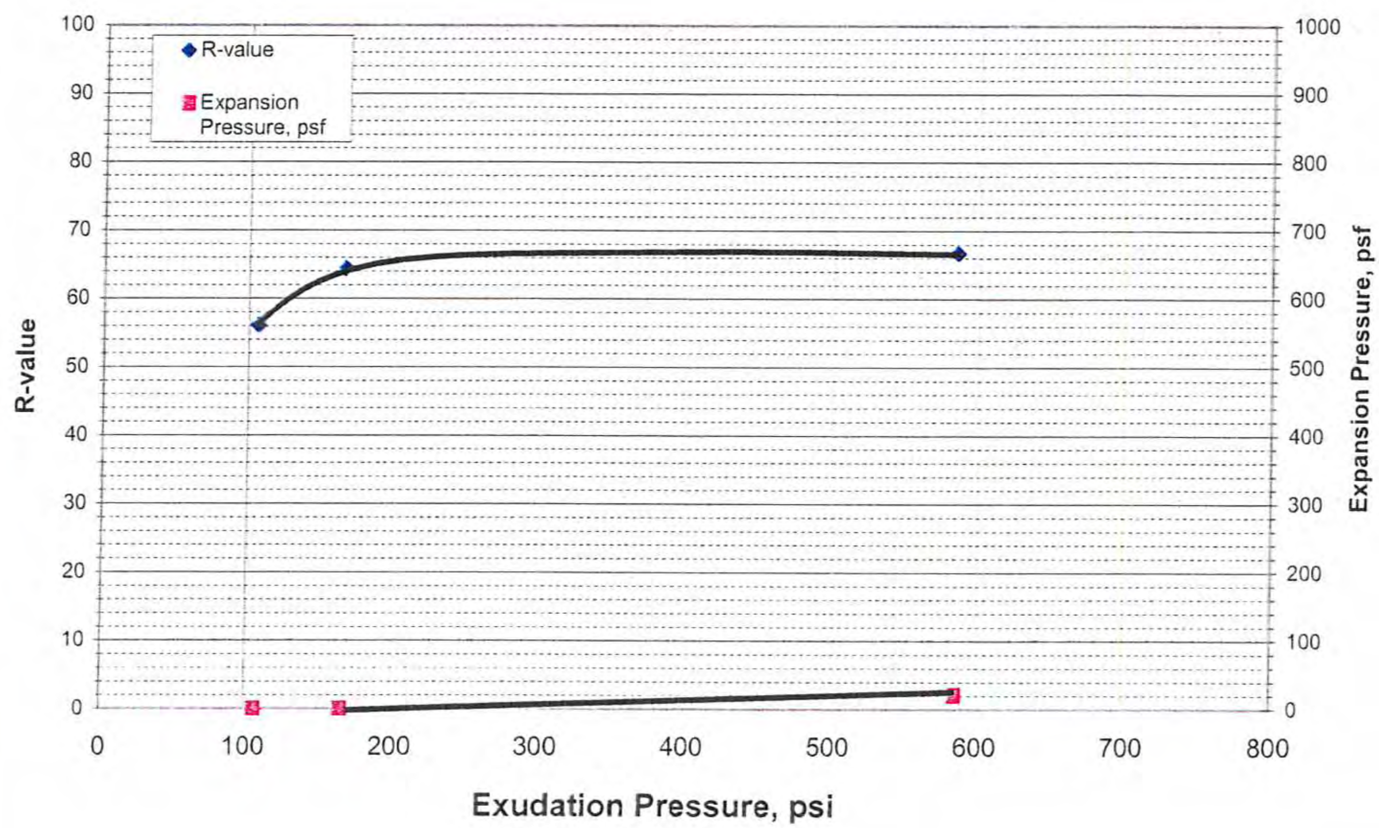


Figure No. 15

R-value Test Report (Caltrans 301)

COOPER TESTING LABORATORY

Job No.:	032-409	Date:	04/04/13	Initial Moisture,	8.2%
Client:	Haro, Kasunich and Associates, Inc.	Tested	MD	R-value by Stabilometer	69
Project:	Pebble Beach Subdivion D - M10473	Reduced	RU		
Sample	B-4	Checked	DC	Expansion Pressure	0 psf
Soil Type:	Grayish Brown Silty SAND				

Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	133	448	206		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	58	33	44		
Weight of Soil & Mold, grams	3103	3122	3069		
Weight of Mold, grams	2191	2106	2094		
Height After Compaction, in.	2.45	2.7	2.68		
Moisture Content, %	13.4	11.1	12.1		
Dry Density, pcf	99.4	102.5	98.2		
Expansion Pressure, psf	0.0	0.0	0.0		
Stabilometer @ 1000					
Stabilometer @ 2000	46	40	40		
Turns Displacement	4.52	4.02	4.3		
R-value	58	69	67		

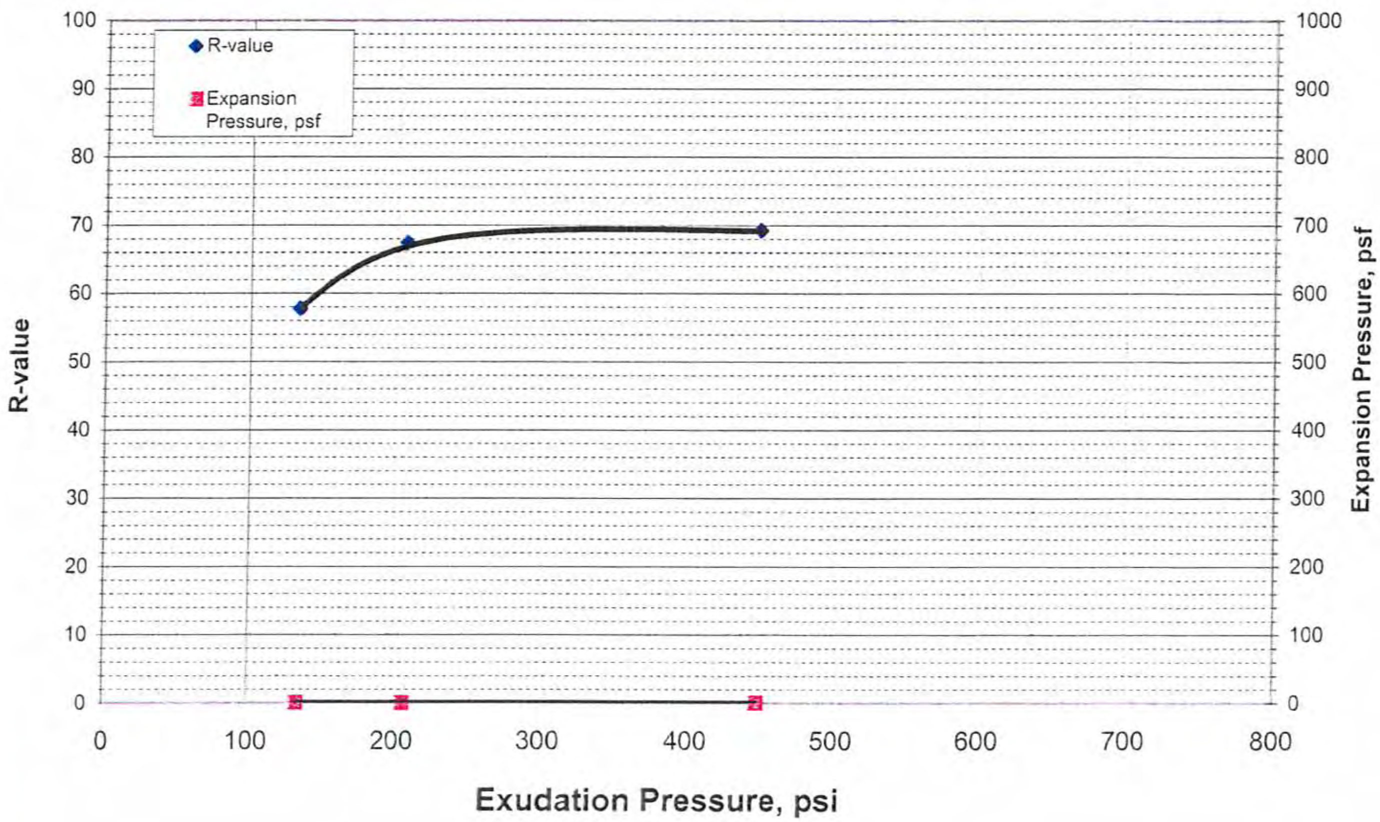


Figure No. 16

Percolation Test Results For Area D

Project No: M10473
Date: 4/3/13
By: Haro, Kasunich and Associates

HOLE NO.:	1		TEST DATE: 3/28/13 DRILL DATE: 3/26/13			
WATER LEVEL AFTER PRE-SOAK: No Water			DEPTH OF BORING: 2.5 feet			
TESTED BY:	MC		PERCOLATION ZONE: 1.5 to 2.5 feet			
READING	ELAPSED TIME (min)	DEPTH TO WATER (feet)	INCREMENTAL FALL OF WATER (inches)		PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	1.45	0.0		0.0	0.0
1	10	2.50	No Water	Refill To 1.5'	0.79	75.9
2	20	2.50	No Water	Refill To 1.6'	0.93	64.5
3	30	2.50	No Water	Refill To 1.5'	0.83	72.3
4	40	2.50	No Water	Refill To 1.5'	0.83	72.3
5	50	2.50	No Water	Refill To 1.5'	0.83	72.3
6	60	2.50	No Water	Refill To 1.5'	0.83	72.3
7	70	2.50	No Water	Refill To 1.5'	0.83	72.3
8	80	2.50	No Water	Refill To 1.5'	0.83	72.3
					Average Perc (in/hr)	71.8
					Design Perc. (in/hr)	36.0

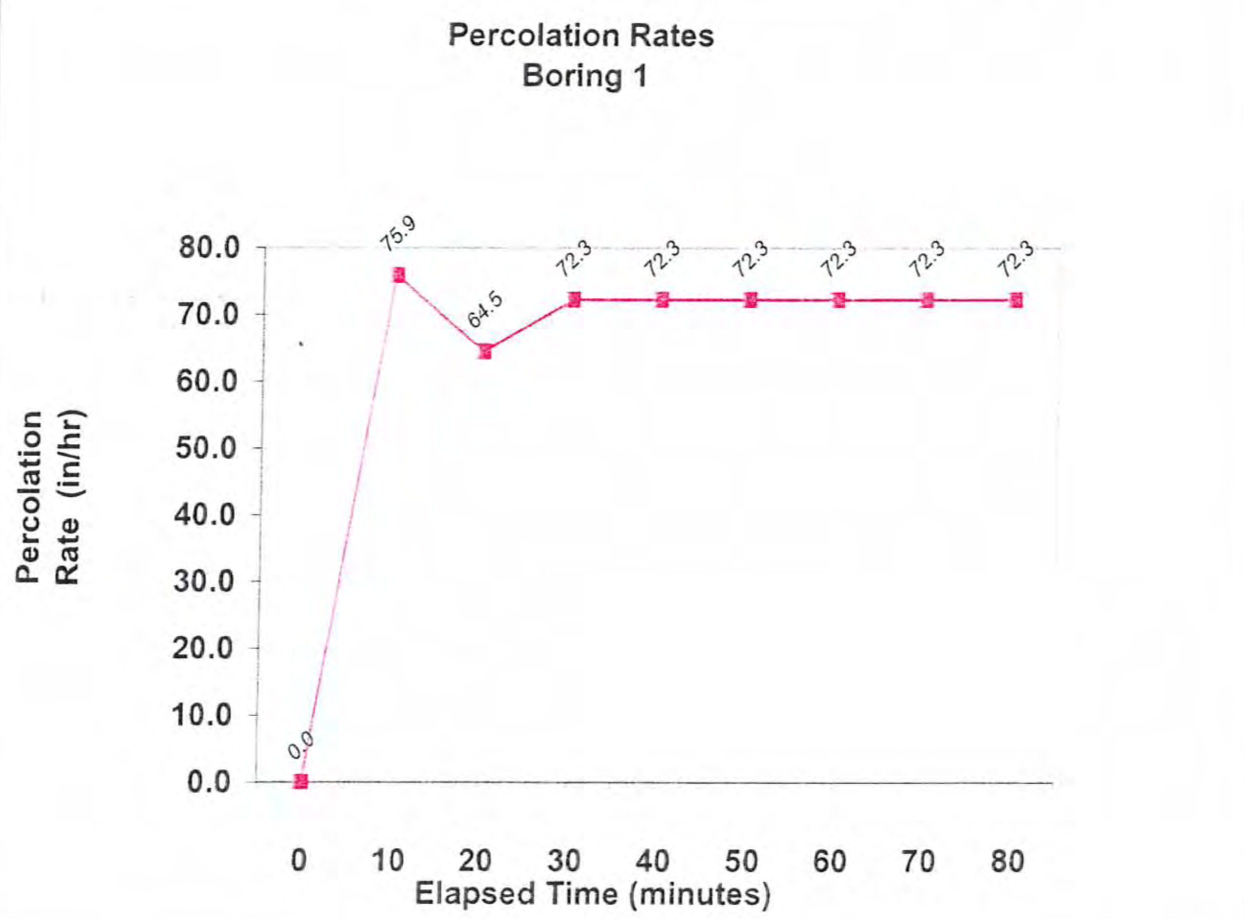


Figure No. 17

Percolation Test Results For Area D
Project No: M10473
Date: 4/3/13
By: Haro, Kasunich and Associates

HOLE NO.:	3		TEST DATE: 3/28/13 DRILL DATE: 3/26/13			
WATER LEVEL AFTER PRE-SOAK: No Water			DEPTH OF BORING: 3.7 feet			
TESTED BY:	MC		PERCOLATION ZONE: 2.4 to 3.7 feet			
READING	ELAPSED TIME (min)	DEPTH TO WATER (feet)	INCREMENTAL FALL OF WATER (inches)		PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	2.40	0.0		0.0	0.0
1	30	3.54	No Water	Refill To 2.7'	2.98	20.2
2	60	3.56	No Water	Refill To 2.7'	2.91	20.6
3	90	3.57	No Water	Refill To 2.5'	2.34	25.7
4	120	3.60	No Water	Refill To 2.5'	2.27	26.4
5	150	3.60	No Water	Refill To 2.5'	2.27	26.4
6	180	3.60	No Water	Refill To 2.5'	2.27	26.4
7	210	3.60	No Water	Refill To 2.5'	2.27	26.4
8	240	3.60	No Water	Refill To 2.5'	2.27	26.4
					Average Perc (in/hr)	24.8
					Design Perc. (in/hr)	24.0

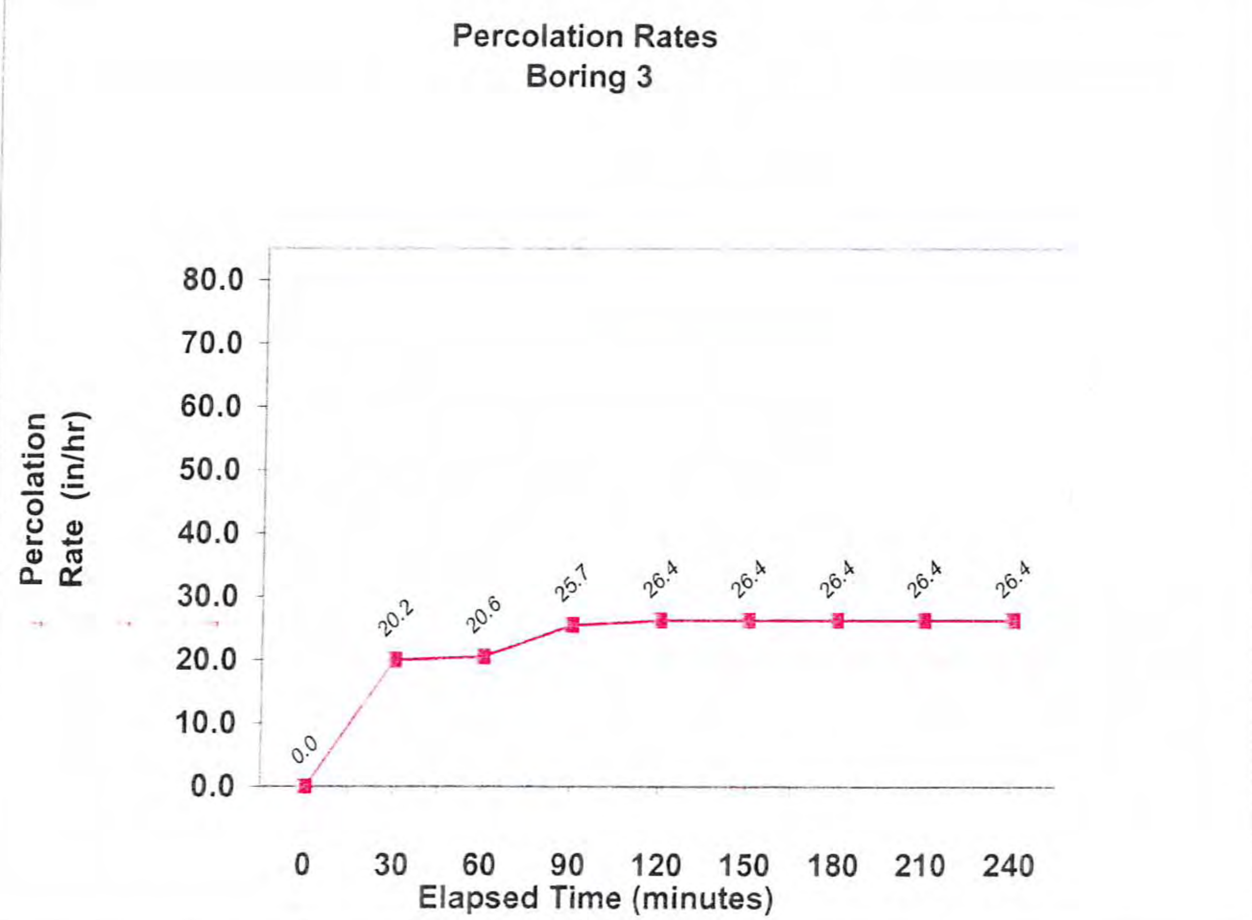


Figure No.18

Percolation Test Results For Area D

Project No: M10473
Date: 4/4/13
By: Haro, Kasunich and Associates

HOLE NO.:	3'		TEST DATE: 3/28/13 DRILL DATE: 3/26/13			
WATER LEVEL AFTER PRE-SOAK: 5.0'			DEPTH OF BORING: 7.8 feet			
TESTED BY:	MC		PERCOLATION ZONE: 5.0 to 7.8 feet			
READING	ELAPSED TIME (min)	DEPTH TO WATER (feet)	INCREMENTAL FALL OF WATER (inches)		PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	5.00	0.0		0.0	0.0
1	30	5.00	0.0		0.00	0.0
2	60	5.00	0.0		0.00	0.0
3	90	5.00	0.0		0.00	0.0
4	120	5.00	0.0		0.00	0.0
5	150	5.00	0.0		0.00	0.0
6	180	5.00	0.0		0.00	0.0
7	210	5.00	0.0		0.00	0.0
8	240	5.00	0.0		0.00	0.0
					Average Perc (in/hr)	0.0
					Design Perc. (in/hr)	0.0

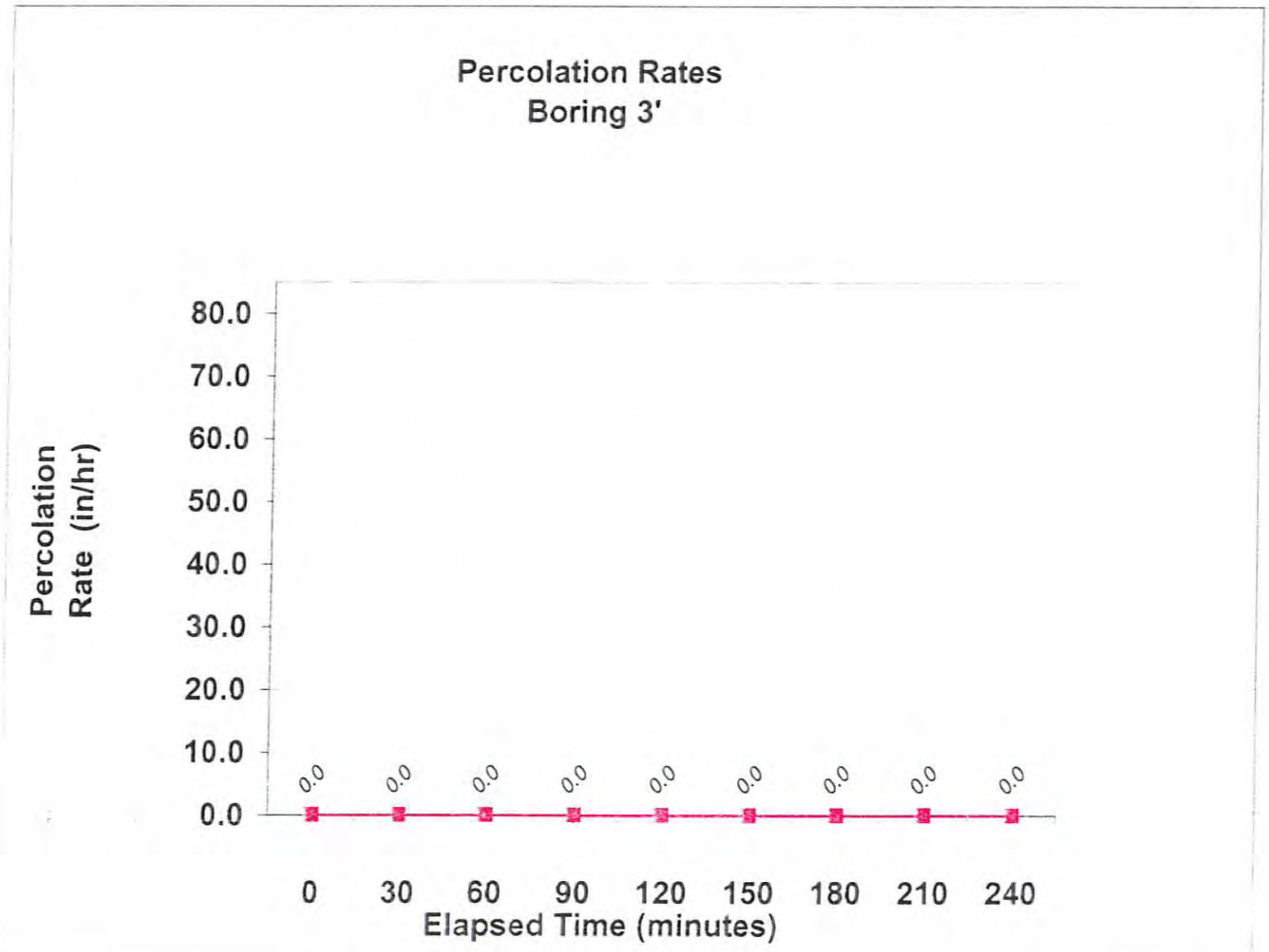
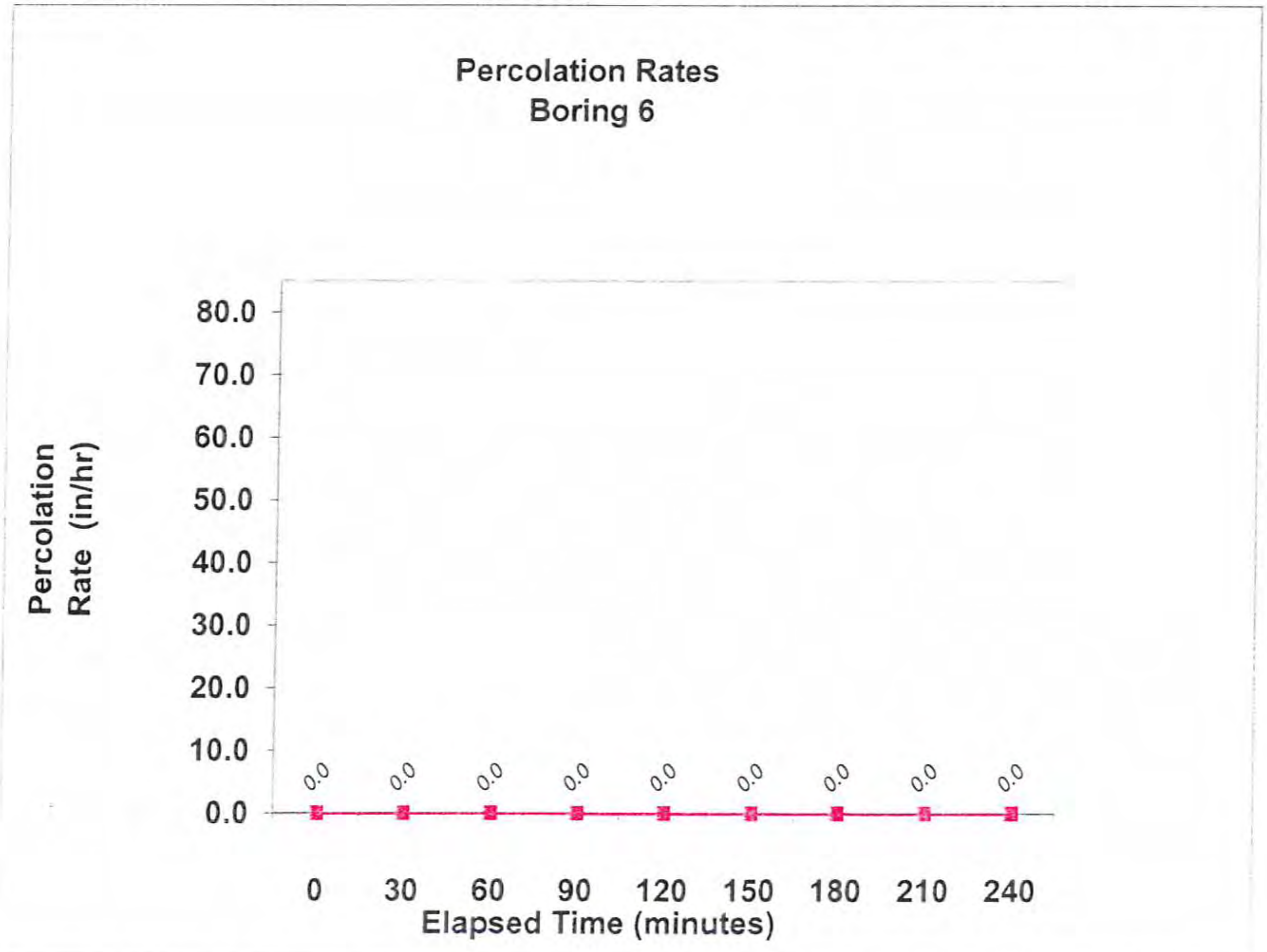


Figure No. 19

Percolation Test Results For Area D

Project No: M10473
Date: 4/4/13
By: Haro, Kasunich and Associates

HOLE NO.:	6		TEST DATE: 3/28/13 DRILL DATE: 3/26/13			
WATER LEVEL AFTER PRE-SOAK: 4.25			DEPTH OF BORING: 5.0 feet			
TESTED BY:	MC		PERCOLATION ZONE: 4.0 to 5.0 feet			
READING	ELAPSED TIME (min)	DEPTH TO WATER (feet)	INCREMENTAL FALL OF WATER (inches)		PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	4.00	0.0		0.00	0.0
1	30	4.00	0.0		0.00	0.0
2	60	4.00	0.0		0.00	0.0
3	90	4.00	0.0		0.00	0.0
4	120	4.00	0.0		0.00	0.0
5	150	4.00	0.0		0.00	0.0
6	180	4.00	0.0		0.00	0.0
7	210	4.00	0.0		0.00	0.0
8	240	4.00	0.0		0.00	0.0
					Average Perc (in/hr)	0.0
					Design Perc. (in/hr)	0.0



Pg. 53

Percolation Test Results For Area D
Project No: M10473
Date: 4/4/13
By: Haro, Kasunich and Associates

HOLE NO.:	6'		TEST DATE: 3/28/13 DRILL DATE: 3/26/13			
WATER LEVEL AFTER PRE-SOAK: 6.7'			DEPTH OF BORING: 8.7 feet			
TESTED BY:	MC		PERCOLATION ZONE: 6.6 to 8.7 feet			
READING	ELAPSED TIME (min)	DEPTH TO WATER (feet)	INCREMENTAL FALL OF WATER (inches)		PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	6.60	0.0		0.0	0.0
1	30	6.70	1.2		25.00	2.4
2	60	6.70	0.0		0.00	0.0
3	90	6.70	0.0		0.00	0.0
4	120	6.70	0.0		0.00	0.0
5	150	6.70	0.0		0.00	0.0
6	180	6.70	0.0		0.00	0.0
7	210	6.70	0.0		0.00	0.0
8	240	6.70	0.0		0.00	0.0
					Average Perc (in/hr)	0.3
					Design Perc. (in/hr)	0.0

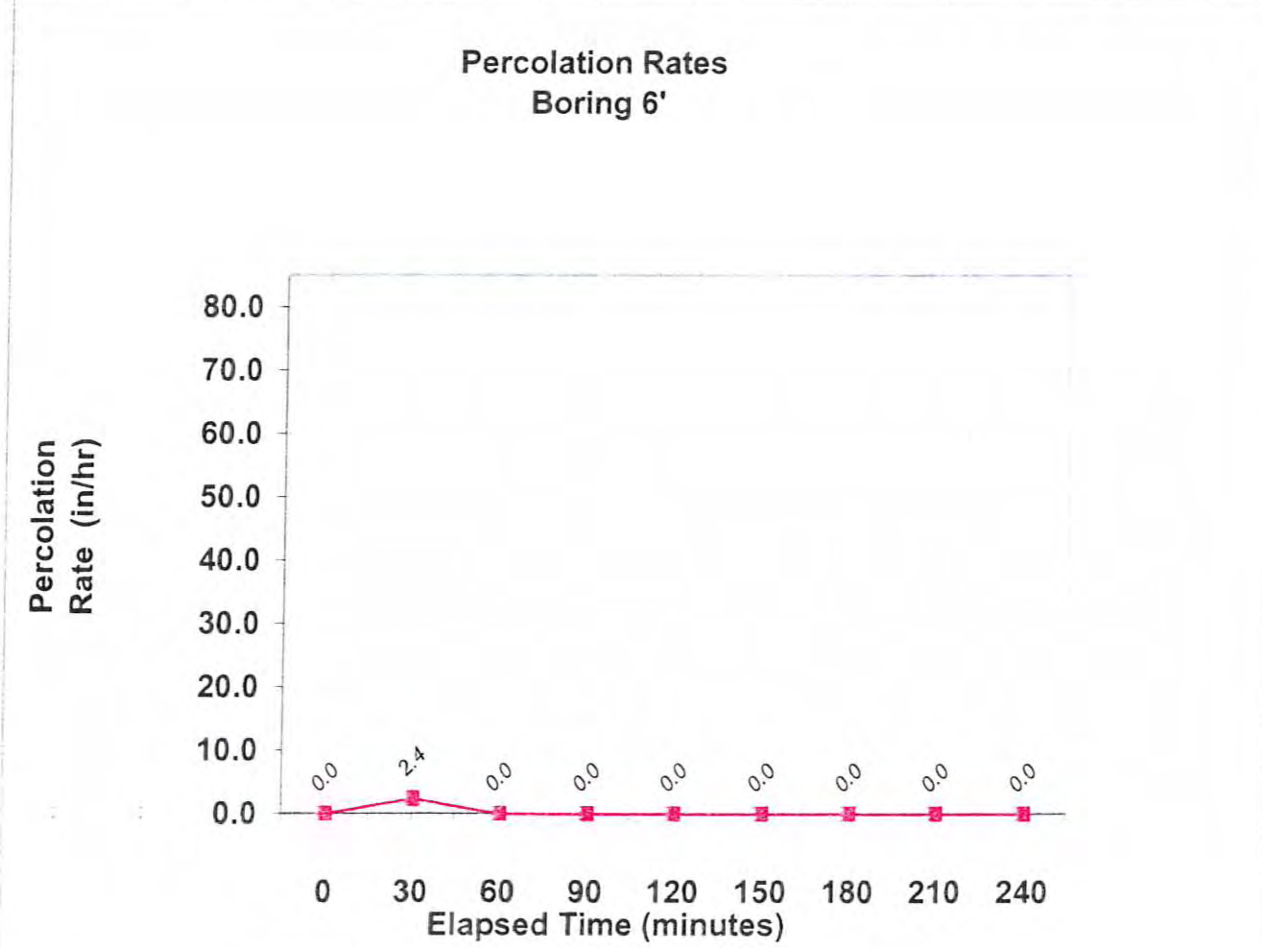


Figure No. 21

Percolation Test Results For Area D

Project No: M10473
Date: 4/4/13
By: Haro, Kasunich and Associates

HOLE NO.:	8'		TEST DATE: 3/28/13 DRILL DATE: 3/26/13			
WATER LEVEL AFTER PRE-SOAK: 5.0'			DEPTH OF BORING: 7.0 feet			
TESTED BY:	MC		PERCOLATION ZONE: 4.8 to 7.0 feet			
READING	ELAPSED TIME (min)	DEPTH TO WATER (feet)	INCREMENTAL FALL OF WATER (inches)		PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	4.80	0.0		0.0	0.0
1	30	4.80	0.0		0.00	0.0
2	60	4.80	0.0		0.00	0.0
3	90	4.80	0.0		0.00	0.0
4	120	4.80	0.0		0.00	0.0
5	150	4.80	0.0		0.00	0.0
6	180	4.80	0.0		0.00	0.0
7	210	4.80	0.0		0.00	0.0
8	240	4.80	0.0		0.00	0.0
					Average Perc (in/hr)	0.0
					Design Perc. (in/hr)	0.0

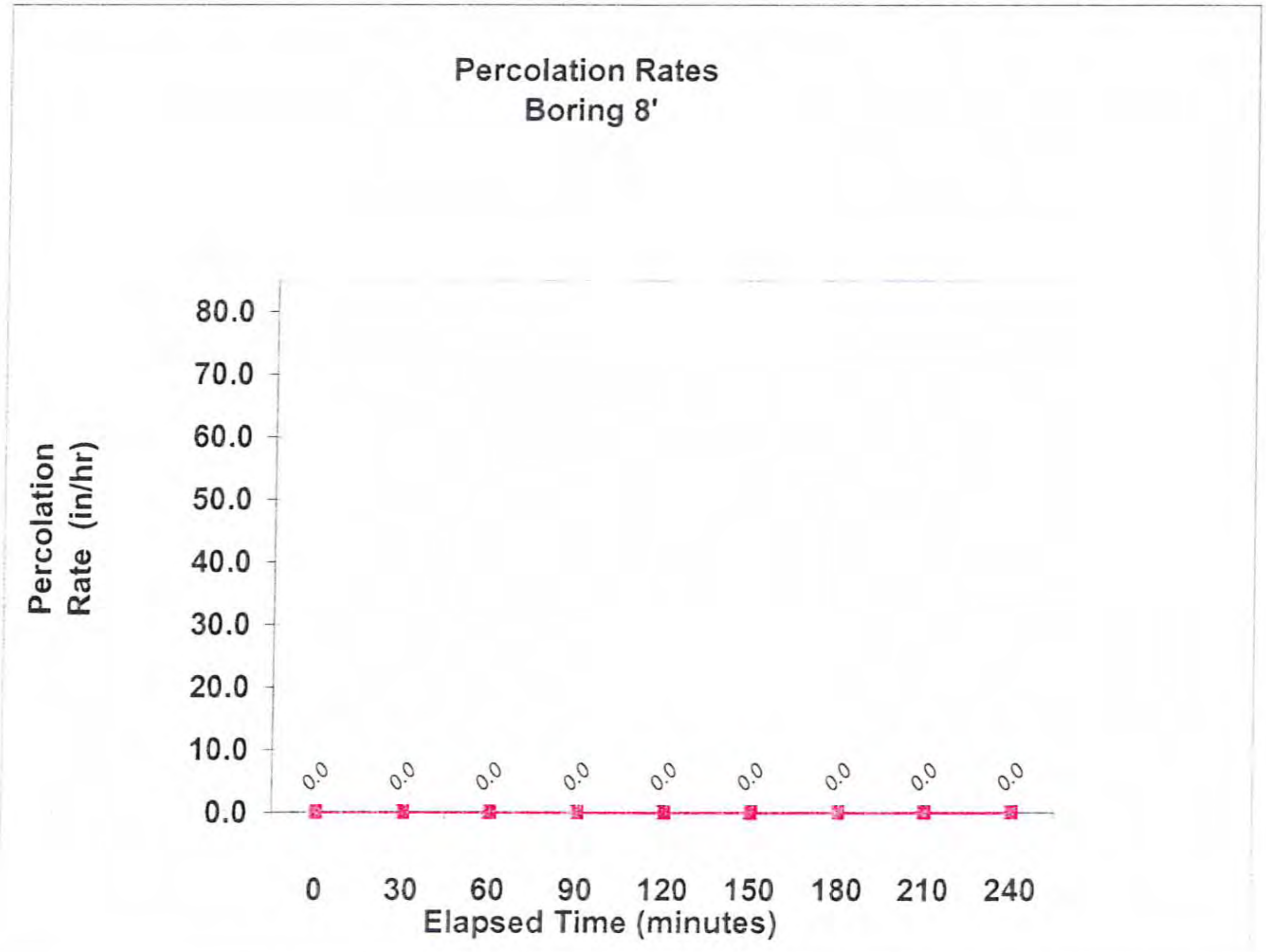
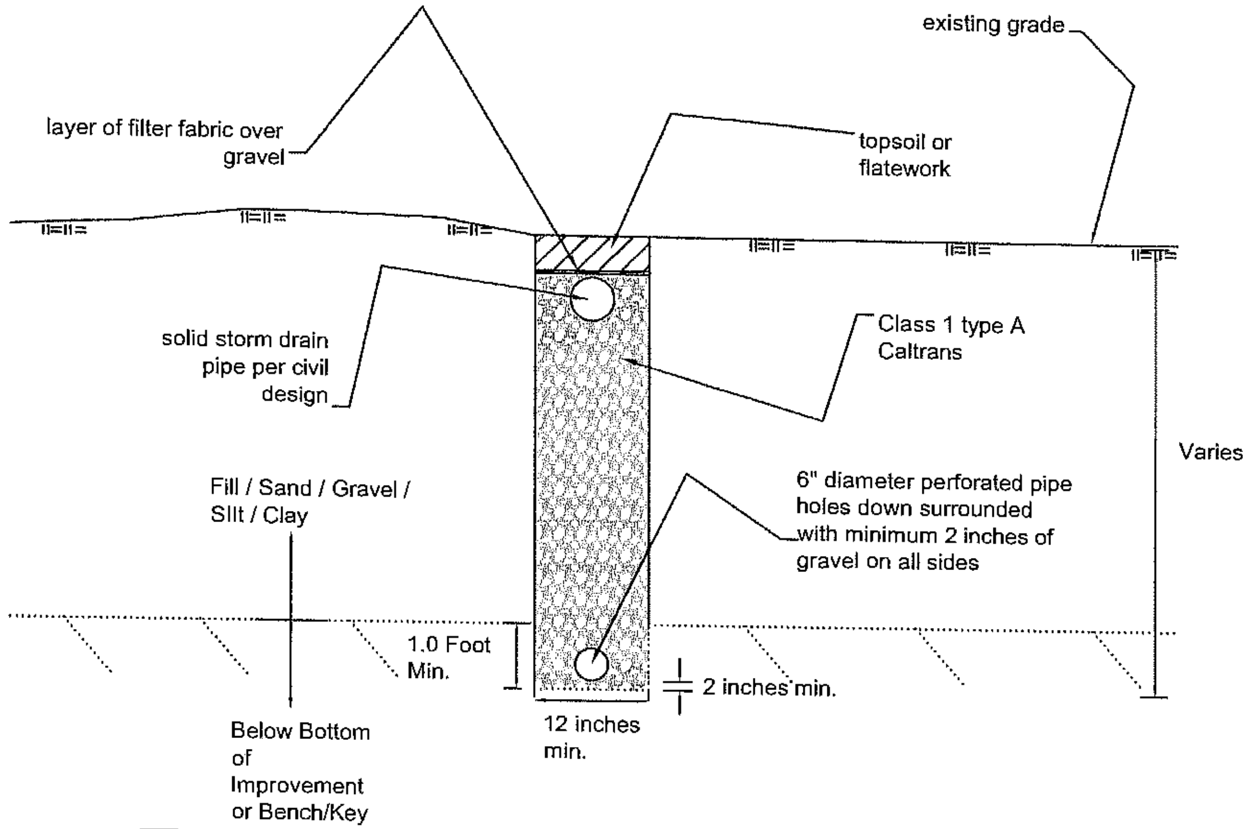


Figure No. 22

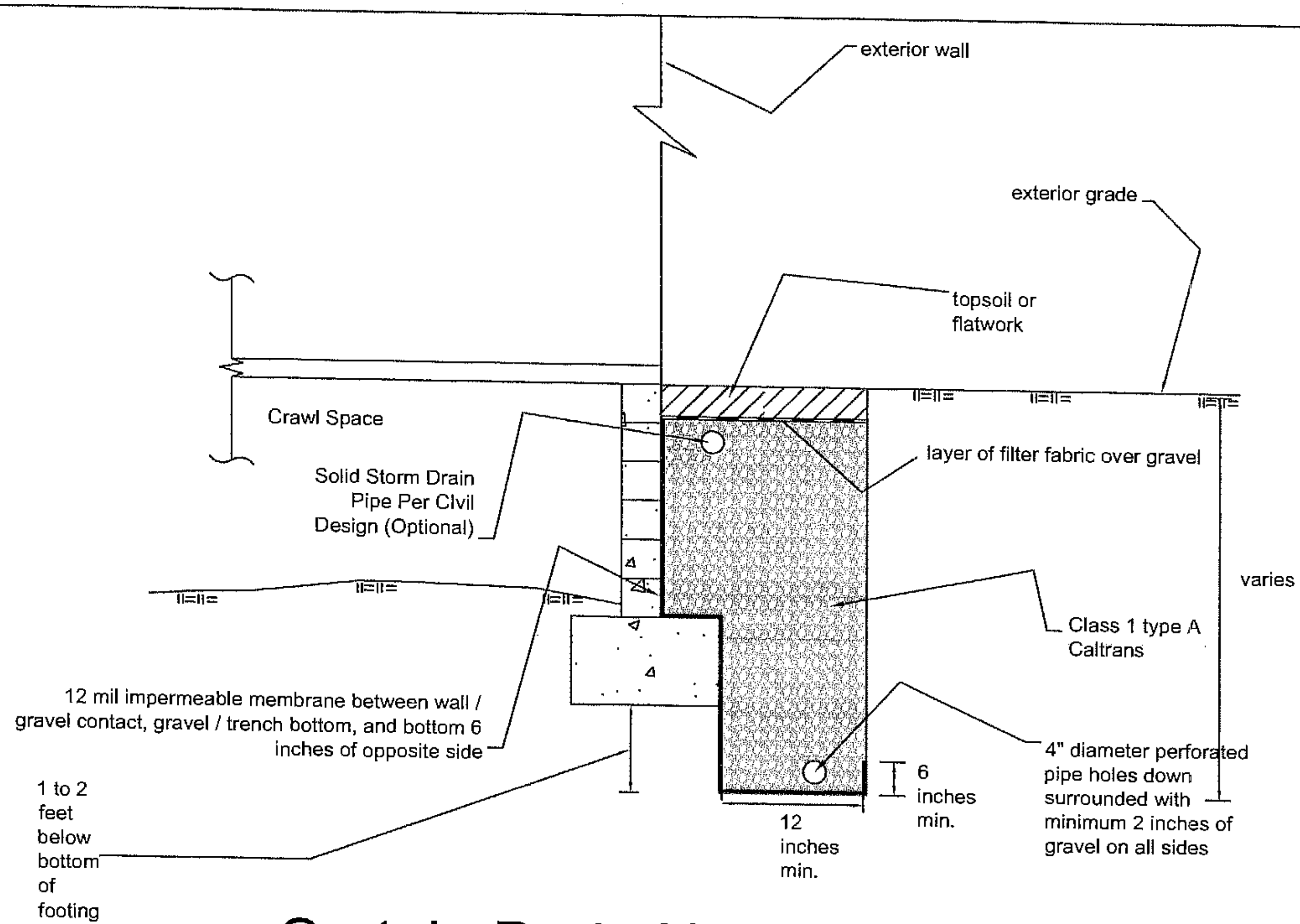


Typical Curtain Drain

N.T.S.

For Illustration Purposes Only Not For Use in Construction .

Typical Site Curtain Drain Pebble Beach Area D Pebble Beach, California	
No Scale	
MC	
April 2013	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
M10473	
FIGURE NO. 23	SHEET NO. 56

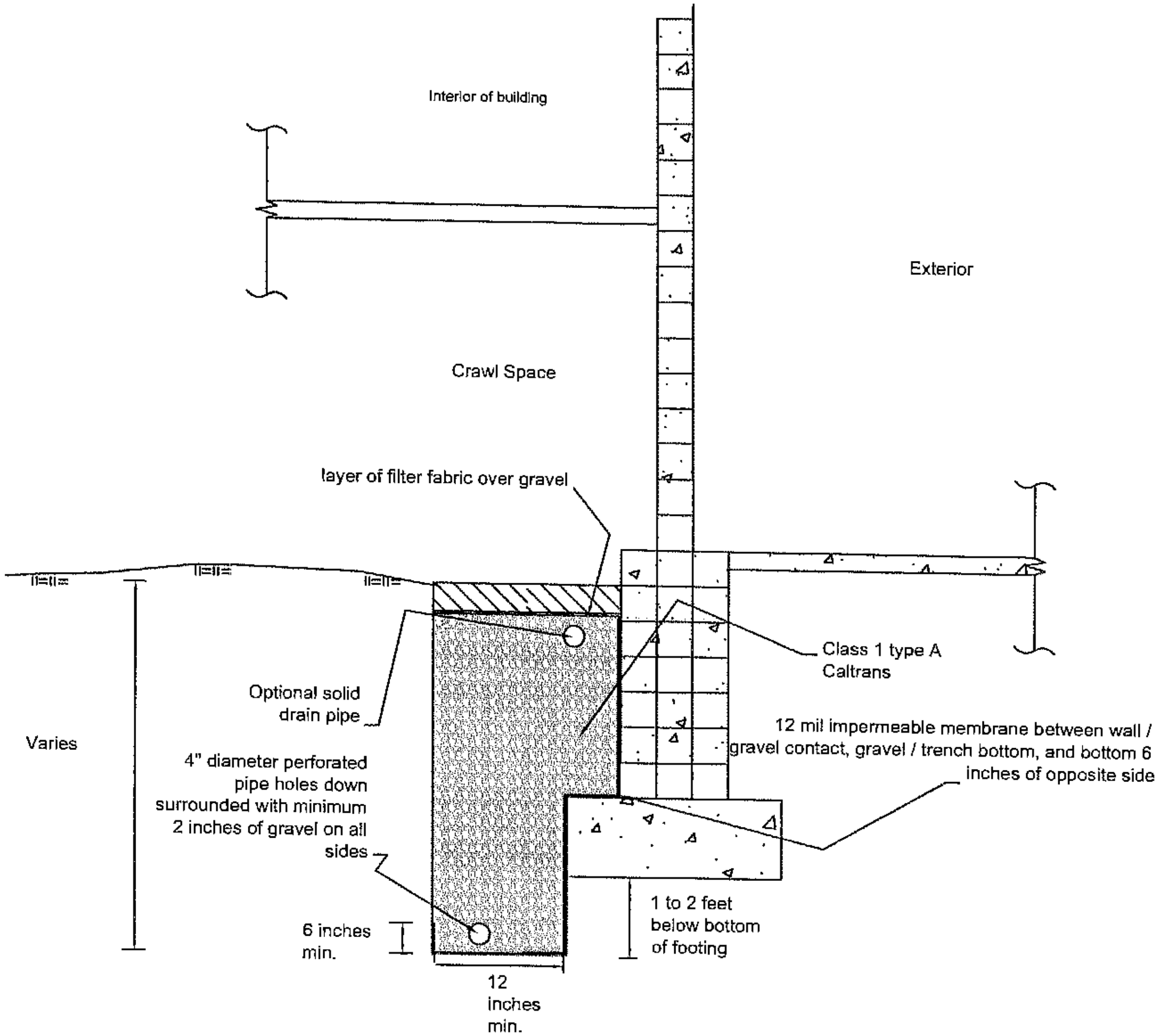


Curtain Drain Next To Foundation

N.T.S.

For Illustration Purposes Only Not For Use in Construction .

Exterior Curtain Drain Next To Foundation Pebble Beach Area D Pebble Beach, California	
No Scale	
MC	
April 2013	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
M10473	
FIGURE NO. 24.	SHEET NO. 57

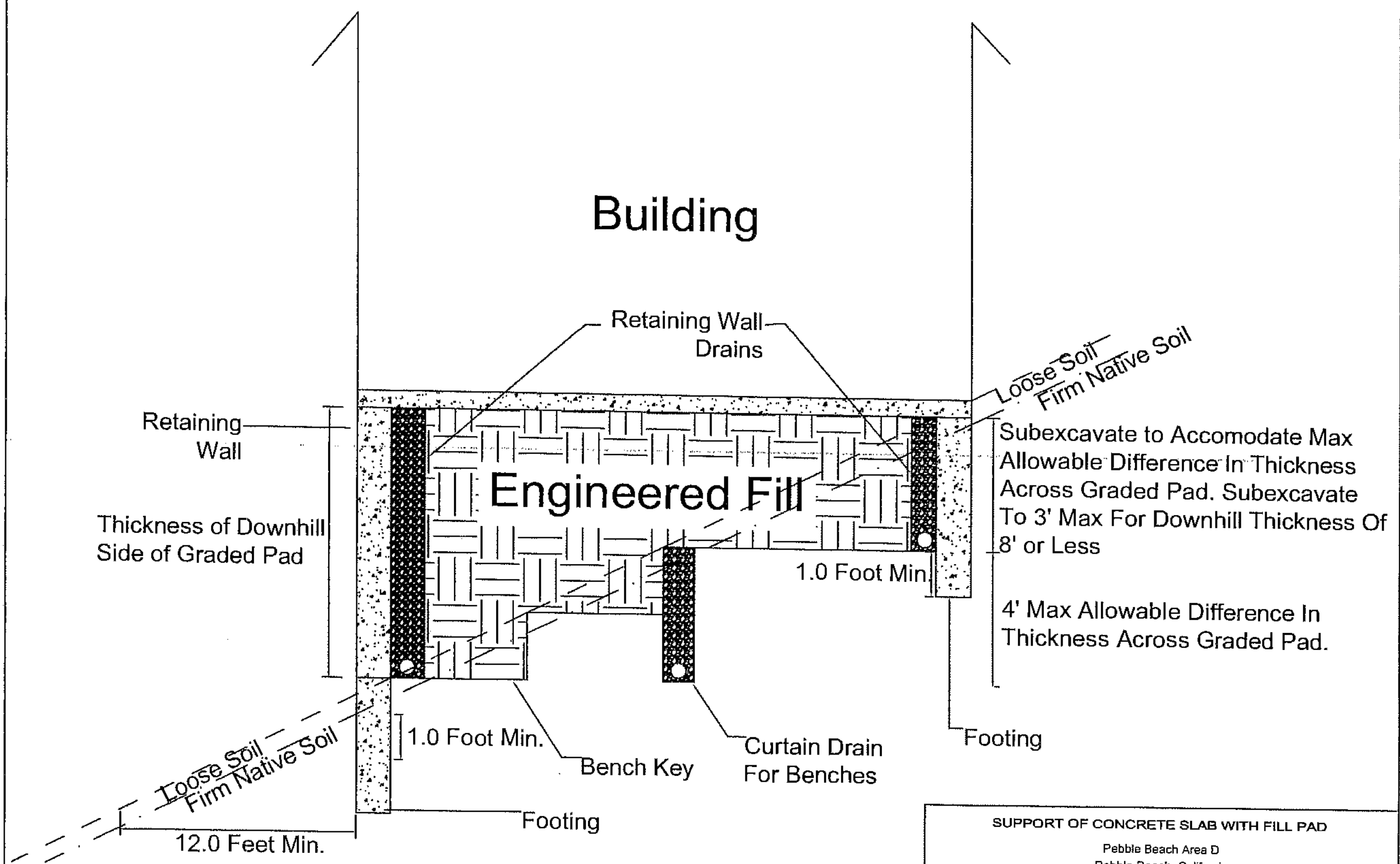


Typical Crawlspace Trench Drain

N.T.S.

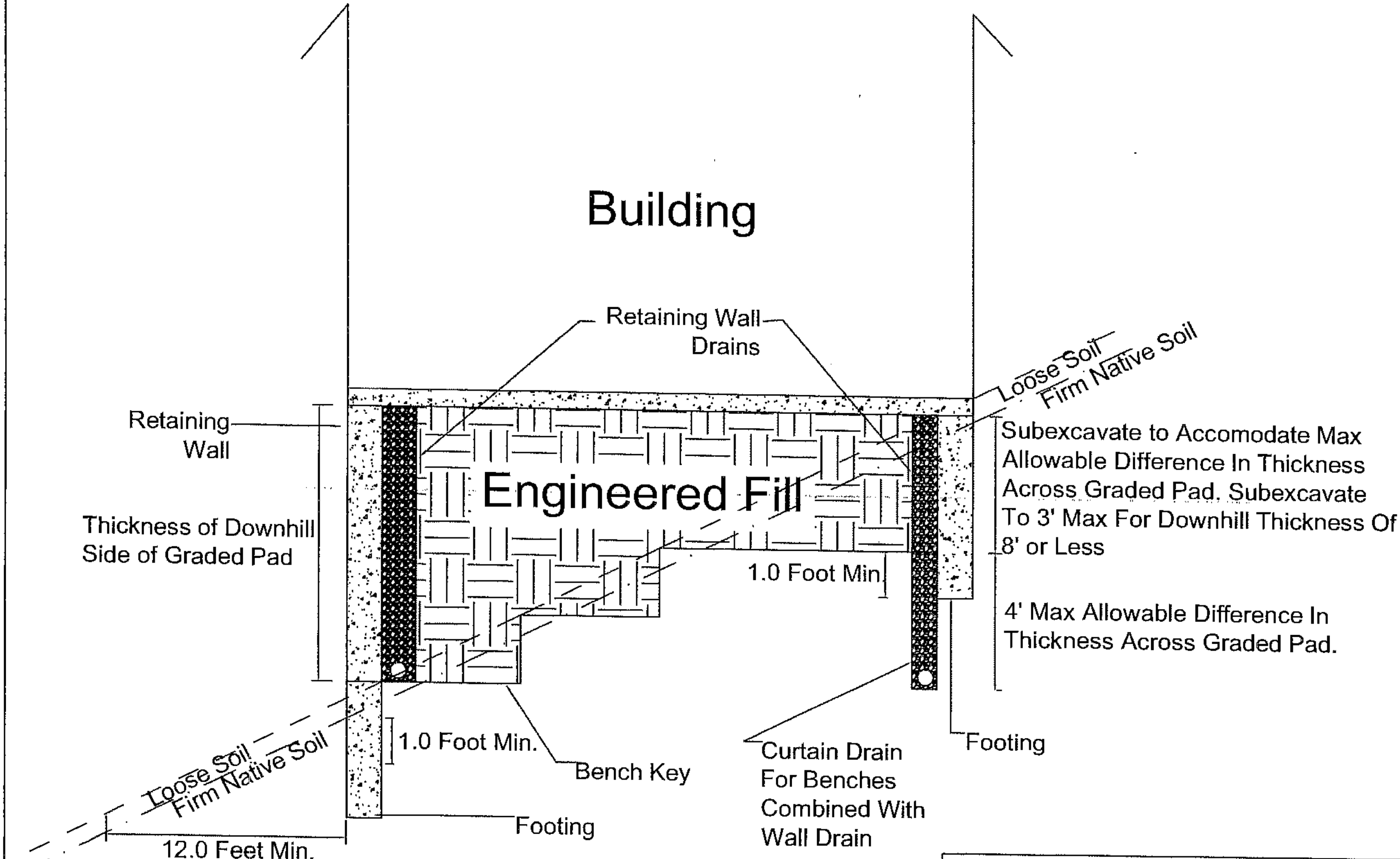
For Illustration Purposes Only Not For Use in Construction .

	Interior Crawl Space Drain Downstream Side Pebble Beach Area D Pebble Beach, California
No Scale	
MC	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
April 2013	
M10473	
FIGURE NO. 24A	SHEET NO. 58



FOR ILLUSTRATION PURPOSES ONLY NOT FOR CONSTRUCTION

SUPPORT OF CONCRETE SLAB WITH FILL PAD Pebble Beach Area D Pebble Beach, California	
No Scale	
MC	
April 2013	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
M10473	
FIGURE NO. 25	SHEET NO. 59



FOR ILLUSTRATION PURPOSES ONLY NOT FOR CONSTRUCTION

SUPPORT OF CONCRETE SLAB WITH FILL PAD

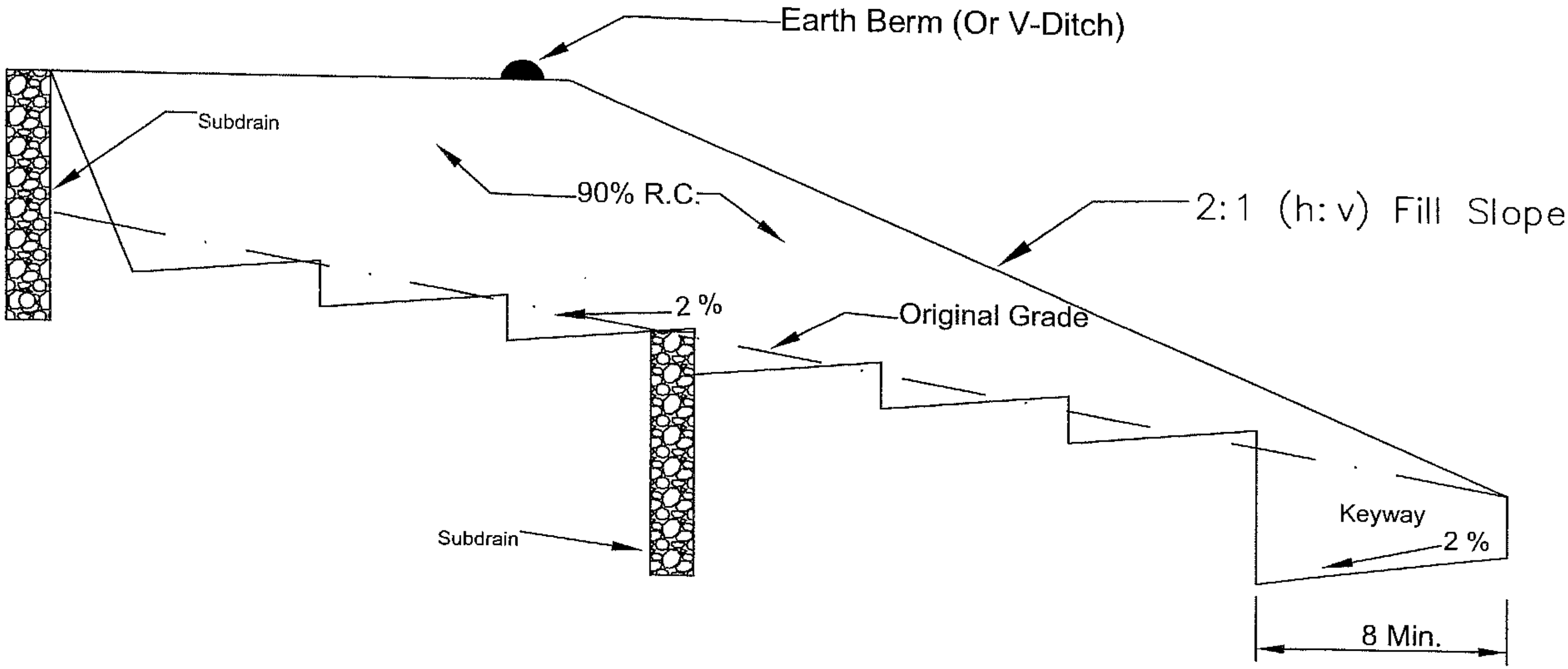
Pebble Beach Area D
Pebble Beach, California

No Scale
MC
April 2013
M10473

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 26

SHEET NO. 60



Notes:

1. For slopes greater than 20' high an intermediate bench at least 8 feet wide shall be constructed
2. Refer to geotechnical report for compaction testing
3. Downslope side of keyway to be at least 2 feet deep below native grade, to be verified by Geotechnical Engineer
4. Drawing is not to scale
5. Refer to Geotechnical Report for subdrain construction requirements
6. Fills Situated on slopes steeper than 5:1 (H:V) should be keyed and benched
7. DRAWING FOR ILLUSTRATIVE PURPOSES ONLY

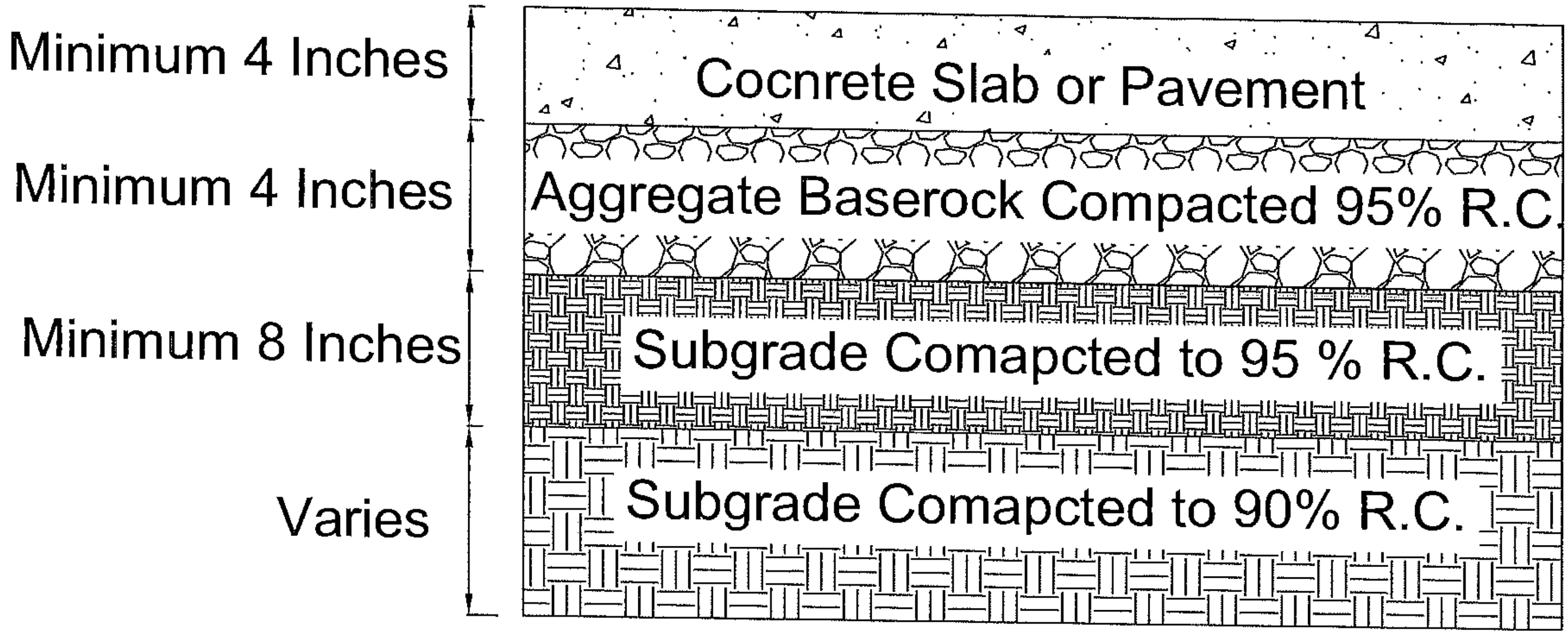
KEY AND BENCH DETAIL
Pebble Beach Area D
Pebble Beach, California

SCALE:	No Scale
DRAWN BY:	MC
DATE:	April 2013
REVISED:	
JOB NO.	M10473

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

Figure No. 27

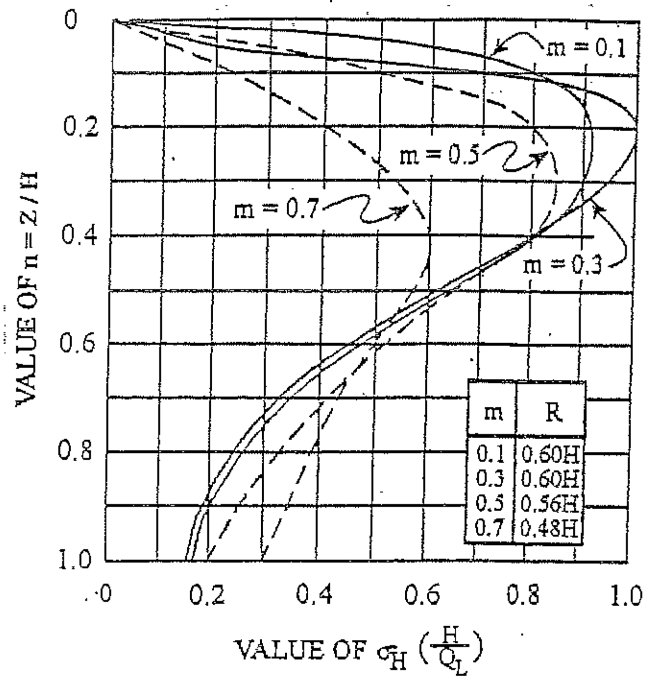
SHEET NO. 61



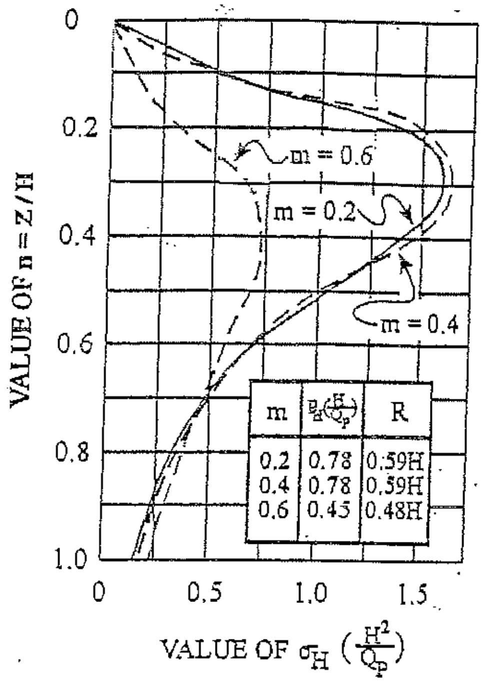
FOR ILLUSTRATION PURPOSES ONLY NOT FOR CONSTRUCTION

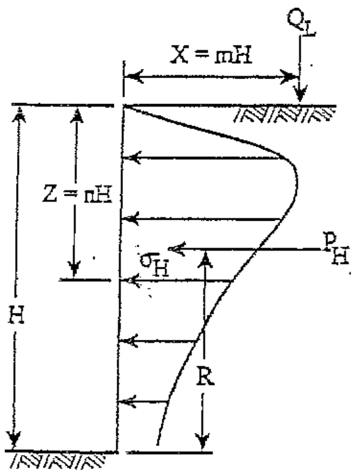
Typical Pavement or Concrete Slab Section Pebble Beach Area D Pebble Beach, California	
No Scale	
MC	
April 2013	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
M10473	
FIGURE NO. 28	SHEET NO. 62

LINE LOAD



POINT LOAD





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:

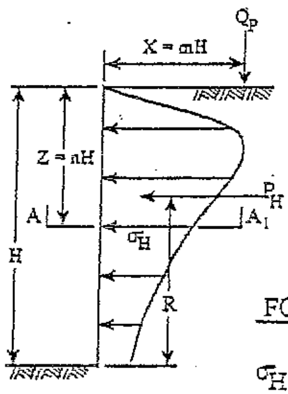
$$\sigma_H \left(\frac{H}{Q_L}\right) = \frac{1.28\, m^2 n}{(m^2 + n^2)^2}$$

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

PRESSURES FROM LINE LOAD Q_L

(BOISSINESQ EQUATION MODIFIED BY EXPERMENT)

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



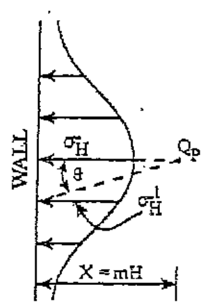
FOR m ≦ 0.4:

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

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)

Haro Kasunich & Associates Geotechnical and Coastal Engineers	Surcharge Pressure Diagram NAVFAC 7.2 Figure 11, Page 7.2-74	Project No. M10473 Area D Pebble Beach California April 2013	Figure No. 29