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REPORT
GEOTECHNICAL INVESTIGATION

ZEIGER RESIDENCE
WILLOW AVENUE, AP 001-193-13
FAIRFAX, CA

2 MARCH 2014

EXHIBIT # E

SALEMHOWESASSOCIATES INC.
GEOTECHNICAL ENGINEERS AND GEOLOGISTS

Willow Ave AP

2 March 2014

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Job :1402008

SUBJECT: Report
Geotechnical Investigation,
Willow Avenue Lot AP 001-193-13
Fairfax

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Introduction

This report presents the results of our geotechnical investigation of the proposed residential building site located at the above address. It conforms to the requirements of section 1803 in the 2010 California Building Code (CBC). The purpose of our investigation was to evaluate the geotechnical feasibility of the proposed development, assess the suitability of the building site, and provide detailed recommendations and conclusions as they relate to our specialty field of practice, geotechnical engineering and engineering geology. The scope of services specifically excluded any investigation needed to determine the presence or absence of issues of economic concern on the site, or of hazardous or toxic materials at the site in the soil, surface water, ground water, or air.

If this report is passed onto another engineer for review it must be accompanied by the approved architectural and structural drawings so that the reviewer can evaluate the exploration and data in the context of the complete project. Ground conditions and standards of practice change; therefore, we should be contacted to update this report if construction has not been started before the next winter or one-year from the report date.

For us to review the drawings for compliance with our recommendations the four following notes must be on the structural drawings:

- The geotechnical engineer shall accept the footing grade / pier holes prior to placing any reinforcing steel in accordance with the CRC requirements. Notify geotechnical engineer before the start of drilling. (If that isn't stated they may require inspections in accordance with CBC Section 1702-Definitions, "Special Inspections, Continuous". This would require a full time inspector during drilling.)
- Drainage details may be schematic, refer to the text and drawings in the geotechnical report for actual materials and installation.
- Refer to Geotechnical Report for geotechnical observation and acceptance requirements. Along with the structural drawings, to complete the review, we need the pertinent calculations from the structural engineer or the geotechnical design assumptions should be included on the drawings notes per requirements of the 2010 CBC.
- ***It is the owner's responsibility*** that the contractor knows of and complies with the BMP's (Best Management Practices) of the Regional Water Quality Control Board, available at www.swrcb.ca.gov, ↲ water quality ↲ stormwater ↲ construction

The fieldwork consisted of reconnaissance mapping of exposed geologic features on the site and in the immediate surrounding area and the drilling of three test borings. The borings were advanced using a portable hydraulic drill rig with 3-inch flight augers and sampled by Standard Penetration Tests* (see "notes to borings logs"). Fieldwork was conducted in February of 2014. During this period we reviewed select geotechnical references pertinent to the area and examined stereo-paired aerial photographs of the site, which were available from Pacific Aerial Surveys in Oakland.

Discussion and Summary

Bedrock is located five feet below the surface in the footprint of the proposed structure. According to Drawing A.5.1, all of the excavations will be in a rock cut and all foundations will be footing type construction.

During our investigation we did not observe any local geologic hazards that would adversely affect the site. We judge that following the recommendations in this report and standard Marin County hillside construction practices a structure can be safely constructed on this site without adversely impacting the slope stability or changing the drainage in any measurable manner. Detailed discussions and recommendations are covered in the following sections of this report.

Geology and Slope Stability

The site has been mapped by others (1) as the Cretaceous Sandstone [Ks] member of the Franciscan Geologic Assemblage. The bedrock as described in the literature is exposed in various road cuts within a few surrounding roadways and driveways. The bedrock is described as fractured, interbedded to massive sandstone and shale, and is highly indurated to somewhat sheared. The bedrock encountered within the borings is massive and fractures sandstone, highly weathered within the residual soil horizons and is stiff to very hard where bedrock is encountered. All of the borings encountered one foot silty [ML] topsoil overlying residual soil with bedrock being encountered at three feet at boring "A" and at five and one half feet at boring "B" and at five feet at boring "C". Rock of this formation has been classified (1) as highly stable on natural slopes and fresh sandstone and shale will stand in vertical cuts except where blocks slip along outward dipping joints or bedding planes. The rock weathers readily to a sandy or silty, non-swelling, easily erodible soil. Rock surfaces of low relief are covered with a thick layer of deeply weathered soil; however steep slopes are stripped essentially bare of soil cover. Landslides and debris flows in this formation are confined to well-developed swales and drainages where deep soil deposits have accumulated. The topographic position of this property does not expose it to these types of natural hazards. During our investigation we did not identify any geomorphic features that would indicate that any unusual geologic hazards would affect this site.

Ground Water

Ground water was not observed in the test borings /pits during our investigation and there were no seeps or clumps of Pampas Grass (Cortaderia Selloana), which are indicators of high ground water. However, ground water conditions vary with the seasons and annual fluctuations in weather. A general rise in ground water can be expected after one or more seasons of above average rainfall. Based on the limited time we have been able to collect ground water data on this site, it is not possible to accurately predict the range of ground water fluctuations in the future. Therefore, ground water sensitive structures such as basements and wine cellars should be designed to anticipate a rise in the water level that could potentially affect their function and stability. During construction it should be anticipated that ground water will be encountered at the rock/soil contact.

Earthquake Hazards and Seismic Design

This site is not subject to any unusual earthquake hazards, located near an active fault, within a current Alquist-Priolo Special Studies Zone or Seismic Hazards Zone as shown on the most recently published maps form the California Geologic Society. There were no geomorphic features observed in the field or on air photos, or geologic features in the literature that would suggest the presence of an active fault or splay fault traces. However, historically the entire San Francisco Bay Area has the potential for strong earthquake shaking from several fault systems, primarily the San Andreas Fault which lies approximately seven miles to the southwest and the Hayward/Rodgers Creek Faults, ten miles to the northeast. The U.S. Geologic Survey presently estimates (2) there is up to 21 percent chance of a major quake (Magnitude 8) from 2000 to 2030 on the San Francisco Bay region segment of the San Andreas Fault. The probability is lower north of San Francisco and increases to the south. However, in the same period, there is a 32 percent chance of a major event (Magnitude 7) on the Hayward fault and Rodgers Creek Faults. The total 30-year probability of one or more large earthquakes occurring in the entire San Francisco region is 70 percent (see Plate 1). Based on the bedrock and soils observed at the site, we do not anticipate those seismically induced hazards, specifically: liquefaction, settlement and differential compaction, landsliding, and flooding are present. Generally speaking structures founded on bedrock fare far better during an earthquake than structures on soil, fill or bay mud.

For California Building Code design purposes this site the top 100 feet of the ground has an average Soil Profile Site of Class B per table 1613.5.2. Seismic Design Site Class and ground-motion parameters, as required by CBC Sections 1613, 1614 and ASCE 7 may be obtained from the calculator on the USGS web site at http://earthquake.usgs.gov/research/hazmaps/design. For seismic design categories D, E or F refer to the Exception in Section 1802.2.7. In California, the standard of practice requires the use of a seismic coefficient of 0.15, and minimum computed Factor of Safety of 1.5 for static and 1.1 to 1.2 for pseudo-static analysis of natural, cut and fill slopes.

Retaining walls which support tall rock cuts will stand vertical with only nominal shoring to prevent weathering. This inherently means there is no active pressure in the rock zone. Therefore, only a nominal value for active pressure is required to support the rock. For seismic analysis the dynamic loads from a slope only occur from the Rankine wedge, which in soils is typically 30 to 40-degrees (from the vertical) in a Ø type material. However, with rock slopes the Rankine wedge is non-existent to near vertical. Consequently there is no measurable seismic force from the slope on the wall in a rock section. In a thin soil section (< 4-ft) the active pressure of 45 lbs/ft^3 is sufficiently conservative to account for any additional seismic loading. In thicker soil sections a simple approach(6) is to include in the design analysis an additional horizontal force P_E to account for the additional loads imposed on the retaining wall by the earthquake, as follows:

$P_E = \frac{3}{8} (\alpha_{max})\gamma_t{}^*H^2$ (acting at a distance of 0.6H above the base of the soil layer)

Where H = height of soil section, $\alpha_{max} = 0.15$ & γ = unit weight of soil in slope. Because P_E = is a short–term loading it is common to allow a ⅓ increase in bearing pressure and passive resistance for earthquake analysis. Also, for the analysis of sliding and overturning of the retaining wall it is acceptable to lower the factor of safety to 1.1 under the combined static and earthquake loads(7).

As a homeowner there are a number of measures one can take to limit structural damage, protect lives and valuable objects in the event of a major earthquake. To be prepared and understand the mechanics of earthquakes we strongly recommend that you purchase a very practical book entitled "Peace of Mind in Earthquake Country" by Peter Yanev. This book is written for the homeowner and, while currently out of print, used copies are available in paperback (Chronicle Books/S.F.) from Amazon.com and other locations.

Foundation Conditions

Sandstone bedrock lies approximately five feet below the surface in the area of development. The depth to the top of bedrock at the location of the test borings is shown on Drawing A. The overlying soil is stiff and will stand in vertical cuts up to five feet when dry. During winter construction shoring will be required. In wet weather ground water can be expected at the soil/rock contact. The rock, albeit hard, is generally highly fractured and can normally be excavated by common means; however, hard massive areas may be encountered that could require the use of an excavator mounted "hoe ram" or core barrel. CalOSHA regulations require shoring on rock cuts over six feet. This is normally most economically accomplished by rock doweling and covering with wire mesh in lifts as the excavation progresses downward. Rock slopes will stand vertically for short periods of time; however, as they are exposed to air and start to dry out block failures will occur; this can happen as soon as the night after excavation.

No laboratory testing was performed; since all foundations will be in rock, soil properties, such as moisture and density, do not provide any relevant engineering data for foundation design. In view of the fact that bedrock features in the Franciscan Formation can rarely be correlated over short distances, testing of small rock pieces provides no viable data for use in design. We based our recommendations on assessment of rock mass properties. During exploration in situ testing and sampling of the soil was performed by Standard Penetration Tests (ASTM D-1586)*. We will continue to evaluate the ground conditions during excavation and modify our recommendation if warranted.

Bedrock is not exposed on the site; however there are road cuts in the are that have typical rock exposures for evaluation of engineering properties. The contractor may use these exposures to determine the difficulty of excavation and the appropriate type of equipment to use.

Structures with foundations on rock will not experience any measurable settlement and there are no conditions that require provisions to mitigate the effects of expansive soils, liquefaction, soil strength or adjacent loads. The slope setback provisions in §1808.7 of the CBC do not apply to foundations on slopes that are bottomed in bedrock. Except for seismic none of the requirements in CBC § 1803.5.11 and .12 apply.

Design Recommendations

All foundations must bear on the unweathered sandstone bedrock by footing type foundations. The depth to rock can be interpolated from the data on Drawing A. Retaining walls in a full rock cut with the recommended toe confinement may use footing type foundations. For tall retaining walls the use of tiebacks for lateral restraint should be considered in lieu of deep keyways or piers. With rock cuts, rock bolting and shotcrete (reinforced shotcrete) may be an economic alternative to traditionally formed retaining walls. There are now local contractors with jackleg air-tract drills that can readily install rock bolts. Per CalOSHA regulations shoring will be required on rock cuts over six feet.

Summary of Design Parameters

Design parameters in this report were determined by field observations and testing and per section 1806.2 of the CBC supersede the presumptive values in the CBC table 1806.2.

- Seismic Design (See Earthquake Hazards Section)
  Soil Profile Site Class Type B, Ground motion parameters from USGS web site at http://earthquake.usgs.gov/research/hazmaps/design with site coordinates.
- Active earth pressure: (see lateral loading formula in Eq. and Seismic Design Section)
  In a Soil Section = 45 lbs/ft^3 equivalent fluid pressure
  In a Rock Section = 35 lbs/ft^2 (pounds per square foot)

- Allowable Bearing Capacity (P_{allow}) On Bedrock[1]
  $P_{allow.}$ = 0.33 * 10.0 * (footing width in feet) = (kips/ft^2) (Not to exceed 10.0)
  A 20-percent increase is allowed for each additional foot, beyond one-foot, of depth that the footing is excavated into the bedrock subgrade.
- Lateral Bearing In Bedrock
  Passive equivalent fluid pressure of 750 lbs/ft^3 and a friction factor of 0.45 to resist sliding. They may be combined and a one third increase is allowed for transitory loading.
- Tiebacks
  Refer to Table 1
- Foundation Drainage
  Include items in "Drainage Check List"

Details on the application of these design values are included in the following sections of this report.

Footings
Footing foundations may be used where the entire footing is excavated into unweathered rock. For retaining wall footings the toe of the footing must be excavated into rock, if a keyway is not used the top of the toe must have three feet of horizontal confinement in the unweathered rock..

As a minimum, spread footings should conform to the requirements of Section 1809 of the CBC except that for foundations bottomed on rock the "Depth Below Undisturbed Ground Surface" in the Table shall be interpreted as to mean "The Depth Below the Top of Weathered Rock". The footings should be stepped as necessary to produce level bottoms and should be deepened as required to provide at least 10 feet of horizontal confinement between the footing base and the edge of the closest slope face. Stepped footing configuration per 1809.3 shall be accepted by the soil engineer. In addition, the base of the footing should be below a 30 degree line projected upward from the toe of the closest cut slope or excavation. For geotechnical considerations, since rock and soil are discontinuous media, footings should be connected up and downslope in a grid like fashion by tie beams. Isolated interior and deck footings should be avoided.

The maximum allowable bearing pressure for dead loads plus Code live loads for footing type foundations bottomed in rock can be determined by the following formula[1] :

$P_{allow.}$ = 0.33 * 10.0 * (footing width in feet) = (kips/ft^2) (Not to exceed 10.0)

A 20-percent increase is allowed for each additional foot, beyond one-foot, of depth that the footing is excavated into the subgrade. The portion of the footing extending into the undisturbed subgrade may be designed with a coefficient of passive earth pressure (K_p) equal to 6.0 with rock unit weight of 130 lbs/ft^3 or a passive equivalent fluid pressure of 750 lbs/ft^3 and a friction factor of 0.45 to resist sliding. Lateral bearing and lateral sliding may be combined and a one third increase is allowed for transitory loading.

Note: (The allowable bearing pressure was based on visual rock mass classification and one-half the presumptive value in NAVFAC DM-7.2 Table 1[1] for this rock type; lateral bearing was calculated assuming $\emptyset = 45°$ and $\gamma = 130$ lbs/ft^3)

Retaining Walls
All retaining walls should be supported on rock by piers or spread footing type foundations. Design parameters for retaining wall foundations are covered under the appropriate section for footings The

toe of footing type retaining walls should be excavated below grade and the concrete poured against natural ground, the toe should not be formed.

Retaining walls supporting *sloping soil slopes* or the soil portion of the cut above the rock contact should be designed for a coefficient of active *soil* pressure (K_a) equal to 0.41, or an equivalent fluid pressure of 45 lbs/ft$^{3(4)}$. For seismic loading from the soil portion of the cut, refer to the previous section on Seismic Design. Since the backfill never truly provides rigid support that prevents mobilization of the active pressure, this value is appropriate for normal or restrained walls. For rigid, tiedback retaining walls that support *soil* slopes an "at rest" value of the coefficient of active soil pressure (K_o) equal to 0.55 or 72 lbs/ft^3 equivalent fluid pressure should be used. Based on the principles of Rock Mechanics, when protected from erosion intact bedrock does not produce an active fluid pressure with a triangular distribution; therefore, the portion of any wall *supporting a rock backslope may be designed for a nominal pressure of 35 lbs/ft^2* (yes, that is square feet). See Drawing A for the depth of the soil layer

When determining wall loads the civil structural engineer should consult with us if using a proprietary design program to be sure the soil loads are appropriately applied.

Allowable foundation bearing and lateral resistance to sliding should be obtained from the formulae in the respective sections on pier or footing foundations. The factor of safety may be reduced to 1.1 for combined static and dynamic loading.

If the shoring is constructed with rock bolts (see following sections), reinforced shotcrete may be used in lieu of structural concrete walls. Conventional concrete structural retaining walls may be constructed without forming by using shotcrete and chimney drains. However, complete waterproofing with this system is very difficult and one should consult a waterproofing specialist.

Piers for 'garden' type walls (supporting only landscaping) founded in the stiff soil may be designed using the criteria in section 1807.3.2.1 (Equation 18-1) of the CBC, with an allowable lateral bearing pressure of 200 lbs/ft^2/ft of depth to calculate S_1. Also Marin County Standard Type A, B or C may be used$^{(3)}$.

All retaining walls should have a backdrainage system consisting of, as a minimum, drainage rock in a filter fabric (e.g. Mirafi™ 140N) with at least three inch diameter perforated pipe laid to drain by gravity. If Caltrans specification Class 2 Permeable is used the filter fabric envelope may be omitted. The pipe should rest on the ground or footing with no gravel underneath. **The pipe should be rigid drainpipe, 3000 triple wall HDPE, 3 or 4 inch ID, ASTM F810 or Schedule 40.** Pipes with perforations greater than 1/16 inch in diameter shall be wrapped in filter fabric. A bentonite seal should be placed at the connection of all solid and perforated pipes. All backdrainage shall be maintained in a separate system from roof and other surface drainage. The two systems may be joined two-feet in elevation below the lowest backdrain at a bubbler to prevent surface water from backing up and into the backdrainage system. Cleanouts should be provided at convenient locations, per §1101.12 of the CPC; however, that is a plumbing and maintenance consideration and not a geotechnical concern.

Retaining walls which are adjacent to living areas should have additional water proofing such as three dimensional drainage panels and moisture barriers (e.g. "Miradrain™ 6000" panels and "Paraseal™") and the invert of the drainage pipe should be a minimum of four inches below the adjacent interior finished floor or crawl space elevation. Drainage panels should extend to 12 inches below the surface and be flashed to prevent the entry of soil material. The heel of the retaining wall footing should be sloped towards the hill to prevent ponding of water at the cold joint; the drainage pipe should be placed on the lowest point on the footing. The backslope of the retaining walls should be ditched to drain to avoid infiltration of surface run-off into the backdrainage system. All waterproofing materials must be installed in strict compliance with the manufacturer's specifications. A specialist in waterproofing should be consulted for the appropriate products, we are not waterproofing experts and do not design waterproofing, we only offer general guidelines that cover the geotechnical aspect of drainage. We have worked with Division 7 in Novato for waterproofing design services.

Tiebacks

The anchor section of the tieback must be in unweathered bedrock. The capacity of tiebacks should be determined by the methods in Table 1, Capacity of Anchor Rods in Fractured Rock⁽¹⁾, which does not use an unbonded length. While a ten-foot long unbonded length is preferred it is not necessary to develop the low capacity tieback normally required for retaining wall stability. One should observe the property lines and not extend the tiebacks into the adjacent property.

Regardless of the type of anchor used (e.g. mechanical, grouted or helical) tiebacks must meet the following two criteria:

- Proof testing to 1.25 times the design capacity
- Depth of anchor must equal or exceed that determined by Table 1

The structural engineer should prepare detailed shop drawings, for approval, of the specific materials and connection methods to be used at the bulkhead. Installation should follow manufacturer's specifications. The anchor rods should be high strength threaded rods specifically manufactured for this application, such as "Williams" or "Dywidag" threadbars. For corrosion protection contact the manufacturer.

Grout should be tremmied to the bottom of each hole so that when the bar is inserted the grout will be displaced to the surface. The bar should be provided with centering guides, and when placed in the hole rotated and vibrated several times to assure thorough contact between the bar and grout.

When the grout has obtained the desired strength the anchor bars should be tested to 125 percent of the design load and tied off at a designated post tensioning load, normally about 33 percent of the design load. The lift-off readings should be taken after the nut has been set to confirm the post tensioning. Typical tieback configuration is attached.

Shoring

For shoring in rock only ("rock" nailing), non-stressed anchors such as Williams Form Engineering Corp "All Thread" bars may be used. For rock and soil shoring the anchors are typically six to eight feet long installed in a 4x4 foot staggered pattern and covered with wire fabric. Shoring should be installed downward in, not more than, six-foot lifts as the excavation progresses. One should observe the property lines and not extend the rock anchors into the adjacent property.

For temporary shoring the "Mackenzie" system is quick and inexpensive to install. The #6 bars are grouted in place, the chain link fabric placed, then #5 horizontal bars are pressed against the fabric and welded to the heads of the #6 dowels acting as wales across the fabric covered slope. In severely fractured rock, vertical bars may also be required.

Typical shoring details are attached.

Geotechnical Considerations for Slab on Grade Construction

Slab on grade construction which spans cut and fill or rock and soil sections will settle differentially and crack. Therefore this type of construction is not recommended for living areas or garages unless the areas are completely excavated into rock or underlain by compacted fill or the slab is designed as a structural slab. If the slab is underlain by a wedge of fill or natural soil over rock a floating slab will still settle differentially, sloping towards the thickest section of fill. Because the loads on a floating slab are usually small the settlement may be negligible.

The base for slabs on grade should consist of a 4-inch capillary moisture break of clean free draining crushed rock or gravel with a gradation between 1/4 and 3/4 inch in size. The base should be compacted by a vibratory plate compactor to 90 percent maximum dry density as determined by ASTM D-1557. A 10-mil impermeable membrane moisture vapor retarder should be placed on top of the gravel. An under-slab drain system, as shown on the attached drawing, should be installed in/under the drainrock. The gravel should be "turned down" by a vibratory roller or plate to provide a smooth surface for the membrane. Recycled material is never acceptable.

Where migration of moisture vapor would be undesirable (e.g. under living spaces and areas covered by flooring) a "true" under-slab vapor barrier, such as "Stego® Wrap", should be installed. In this case one should consult an expert in waterproofing, our recommendations only apply to the geotechnical aspect of drainage and do not address the prevention of mold or flooring failures.

The top of the membrane should be protected during construction from puncture. .Any punctures in the membrane will defeat its purpose. The contractor is responsible for the method of protecting the membrane and concrete placement. *Drains and outlets should be provided from the slab drain rock.* (See attached Drawing for Typical Under-slab Drains)

Cuts and Fills

Unsupported cuts and fills are generally not recommended for this site. Fills behind retaining walls should be of material approved by the geotechnical engineer and compacted to a maximum dry density of 90 percent as determined by ASTM D-1157. Fills underlying pavements shall have the top 12 inches compacted to 95 percent maximum dry density. For fill specifications in utility trenches refer to the project civil drawings; for fill specifications in utility trenches refer to the project civil drawings. Do not used standard PG&E trench specification, as the trench will act as a drain and has caused landslides.

Geotechnical Drainage Considerations
These recommendations apply to the geotechnical aspect of the drainage as they affect the stability of the construction and land. They do not include site grading and area drainage, which is within the design responsibility of civil engineers and landscape professionals. The civil and landscape professionals should make every effort to comply with the Marin County "Stormwater Quality Manual for Development Projects In Marin County" by the Marin County Stormwater Pollution Prevention Program (MCSTOPPP www.mcstoppp.org) and Bay area Stormwater Management Agencies Association (BASMAA www.basmaa.org) when possible.

The site should be graded to provide positive drainage away from the foundations at a rate of 5 percent within the first ten feet (per requirements of the CBC section1804.3). All roofs should be equipped with gutters and downspouts that discharge into a solid drainage line. Gutters may be eliminated if roof runoff is collected by shallow surface ditches or other acceptable landscape grading. All driveways and flat areas should drain into controlled collection points and all foundation and retaining walls constructed with backdrainage systems. Surface drainage systems, e.g. roofs, ditches and drop inlets *must be maintained separately* from foundation and backdrainage systems. The two systems may be joined into one pipe at a drop-inlet that is a minimum of two feet in elevation below the invert of the lowest back or slab drainage system. A bentonite seal should be placed at the transition point between drainpipes and solid pipes.

One should observe the ponding of water during winter and consult with you landscape professional for the location of surface drains and with us if subdrains are required.

All drop inlets that collect water contaminated with hydrocarbons (e.g. driveways) should be filtered before discharged in to a natural drainage.

All cross slope foundations should have backdrainage. In compliance with section 1805.4.2 of the CBC foundation drains should be installed around the perimeter of the foundation. On sloping lots only the upslope foundation line requires a perimeter drain. Interior and downslope grade beams and foundation lines should be provided with weep holes to allow any accumulated water to pass through the foundation. The top of the drainage pipe should be a minimum of four inches below the adjacent interior grade and constructed in accordance with the attached Typical Drainage Details. All drainpipes should rest on the bottom of the trench or footing with no gravel underneath. Drain pipes with holes greater than ⅛-inch should be wrapped with filter fabric, if Class 2 Permeable is used, to prevent piping of the fines into the pipe. If drain rock, other than Class 2 Permeable, is used the entire trench should be wrapped with filter fabric to prevent the large pore spaces in the drain rock from silting up. On hillside lots it may not be possible to eliminate all moisture from the substructure area and some moisture is acceptable in a well-ventilated area. Site conditions change due to natural (e.g. rodent activity) and man related actions and during years of below average rainfall, future ground water problems may not be evident. One should expect to see changes in ground water conditions in the future that will require corrective actions.

All surface and ground water collected by drains or ditches should be dispersed across the property below the structure. Since a legally recognized storm drainage system is not present downslope, we recommend that your attorney be consulted to determine the legal manner of discharging drainage from the roof and surface area drains. It should be noted that improperly discharged concentrated drainage might be a source of liability and litigation between adjacent property owners. The upslope property owner is always responsible to the adjacent lower property owner for water, collected or natural, which may have a physical effect on their property.

One suggestion is that water from drains or ditches should be naturally dissipated across the surface of the slope along a length equal to that of the collected area. Some engineers believe that a buried dispersal system might increase the risk of slope instability and surficial soil sliding. There are numerous civil engineering and landscape solutions to the dispersal of surface water; some are more ascetically pleasing than others, for instance the dispersion pipe can be located behind garden walls or in shrubbery. We should discuss possible solutions with your landscape professional at an appropriate time. Suggested dispersion field details are attached. When it is not possible to locate outfalls in an established drainage, there is a risk that sloughing may occur. The owner should be diligent in maintaining the energy dissipating riprap and correcting minor slumps as they occur. The upslope property owner is always responsible to the adjacent lower property owner for water, collected or natural, which may have a physical effect on their property.

All laterals carrying water to a discharge point should be SDR 35, Schedule 40 or 3000 triple wall HDPE pipe, depending on the application and should be buried. 'Flex pipe' is never acceptable. Cleanouts for stormwater drains should be installed in accordance with §1101.12 of the CPC. However, this is not a geotechnical consideration and is the responsibility of the drainage contractor.

Retaining walls, cut and fill slopes should be graded to prevent water from running down the face of the slope. Diverted water should be collected in a lined "V" ditch or drop inlet leading to a solid pipe.

If the crawl space area is excavated below the outside site grade for joist clearance, the crawl space will act as a sump and collect water. If such construction is planned, the building design must provide for *gravity or pumped drainage from the crawl space.* If it is a concern that moisture vapor from the crawl space will affect flooring, a specialist in vapor barriers should be consulted, we only design drainage for geotechnical considerations.

The owner is responsible for periodic maintenance to prevent and eliminate standing water that may lead to such problems as dry rot and mold.

Construction grading will expose weak soil and rock that will be susceptible to erosion. Erosion protection measures must be implemented during and after construction. These would include jute netting, hydromulch, silt barriers and stabilized entrances established during construction. Typically fiber rolls are installed along the contour below the work area. Refer to the current ABAG(9) manual for detailed specifications and applications. Erosion control products are available from Water Components in San Rafael. The ground should not be disturbed outside the immediate construction area. Prevention of erosion is emphasized over containment of silt. Post construction erosion control is the responsibility of your landscape professional. ***It is the owner's responsibility*** that the contractor knows of and complies with the BMP's (Best Management Practices) of the Regional Water Quality Control Board, available at www.swrcb.ca.gov, ↵ water quality ↵ stormwater ↵ construction. In addition, summer construction may create considerable dust that should be controlled by the judicial application of water spray. After construction, erosion resistant vegetation must be established on all slopes to reduce sloughing and erosion this is the responsibility of a landscape professional. Periodic land maintenance should be performed to clean and maintain all drains and repair any sloughing or erosion before it becomes a major problem.

Drainage Checklist

Before submitting the project drawings to us for review the architect and structural engineer should be sure the following applicable drainage items are shown on the drawings:

- Under-slab drains and outlets
- Crawl space drainage
- Cross-slope footing and grade beam weep holes
- Retaining wall backdrainage pipes with no gravel under the pipes
- Top of retaining wall heel sloped towards rear at ⅛ - inch per foot
- Drain pipe located at lowest part of footing
- Invert of foundation drains located 4-inches below interior grade
- No gravel under any drainpipe
- Upslope exterior foundation drains
- Bentonite seals at drainpipe transition to solid pipe
- Proper installation of the drainage panels
- Outfall details and location
- Subdrains under any fill slopes

In lieu of the above details actually being shown on the drawings there may be a:

- ***Note on the structural drawings***: "Drainage details may be schematic and incomplete, refer to the text and drawings in the geotechnical report for actual materials and installation"

Construction Inspections

In order to assure that the construction work is performed in accordance with the recommendations in this report, SalemHowes Associates Inc. must perform the following applicable inspections. We will provide a full time project engineer to supervise the foundation excavation, drainage, compaction and other geotechnical concerns during construction and accept the footing grade / pier holes prior to placing any reinforcing steel in accordance with the CRC or CBC Section 1702-Definitions and Table 1704.9 continuous inspections for drilled piers and earthwork, if required. Otherwise, if directed by the Owner, these inspections will be performed on an "periodic as requested basis" by the Owner or Owner's representative. We will not be responsible for construction we were not called to inspect. In this case it is the responsibility of the Owner to assure that we are notified in a timely manner to observe and accept each individual phase of the project.

Key Inspection Points

- Map excavations in progress to identify and record rock/soil conditions.
- Accept final footing grade prior to placement of reinforcing steel.
- Accept subdrainage prior to backfilling with drainage rock.
- Accept drainage discharge location.

Additional Engineering Services

We should work closely with your project engineer and architect to interactively review the site grading plan and foundation design for conformance with the intent of these recommendations. We should provide periodic engineering inspections and testing, as outlined in this report, during the construction and upon completion to assure contractor compliance and provide a final report summarizing the work and design changes, if any.

Any engineering or inspection work beyond the scope of this report would be performed at your request and at our standard fee schedule.

Limitations on the Use of This Report

This report is prepared for the exclusive use of Kalman Zeiger and his design professionals for construction of the proposed new residence. This is a copyrighted document and the unauthorized copying and distribution is expressively prohibited. Our services consist of professional opinions, conclusions and recommendations developed by a Geotechnical Engineer and Engineering Geologist in accordance with generally accepted principles and practices established in this area at this time. This warranty is in lieu of all other warranties, either expressed or implied.

All conclusions and recommendations in this report are contingent upon SalemHowes Associates being retained to review the geotechnical portion of the final grading and foundation plans prior to construction. The analysis and recommendations contained in this report are preliminary and based on the data obtained from the referenced subsurface explorations. The borings and exposures indicate subsurface conditions only at the specific locations and times, and only to the depths penetrated. They do not necessarily reflect strata variations that may exist between such locations. The validity of the recommendations is based on part on assumptions about the stratigraphy made by the geotechnical engineer or geologist. Such assumptions may be confirmed only during earth work and foundation construction for deep foundations. If subsurface conditions are different from those described in this report are noted during construction, recommendations in this report must be re-evaluated. It is advised that SalemHowes Associates Inc. be retained to observe and accept earthwork construction in order to help confirm that our assumptions and preliminary recommendations are valid or to modify them accordingly. SalemHowes Associates Inc. cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

In preparation of this report it is assumed that the client will utilize the services of other licensed design professionals such as surveyors, architects and civil engineers, and will hire licensed contractors with the appropriate experience and license for the site grading and construction.

We judge that construction in accordance with the recommendations in this report will be stable and that the risk of future instability is within the range generally accepted for construction on hillsides in the Marin County area. However, one must realize there is an inherent risk of instability associated with all hillside construction and, therefore, we are unable to guarantee the stability of any hillside construction. For houses constructed on hillsides we recommend that one investigates the economic issues of earthquake insurance.

In the event that any changes in the nature, design, or location of the facilities are made, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by SalemHowes Associates Inc. We are not responsible for any claims, damages, or liability associated with interpretations of subsurface data or reuse of the subsurface data or engineering analysis without expressed written authorization of SalemHowes Associates Inc. Ground conditions and standards of practice change; therefore, we should be contacted to update this report if construction has not been started before the next winter.

We trust this provides you with the information required for your evaluation of geotechnical properties of this site. If you have any questions or wish to discuss this further please give us a call.

Prepared by:

SalemHowes Associates, Inc.
A California Corporation

E. Vincent Howes
No. 965

Reviewed by:

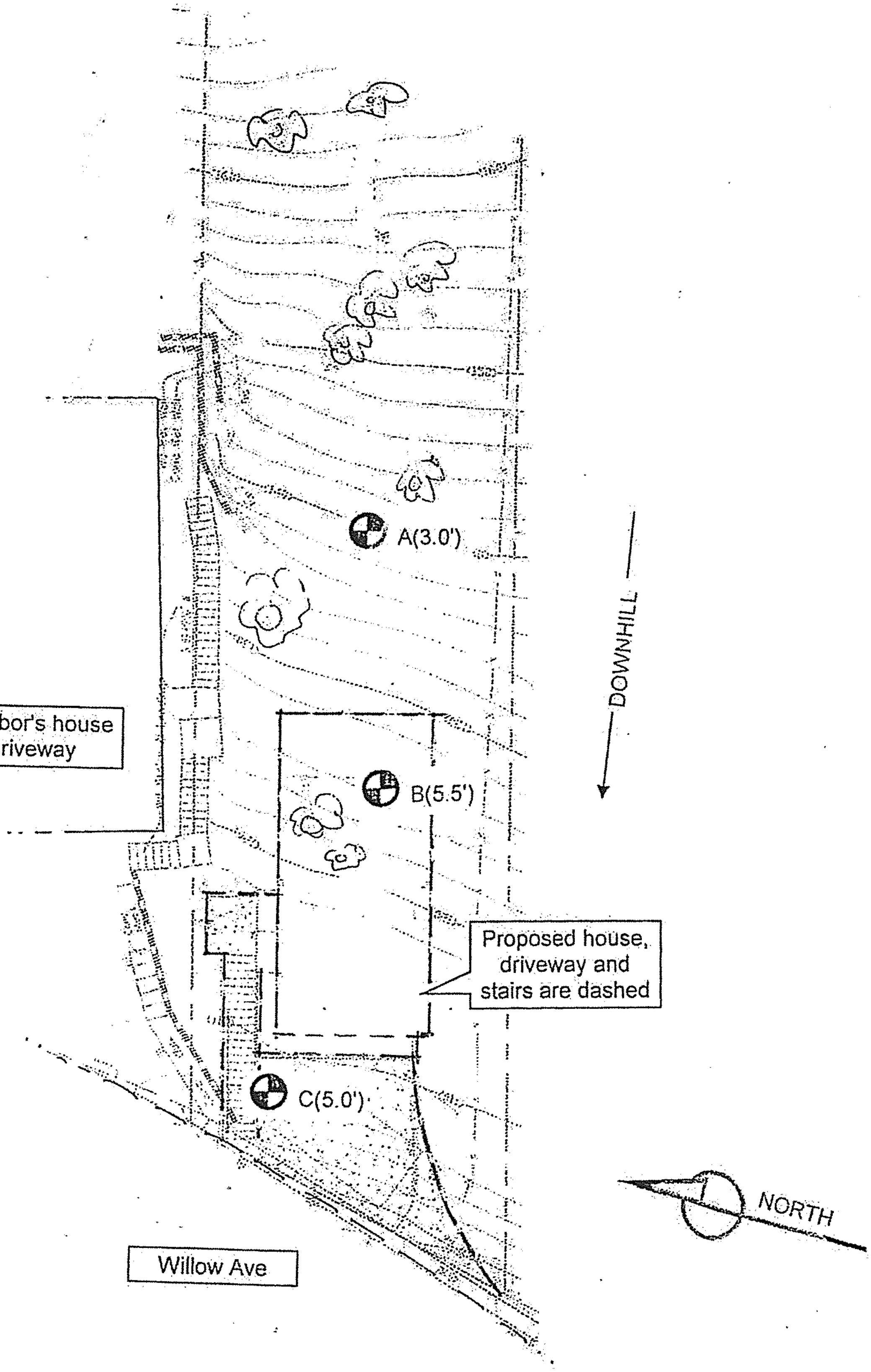
E Vincent Howes

E Vincent Howes
Geotechnical Engineer
GE #965 exp. 31 Mar 14

Attachments:
- Drawing A, Site Plan and Location of Test Borings
- Drawing B, Typical Site Section
- Typical Under-slab Drains
- Typical Drain Detail
- Typical Dispersion Field Details
- Typical Shoring Details
- Typical Retaining Wall Drainage
- Logs of Test Borings
- Plate 1, San Francisco Bay Region Earthquake Probabilities

References: General: 2010 California Building Code and Residential Building Code

(1) Rice, Salem J; Smith, Theodore C and Strand, Rudolph G.; Geology for Planning Central and Southeastern Marin County, California, California Divisions of Mines and Geology, 1976 OFR 76-2 SF.
(2)) USDA, Soil Conservation Service, Soil Survey of Marin County California, March 1985
(2) U.S. Geological Survey, Probabilities of Large Earthquakes in the San Francisco Bay Region, 2000 to 2030, Open-File Report 99-517, 1999
(3) California Department of Conservation, Division of Mines and Geology, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, February 1988, International conference of Building Officials
(4) Department of the Navy, Naval Facilities Engineering Command, Soil Mechanics, Design Manual 7.1, 7.2, (NAVFAC DM-7) May 1982,
(5) Uniform Construction Standards, most recent edition, Marin County Building Department
(6) Leps, Thomas M.,Review of Shearing Strength of Rockfill, Journal of the Soil Mechanics and Foundation Division, Proc. ASCE, Vol.96 No.SM4. July 1970, pp1159
(7) Bowles, Joseph, E., Foundation Analysis and Design, fourth edition, McGraw-Hill, 1988 pg. 614
(8) Seed, H.B. and Whitman, R.V. (1970) Design of Earth Structures for Dynamic Loads. Lateral Stresses in the Ground and Design of Earth Retaining Structures, ASCE, Cornell University
(9) Association of Bay Area Governments (ABAG), Manual of Standards for Erosion & Sediment Control Measures. Most recent edition.
Storm Water Quality Task Force, California Storm Water Best Management Practice Handbooks, Construction Activity, March 1993.
(10) USGS web site at http://earthquake.usgs.gov/research/hazmaps/design



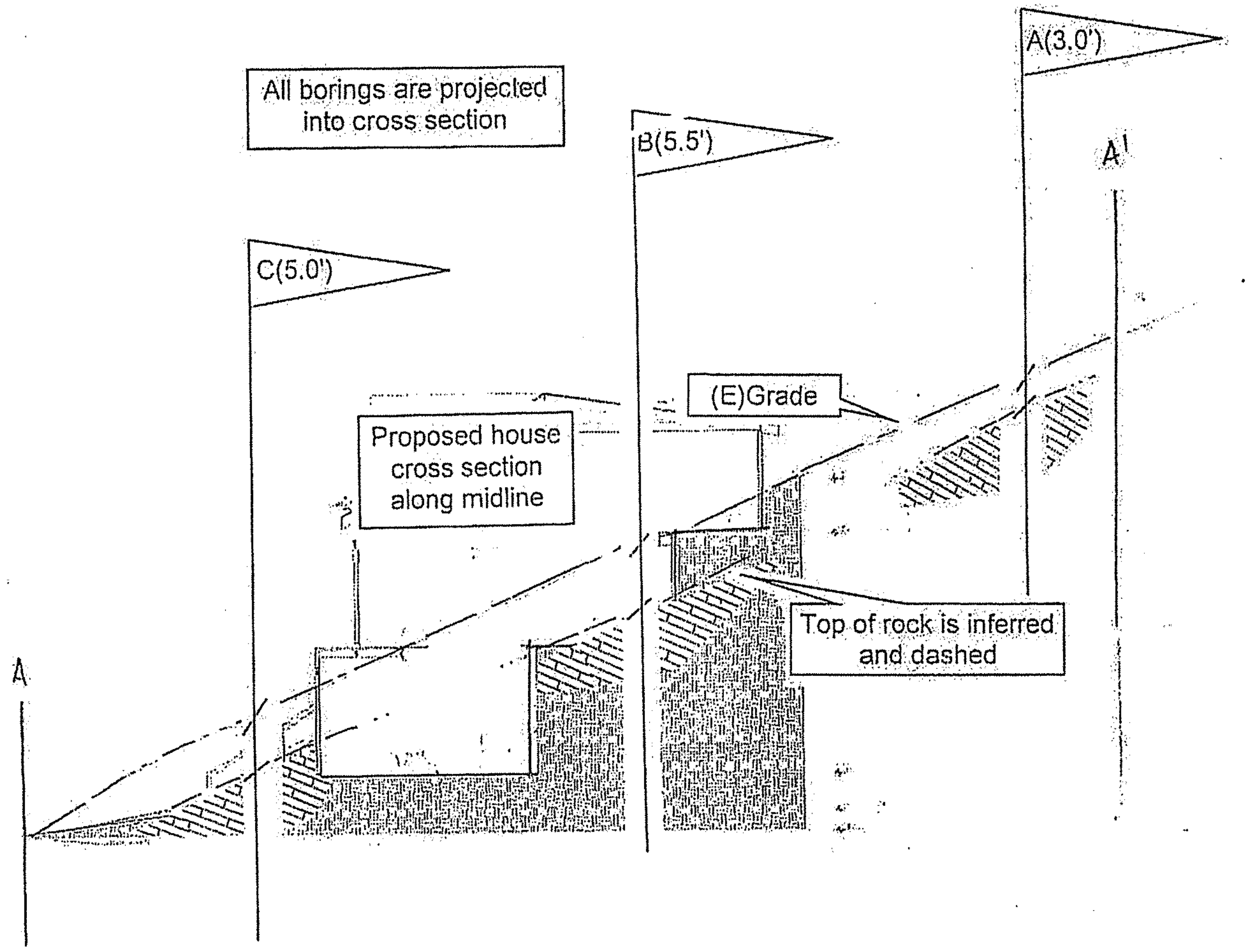
A SITE PLAN AND LOCATION OF TEST BORINGS

REDUCED COPY == S.A.D.

LEGEND

⊕ Location of Borings

(n') Depth of boring in feet



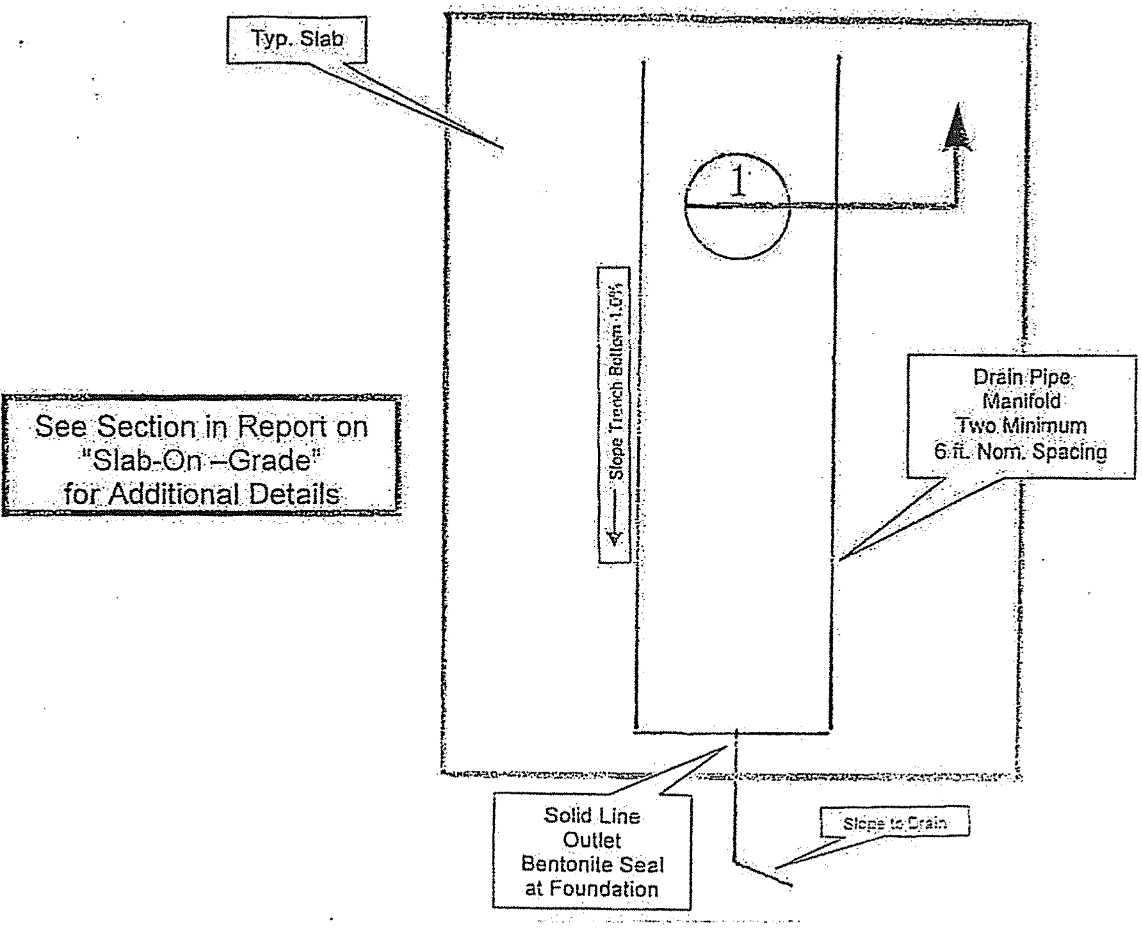
B

Typical Site Sections

SCALE == SEE SCALE BAR

LEGEND

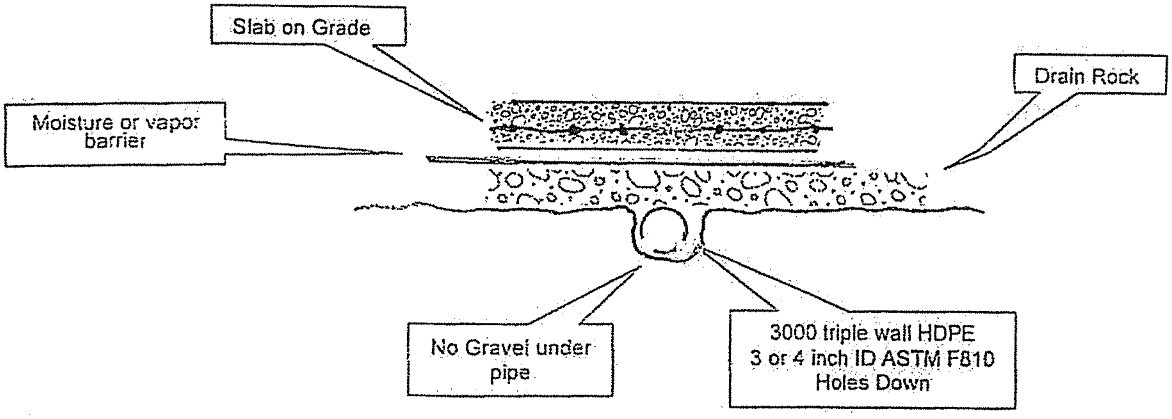
(n') Depth to Rock in feet



See Section in Report on "Slab-On –Grade" for Additional Details

PLAN VIEW

NO SCALE

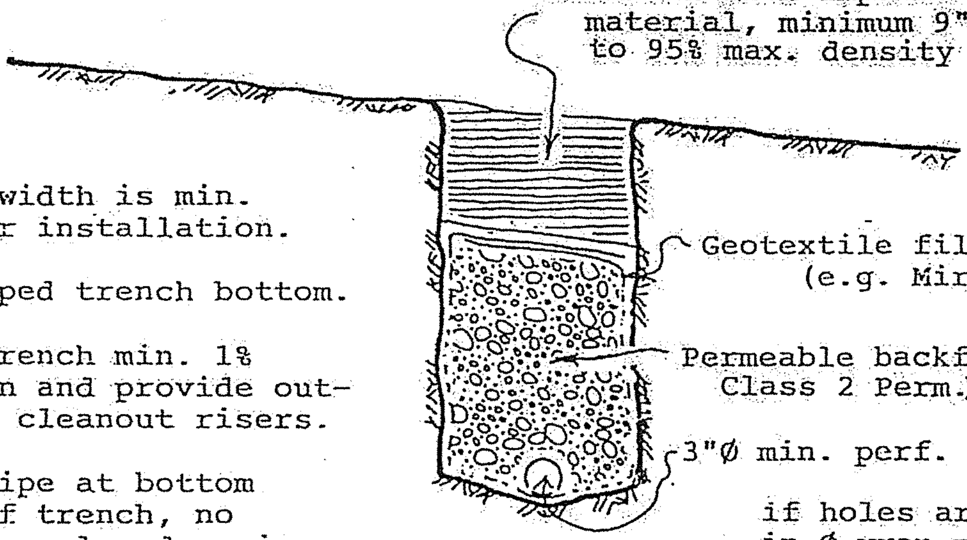


① **Typ. Section Thru Drain Line**

No Scale

TYPICAL UNDERSLAB DRAINS

NO SCALE



Backfill with impermeable (clay rich) material, minimum 9" thick. Compact to 95% max. density per ASTM D-1557.

Trench width is min. req. for installation.

'U' Shaped trench bottom.

Slope trench min. 1% to drain and provide outlet and cleanout risers.

Note: pipe at bottom of trench, no gravel under pipe.
Top of pipe 4" below adjacent interior grade.

Geotextile filter fabric on top. (e.g. Mirafi 140N).

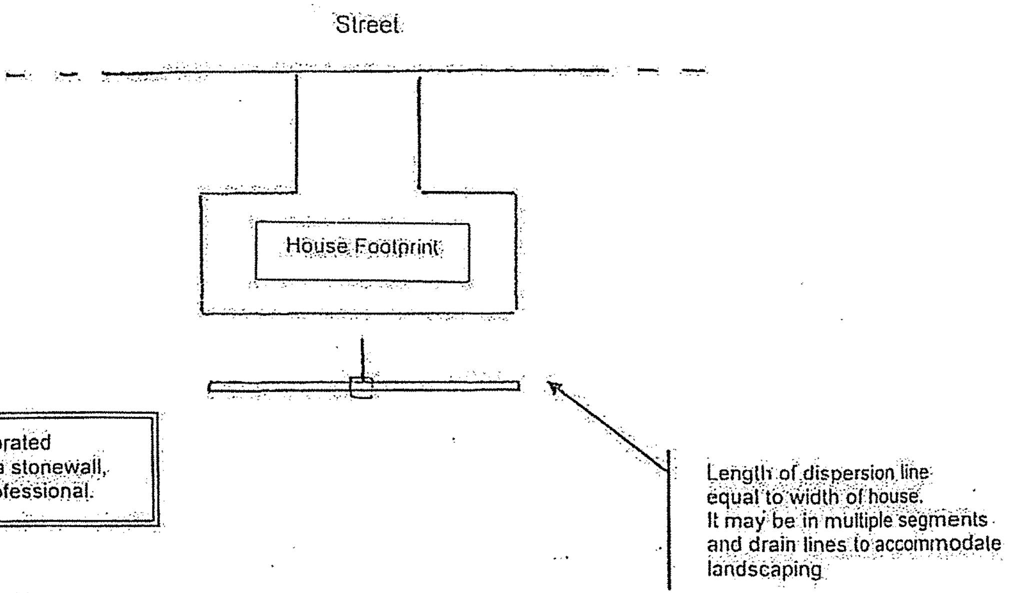
Permeable backfill (e.g. Caltrans Class 2 Perm.) Vibrate into place.

3"Ø min. perf. pipe (See Note) perforations down . if holes are greater than 0.1" in Ø wrap pipe in fabric.

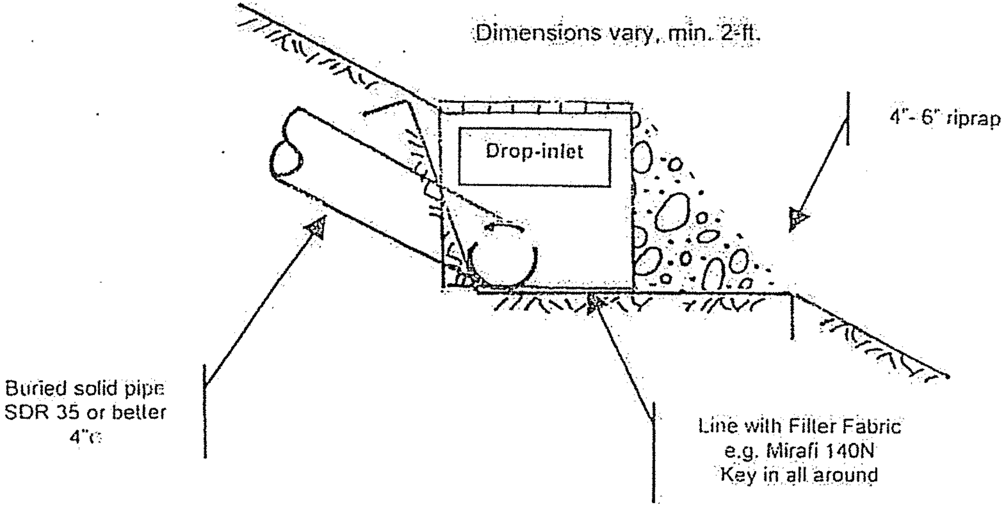
Bentonite clay seal at transition to solid pipe.

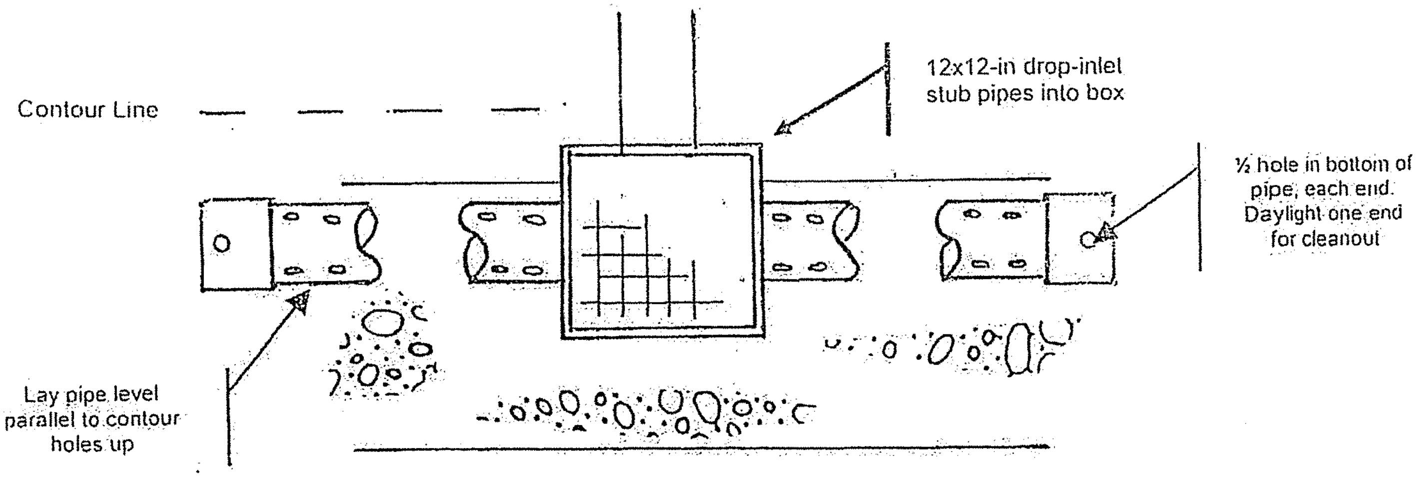
NOTE: We recommend rigid drainpipe 3000 triple wall HDPE, 3 or 4 inch ID, ASTM F810.

TYPICAL DRAIN DETAILS



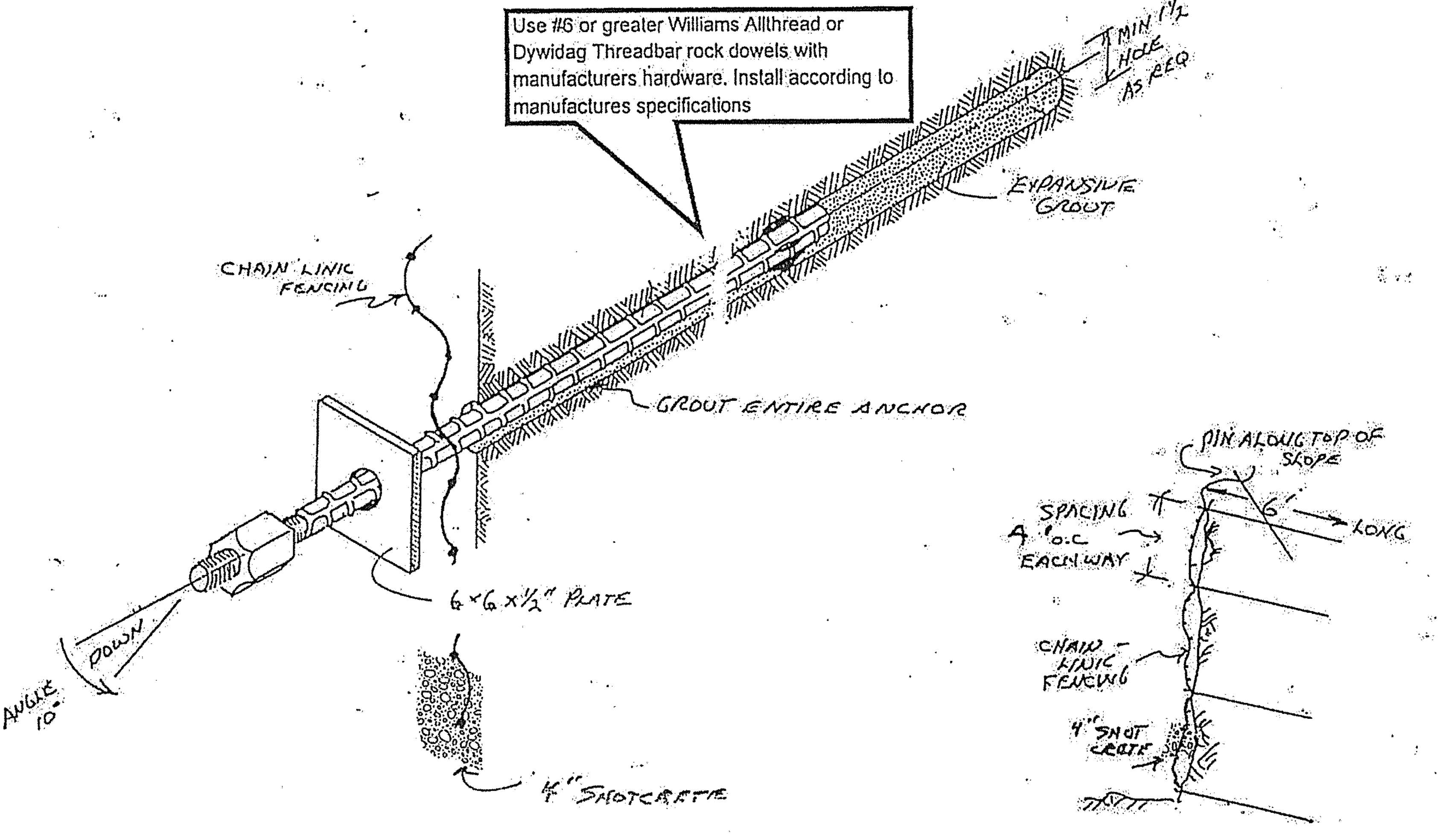
NOTE: Dispersion field may be incorporated into the landscaping, such as behind a stonewall, consult with you landscape design professional.





SKETCH-TYPICAL DISPERSION FIELD DETAILS

NO SCALE



Use #6 or greater Williams Allthread or Dywidag Threadbar rock dowels with manufacturers hardware. Install according to manufactures specifications

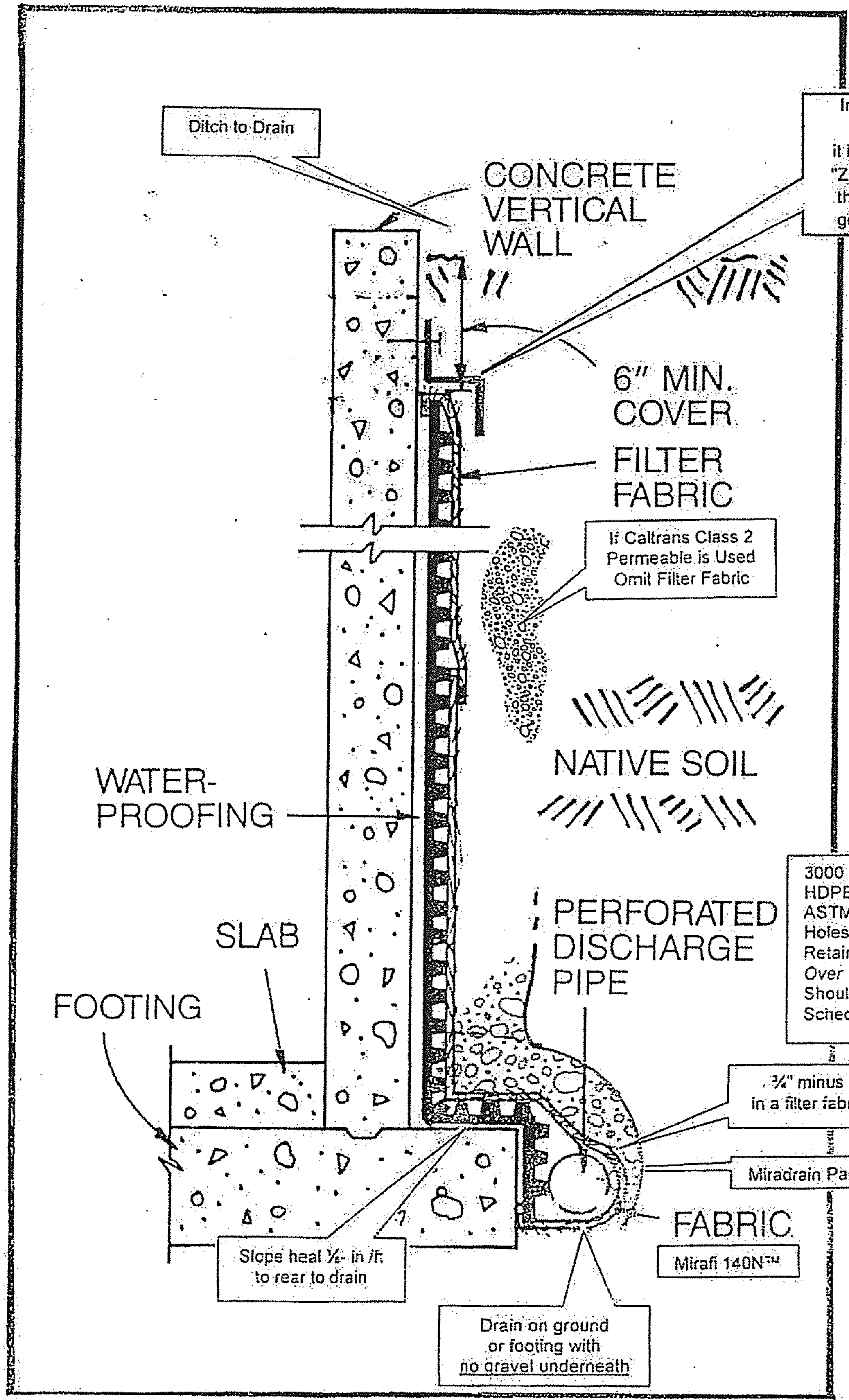
INSTALL BOLT AT BOTTOM OF BENCH BEFORE EXCAVATING FURTHUR.

INSTALL CHAIN LINK AS SOON AS POSSIBLE AFTER.

Alternate: "Mackenzie System" uses regular #6 reinforcing bars and welds #5 wales to the protruding #6 bars.

TYPICAL SHORING INSTALLATION

NO SCALE



In addition to over lapping the fabric at the top it is recommended that "Z" flashing be placed along the top to prevent dirt from getting in behind the panel

3000 triple wall HDPE, 4 inch ID, ASTM F810 Holes Down Retaining Walls *Over 10' high* Should use Schedule 40 pipe

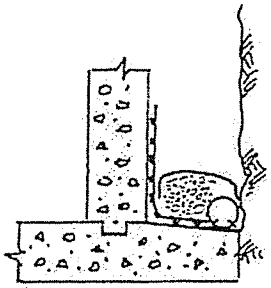
¾" minus drainrock in a filter fabric envelope

Miradrain Panels require a "Burrito" of Drain Rock in Filter Fabric

Backdrainage Options
(one of the three)

- Miradrain™ 6000 Panels
- Drainrock in a filter fabric envelope
- Class 2 Permeable

All options require a moisture barrier on the concrete and a drainpipe.

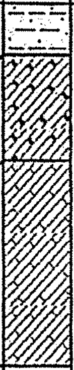


Alternate Drainpipe Location

NOTE: There are alternative flat drain collectors, in lieu of pipes, available from Water Components in San Rafael

TYPICAL RETAINING WALL DRAINAGE DETAILS

PROJECT: Willow Ave AP
BORING: A
ENGINEER: E. V. Howes
LOGGED BY: J. Gillis
JOB # : 1402008
DATE: 19 February 2014

PLASTICITY INDEX (PI)	LIQUID LIMIT	SAMPLE TYPE	(N) Blows Per foot	DEPTH (feet)	WATER LEVEL	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				1		TOPSOIL 0.0'-1.0' dark brown silty [ML] soil with fine rooting and no rock fragments, grades to resiudal soil at 1.0'		
				2 3		RESIDUAL SOIL 1.0'-3.0' reddish brown silty [ML] soil with increasingly frequent weathered sandstone fragments and somewhat moist. grades to bedrock at 3.0'		
		SPT	45	4 5 6 7		SANDSTONE [Ks] 3.0'-4.5' very hard, weathered and somewhat fractured fine to medium-grained sandstone with some rooting within fractures. dry and well indurated		Top of rock 3.0' SANDSTONE [Ks]
				8 9 10 11 12 13 14 15 16 17 18 19 20		End of Log		Ground water was not Encountered in boring

DRILLED BY: TransBay
EQUIPMENT: Portable Hydraulic
BORING SIZE: 3"
SHEET: 1 of 1

PROJECT: Willow Ave AP
BORING: B
ENGINEER: E. V. Howes
LOGGED BY: J. Gillis
JOB # : 1402008
DATE: 19 February 2014

PLASTICITY INDEX (PI)	LIQUID LIMIT	SAMPLE TYPE	(N) Blows Per foot	DEPTH (feet)	WATER LEVEL	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				1		TOPSOIL 0.0'-1.0' dark brown silty [ML] soil with fine rooting and no rock fragments. grades to resiudal soil at 1.0'		
				2 3 4 5		RESIDUAL SOIL 1.0'-5.5' reddish brown silty [ML] soil with increasingly frequent weathered sandstone fragments and somewhat moist. grades to bedrock at 5.5'	▨	
		SPT	28	6 7		SANDSTONE [Ks] 5.5'-7.0' very hard, weathered and somewhat fractured fine to medium-grained sandstone with some rooting within fractures. dry and well indurated	▨	Top of rock 5.5' SANDSTONE [Ks]
				8 9 10 11 12 13 14 15 16 17 18 19 20		End of Log		Ground water was not Encountered in boring

DRILLED BY: TransBay
EQUIPMENT: Portable Hydraulic
BORING SIZE: 3"
SHEET: 1 of 1

SHA

PROJECT: Willow Ave AP
ENGINEER: E. V. Howes
JOB # : 1402008

BORING: C
LOGGED BY: J. Gillis
DATE: 19 February 2014

PLASTICITY INDEX (PI)	LIQUID LIMIT	SAMPLE TYPE	(N) Blows Per foot	DEPTH (feet)	WATER LEVEL	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				1		TOPSOIL 0.0'-1.5' dark brown silty [ML] soil with fine rooting and no rock fragments. grades to resiudal soil at 1.5'	▤	
				2 3 4		RESIDUAL SOIL 1.5'-5.0' reddish brown silty [ML] soil with increasingly frequent weathered sandstone fragments and somewhat moist. grades to bedrock at 5.0'	▨	
		SPT	45	5 6 7		SANDSTONE [Ks] 5.0'-7.0' very hard, weathered and somewhat fractured fine to medium-grained sandstone with some rooting within fractures. dry and well indurated	▨	Top of rock 5.0' SANDSTONE [Ks]
				8 9 10 11 12 13 14 15 16 17 18 19 20		End of Log		Ground water was not Encountered in boring

DRILLED BY: TransBay
EQUIPMENT: Portable Hydraulic

BORING SIZE: 3"
SHEET: 1 of 1

Notes to Boring Logs

1) Soil designations in this report conform to the Unified Soil Classifications per ASTM D22487, Classification of Soil for Engineering Purposes. Rock classifications conform to NAVFAC DM-7.

2) The SPT, Standard Penetration Test, is made using a standard 2" OD - 1.375" ID sampler driven by a 140# hammer falling 30" (per ASTM D-1586). A MPT, Modified penetration Test, is made using the same standard sampler driver by a 70# hammer falling 30". Other sampler and hammer size data for information only. TW indicates a Thin Wall sampler. The sample is driven 18" and the number of blows required to penetrate the last 12" is indicated on the log. "REF" (refusal) indicates the number of blows required to penetrate 6" exceeded 50.

3) Borehole and test pit data are considered representative of the subsurface condition only for the time and location at which the data were obtained. Interpretation or extrapolation of these data represent an exercise in judgment based on education and experience and is not warranted as precisely representing subsurface conditions at all locations. During construction variations will be observed in the field and field design changes should be expected.

4) PP indicates in situ measurements made by a standard pocket penetrometer in tons per square foot unconfined compressive strength.

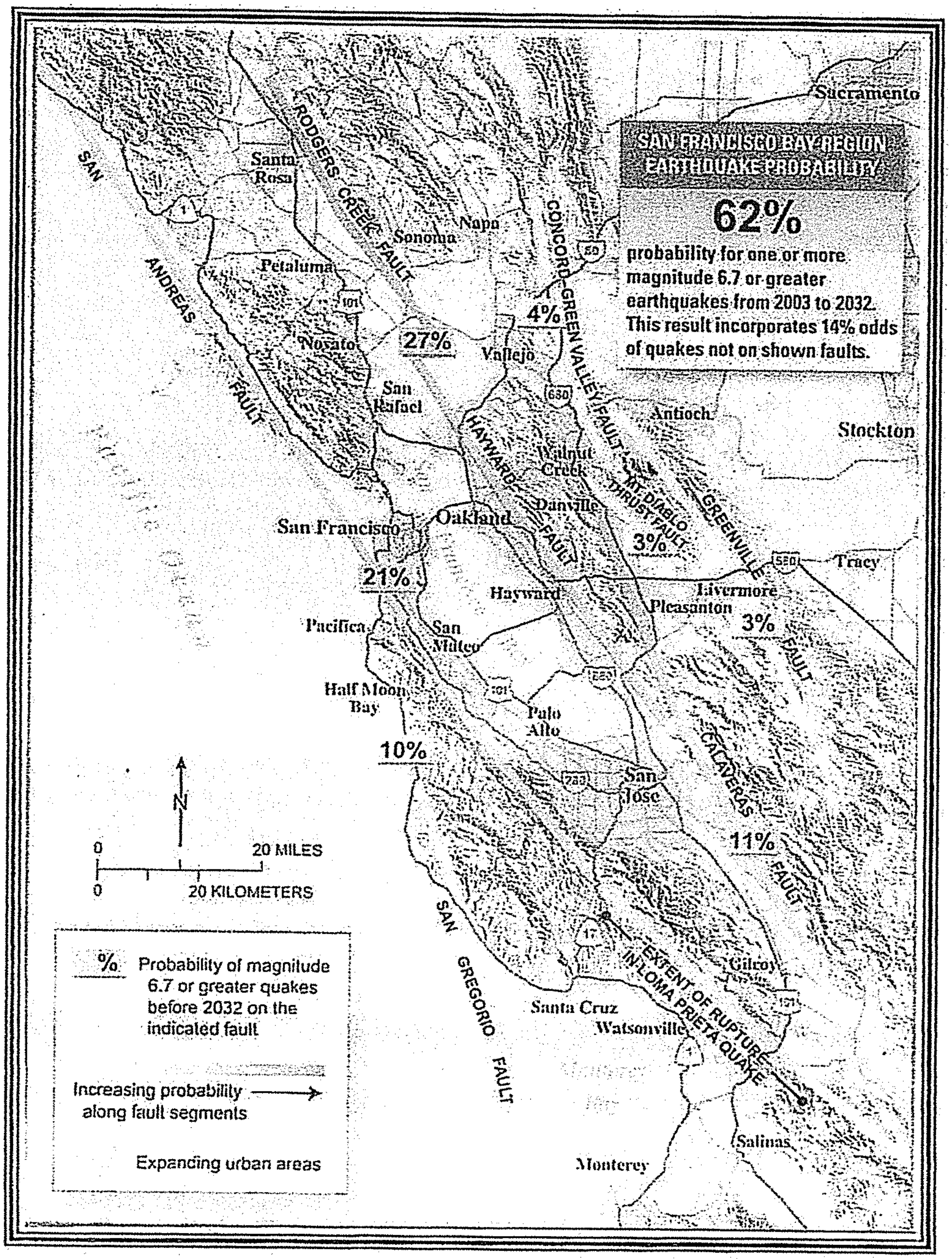
TV indicates in situ measurements made by a Torvane in kilograms per square centimeter.

5) LL indicates the Liquid Limit of soils and
PI indicates the Plasticity Index of soils per ASTM D-4318
Quc indicates the unconfined compressive strength per ASTM D-2166
TX/UU indicates an Unconsolidated Undrained Triaxial Test, Confinement pressure/Ultimate strength in psf.
DD indicates dry density in pcf.
mc indicates moisture content in percent.

6) Qaf = artificial fill
Qc = colluvium
Ks = sandstone bedrock

(*)Colluvium - Unconsolidated and unsorted soil material and weathered rock fragments which have accumulated on or at the base of slopes by natural gravitational or slope wash processes, derived by weathering and decomposition of the underlying bedrock material.

Residual Soil- Soil formed in place by the disintegration and decomposition of the rocks and the consequent weathering of the mineral materials. Presumably developed from the same kind of rock as that on which it lies.



Using newly collected data and evolving theories of earthquake occurrence, U.S. Geological Survey (USGS) and other scientists have concluded that there is a 62% probability of at least one magnitude 6.7 or greater quake, capable of causing widespread damage, striking somewhere in the San Francisco Bay region before 2032. A major quake can occur in any part of this densely populated region. Therefore, there is an ongoing need for all communities in the Bay region to continue preparing for the quakes that will strike in the future.

Plate 1, San Francisco Bay Region Earthquake Probabilities

From: USGS Fact Sheet 039-03
Revised September 2004

Ross Valley Fire Department
777 San Anselmo Avenue, San Anselmo, CA 94960

Mark Mills
FIRE CHIEF

TOWN OF FAIRFAX
SEP 29 2014
RECEIVED

April 1, 2014

Address: 164 Willow, Fairfax
Applicant: G Family Construction
Application #: 14-0082

The Vegetation Management Plan submitted for review by the Ross Valley Fire Department is approved with the following conditions:

Defensible space shall be provided a minimum 100 feet from all structures

All vegetation within the 30 foot zone shall be irrigated.

Every effort shall be taken to ensure erosion control efforts are in compliance with standards established by Town regulations.

The approved plan is to last the life of the property. Any changes to the plan now or in the future will require Fire Department review. It is recommended that if the applicant has plans to landscape in the future that those plans be intermingled into this plan.

Vegetation shall be maintained to ensure address numbers are visible from both angles of approach.

Minimum standards shall be in place prior to final fire clearance.

If you have any questions about any of the items listed above please call me. I am available to meet with you on site to help you develop a plan. Please contact me to schedule (415) 258-4673 if you desire my assistance.

Sincerely,

Robert Bastianon

Fire Inspector

Committed to the protection of life, property, and environment.
SAN ANSELMO • FAIRFAX • ROSS • SLEEPY HOLLOW
HEADQUARTERS: 777 San Anselmo Avenue, San Anselmo, CA 94960 TEL: (415) 258-4686 FAX: (415) 258-4689 www.rossvalleyfire.org

EXHIBIT # F