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REPORT
GEOTECHNICAL INVESTIGATION

BLOM RESIDENCE
31 LINCOLN PARK
SAN ANSELMO, CA

13 DECEMBER 2021

SALEMHOWESASSOCIATES INC.
GEOTECHNICAL ENGINEERS AND GEOLOGISTS

13 December 2021

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SUBJECT: Report
Geotechnical Investigation,
Pool and Landscape Improvements
31 Lincoln Park, San Anselmo

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Introduction

This report presents the results of our geotechnical investigation of the proposed pool and landscape improvements at the above address. It conforms to the requirements of section 1803 in the 2019 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 Chapter 2-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 2019 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 two test boring in the project area. 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 November of 2021. 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 was found at depths averaging six feet below the existing surface at the locations shown on Drawing A. The sheared shale bedrock will provide substantial bearing for drilled pier or footing type foundations. The depth to the top of rock at the location of the test borings is shown on Drawing A. Ground water was not encountered in the test borings.

This is a difficult site and we should work closely with your landscape design professional to optimize the site grading. The upslope features may bottom in bedrock however the downslope structures will need piers into bedrock for support.

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.

These recommendations apply to the structural elements of a pool or spa.

Geology and Slope Stability

The site has been mapped by others [(1)] as the Franciscan Melange [fm] member of ther Franciscan Geologic Assemblage. The mélange is described in the literature as a sheared shale argillite matrix that contains inclusions of sandstone, shale, serpentinite, chert and other exotic rock units. Some of these rock units can be tiny to many tens or hundreds of feet in dimension. Some rock units can be more weathered than others and the sheared shale typically is weathered and forms a thickened residual soil horizon that can be expansive in nature. The residual soil exists below three and one half feet to four and one half feet of colluvium that contains topsoil and rocky debris as encountered at three feet before transitioning to residual soil. Bedrock was encountered in borings "A" and "B" at six feet from existing grade. The site is gently to moderately sloped from the rear of the residence to the rear property line and approaches 2:1 (H:V) in steepness where the proposed pool is to be located. The slope to the east above the property line is developed with housing and associated structures.

Rock of this formation has been classified [(1)] as moderately stable on natural slopes and fresh and highly sheared shale will not stand in vertical cuts for short periods of time as the sheared shale is friable where exposed. The rock weathers readily to a clayey moderately 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 or wet, clayey conditions exist upon moderate slopes. The topographic position of this property along the flanks of the slope may expose it to these types of natural hazards if extremely wet or adverse site conditions are exacerbated such as excess excavation, excess water within excavations or long exposures to the elements. 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 during our investigation yet moist conditions exist in all of the borings. Prior to the winter season, moisture generally increases and is present well into the summer months. Pampas Grass (Cortderia Jubata), blackberry bushes and surface seeps which are indicators of high ground water were not observed on site. 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, pools 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 may be encountered approaching the bedrock horizon.

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 8 miles to the southwest and the Hayward/Rodgers Creek Faults, 10miles to the northeast. The U.S. Geologic Survey estimates [2] (we realize these percentage estimates have been up dated practically every year; however, the basic message is that we live in earthquake country and one should be prepared) 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 on this site the top 100 feet of the ground has an average Soil Profile Site of Class B per Table 20.3-1 ASCE-7. Seismic design criteria in conformance with the latest edition of the CBC and ASCE-7 should be obtained from the USGS web site. 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.

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 six feet feet below the existing 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 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 cuts over five feet. Temporary slopes and shoring design are the responsibility of the contractor.

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.

Design Recommendations

All foundations must bear on the unweathered shale bedrock by drilled pier/piles or footings. Structures with footings in the soil section above he bedrock are not recommended. The depth to rock can be interpolated from the data on Drawings A.

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.

Summary of Design Parameters

The design engineer should compare the topography, building elevations and geotechnical report to determine the appropriate active earth pressures and type of foundation to be used. The actual type of foundation should be determined by the architect and design engineer based on construction and economic considerations. The use of a mixed foundation design is usually a practical solution. 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 with site coordinates.
- Active earth pressure: (see lateral loading formula in Eq. and Seismic Design Section)
  In a Soil Section = 35 for level and 45 lbs/ft^3 equivalent fluid pressure for sloping backslope
  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.

- Pier Design (Per 2019 CBC section 1807)
  Rock passive pressure: 800 lbs/ft^2/ft to calculate S_1 or S_3 (1.5-ft below the top of rock on slopes)
  Adhesion: (skin friction) 900 lbs/ft^2 (In the rock)
- 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.

Drilled Piers (CIDH)

Drilled, cast-in place, reinforced concrete piers should be a minimum of 18 inches in diameter and should extend at least six feet into competent bearing stratum as determined by the Engineer in the field. The structural engineer may impose additional depths. The piers shall extend into the bearing stratum six feet below a 30° line projected up from the bottom of the nearest cut slope or bank. Piers should be designed to resist forces from the gravitational creep of the soil layer. The height of the piers subject to the creep forces is equal to the depth to the top of rock. For design purposes this may be, interpolated from the data on Drawing A. Creep forces should be calculated using an equivalent fluid pressure(3) of 45 lbs/ft^3 acting on two pier diameters. Because the rock and soil are discontinuous media, for geotechnical considerations, the piers should have a nominal spacing of eight feet or less on center and connected by tie and grade beams in a grid like configuration. The piers should be no closer than two-diameters, center to center. In general, isolated interior and deck piers should be avoided. Normally end bearing should be neglected (see conditions below).

Piers should be designed by the formula in section 1807 of the 2019 CBC, with 'P' equal to the soil creep forces between the surface and top of rock (plus any lateral loads from the structure) and 800 lbs/ft^2/ft used to calculate 'S_1' or 'S_3'. ***Note*** that in this formula 'b' is the actual diameter of the pier not a multiple and 'h' is measured from the point of fixity. These values are not appropriate for other methods of design. The structural engineer should contact us for the applicable values if another method of pier design is to be used.

Note: (The value used to calculate "s" for the fractured bedrock was selected by rock mass classification and conservatively assuming the bedrock to be a dense gravel with a Ø = 50° (4) then equating the results of Bowles(3) design for cantilevered sheet piles in a granular soil to the CBC formula. Since bed rock features in the Franciscan Formation can rarely be correlated over short distances, testing of small rock pieces provides no viable data for design. Using these values to calculate "s" in the CBC formula results in a conservative pier depth calculation. The "s" values are not passive pressure in the technical soil mechanics sense; they are only related to the CBC formula)

We judge that when piers are in a full rock cut or the tops are connected by rigid moment connections, in the upslope-downslope direction, fixity occurs at the rock surface and the conditions result in a constrained top of the pier. For this case the depth may be calculated by using the CBC formula in section **1807.3.2.2 Constrained.**

Design Parameters

Depth of fixity below top of bedrock surface for a sloping area:	1.5 feet
Soil active pressure on pier	45 lbs/ft^3 on 2 Ø
Rock active pressure:	$K_a = 0.0$
Rock passive pressure:	800 lbs/ft^2/ft to calculate S_1 or S_3
Adhesion: (skin friction)	900 lbs/ft^2

Neglect adhesion in the soil section

The values recommended for the calculation of "S" incorporate a factor of safety. There is no requirement for the retaining wall designer to add an additional factor of safety for overturning.

Piers drilled into bedrock are completely confined and should not be designed as columns; there is no shear in the pier below the rock surface.

In order for these strength values to be realized, the sides of the pier holes must be scaled of any mudcake.

End bearing may be used if the bottoms of the holes are thoroughly cleaned out with a "PG&E" spoon or other means. Drilled piers may be any convenient diameter that allows for readily cleaning the bottom of the holes. The end allowable bearing capacity may be determined as follows:[1]

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

Bearing may be increased 10 percent of the allowable value for each foot of depth extending below one foot of the rock surface.

Notice: We will not accept the foundation for concrete placement if the pier holes are over 48 hours old and will require that they be redrilled. One should plan ahead and have the pier cages assembled prior to drilling the holes so that there is no delay in placing the concrete. The contractor may submit plans for remedial measures, such as spraying or covering the excavation, to extend this time period. However, acceptance is always subject to the condition of the foundation grade immediately prior to the pour.

Ground water may be encountered in the drilled pier holes and it may be necessary to dewater, case the holes and/or place the concrete by tremie methods. All construction water displaced from the pier holes must be contained on site and filtered before discharging into the storm water system or natural drainages. Hard drilling will be necessary to reach the required depths. The contractor should be familiar with the local conditions in order to have the appropriate equipment on hand. The rock to be encountered in the drilling can be observed in outcrops in the area.

Footings

Footing foundations may only 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] :

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

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 $\varnothing = 45°$ 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 or drilled piers. 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]. Level backslope may use 35 lbs/ft^3 for active pressure. 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.. 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. Any wall where the backfill is subject to vehicular loads within an area defined by a 30-degree (from vertical) plane projected up from the base of the wall or *top of bedrock*, should have the design pressure increased equivalent to a 200-lbs/ft^2 (q') surcharge. In this case if a uniform surcharge load q' acts on the soil behind the wall it results in a pressure P_s in lbs/ft. of wall equal to:

$$P_s = q' * \text{(height of wall)} * K_a \quad \text{(where } K_a \text{ is taken as 0.41)}$$

It acts midway between the top and bottom of the wall. Or the design height of wall may be increased two feet to account for the surcharge.

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.

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.

At the slab-on-grade location remove loose deleterious substances such as expansive clay, rubbish, and organic, perishable or uncompactable material. Compact the footing bottom with a "jumping jack" hand compactor. This applies to larger areas such as the sub-base for slabs-on- grade. If soft areas of soil are encountered at foundation grade they should be overexcavated to firm material as directed by the engineer and backfilled to grade with Caltrans Specification Class 2 Material. All fill densities should be verified by testing procedures ASTM D-1556 and D-1557, or ASTM D-2292 and D-3017 (Nuclear Method).

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 [MDD] of 90 percent as determined by ASTM D-1157. Fills underlying pavements shall have the top 12 inches compacted to 95 percent MDD. Unclassified landscape fills need only be compacted to 80-percent MDD. After clearing and grubbing native soil (if accepted by the engineer) underlying pavements and hardscape shall be scarified to a depth of 12-inches and compacted to 90-pecent MDD. Structural fills shall be compacted to 90-percent MDD and placed under the direction of the geotechnical engineer.

For fill specifications in utility trenches refer to the project civil drawings. Do not used standard PG&E trench specifications, 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, without pressure testing. However, this is not a geotechnical consideration and is the responsibility of the drainage contractor.

Retaining walls 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
- 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
- Drains installed in accordance with §1101.12 of the CPC
- Bentonite seals at drainpipe transition to solid pipe
- Proper installation of the drainage panels
- Outfall details and location

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 Observations

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 Observation Points

- Map excavations in progress to identify and record rock/soil conditions.
- Observe and accept pier drilling and final depth and conditions of all pier holes. *We must be on site at the start of drilling the first hole.* We will perform special inspections in accordance with the CRC or, unless otherwise required by the building official, CBC Chapter 2-Definitions, "Special Inspections, Continuous".
- Accept final footing grade prior to placement of reinforcing steel.
- Accept subdrainage prior to backfilling with drainage rock.
- Accept drainage discharge location.
- Observe tieback placement and proof testing

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 Karin and James Blom and their design professionals for the pool and landscape improvements. 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

Reviewed by:

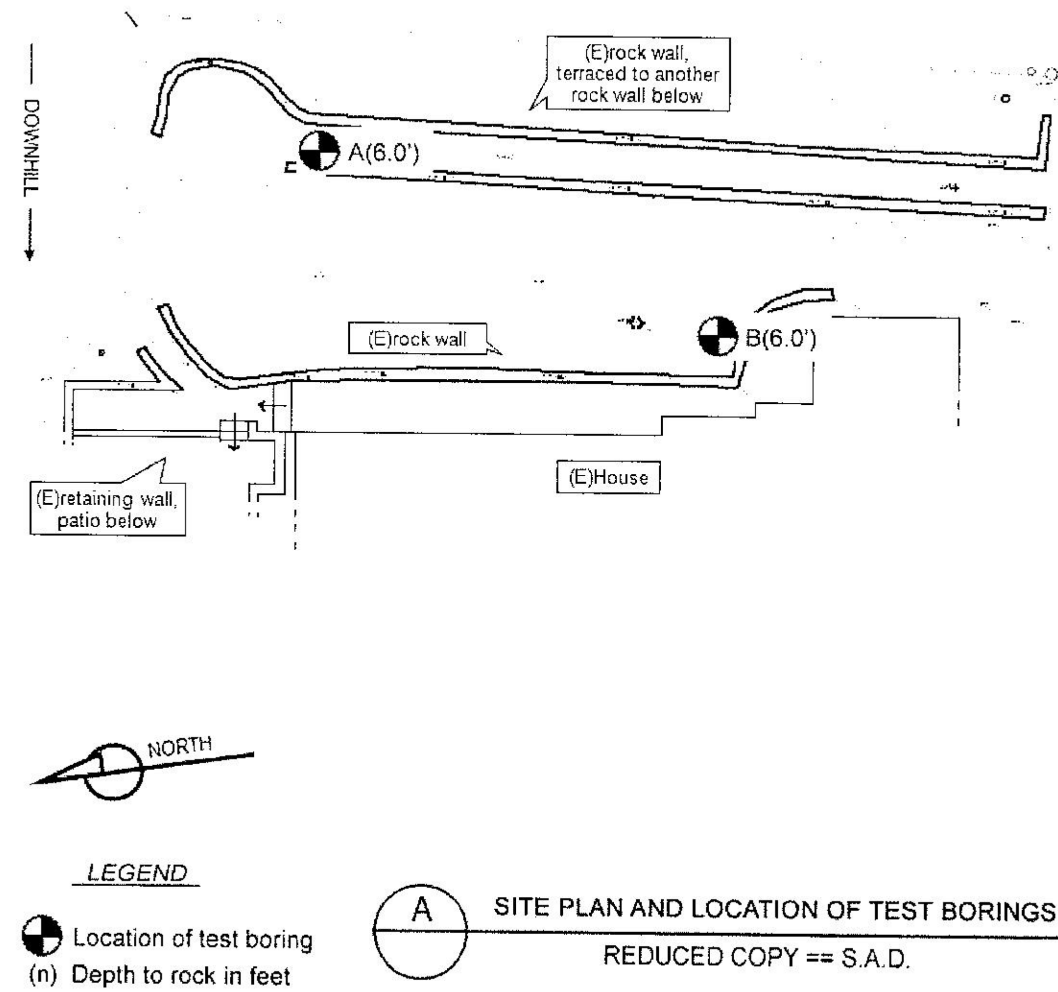
E Vincent Howes
Geotechnical Engineer
GE #965 exp. 31 Mar 22

REGISTERED PROFESSIONAL ENGINEER
E. VINCENT HOWES
No. 965
GEOTECHNICAL
STATE OF CALIFORNIA

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

References: General: 2019 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
Terzaghi and Peck 1967 *Soil Mechanics in Engineeering Practice* 2nd ed, Wile and Sons, NY
Teng, W.C. 1962 *Foundation Design*, Prentice-Hall, Englewood Clifs, N.J.



A SITE PLAN AND LOCATION OF TEST BORINGS

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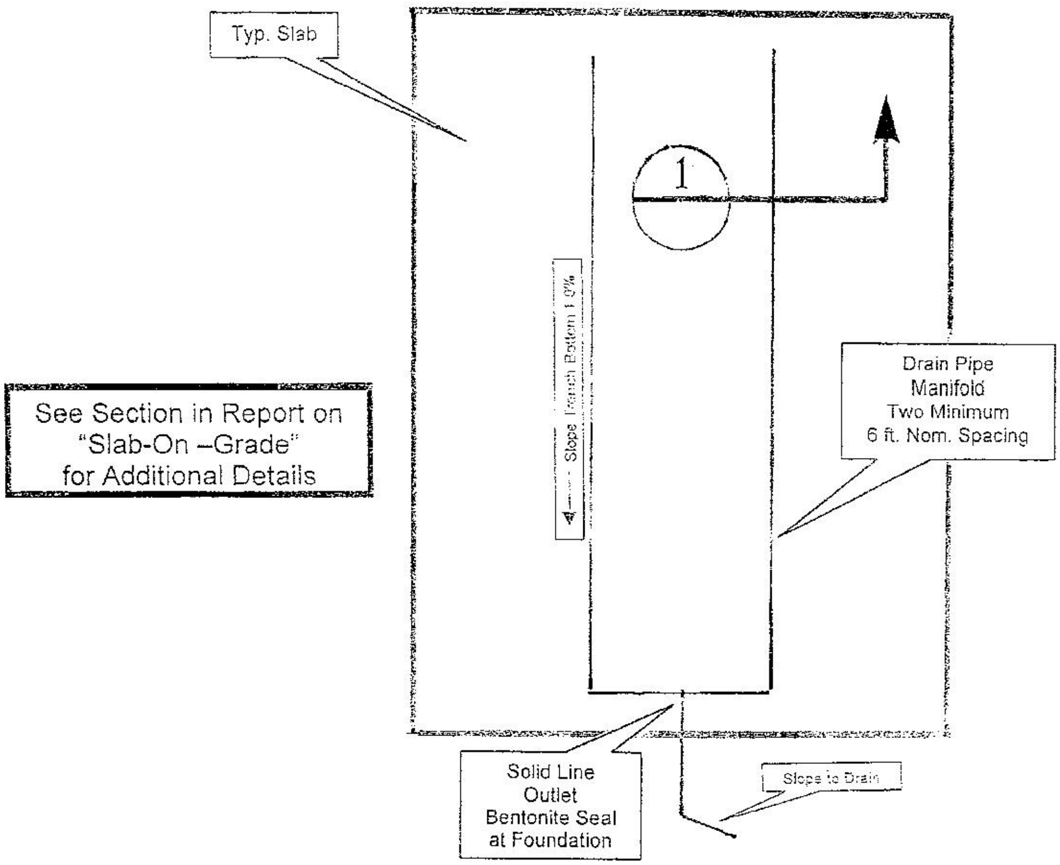
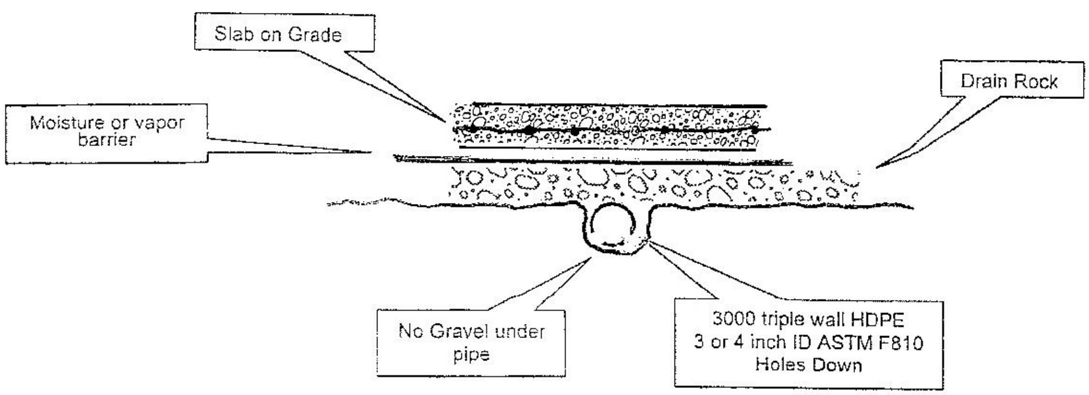


PLAN VIEW

NO SCALE



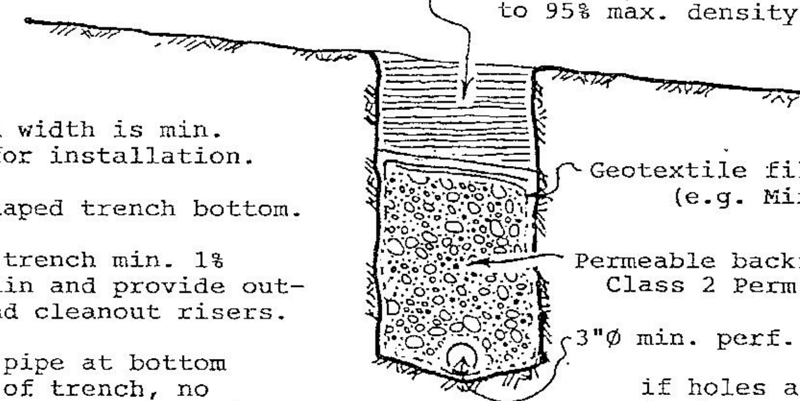
Typ. Section Thru Drain Line

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).

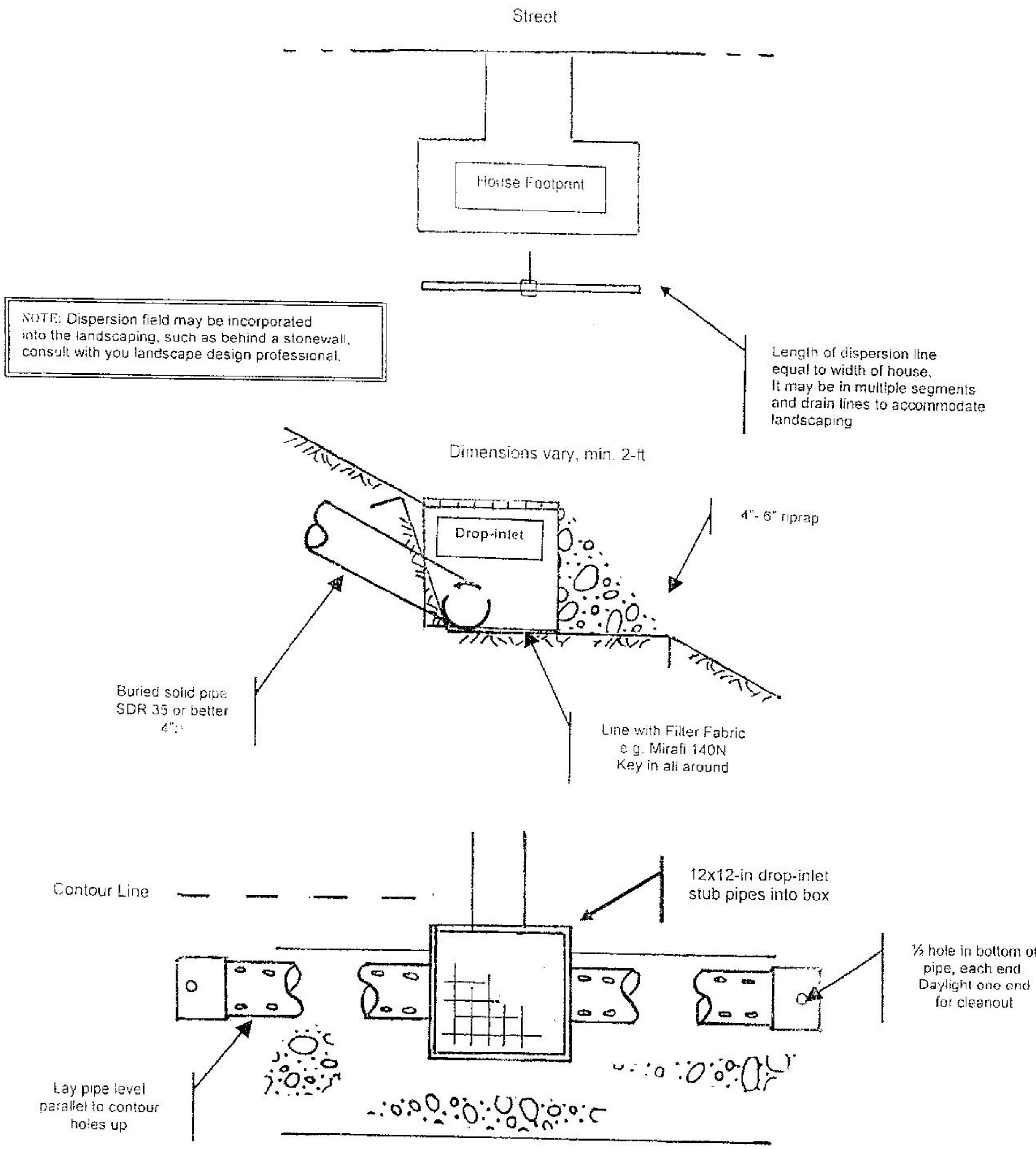
Permeable backfill (e.g. Caltrans Class 2 Perm.) Vibrate into place.

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.

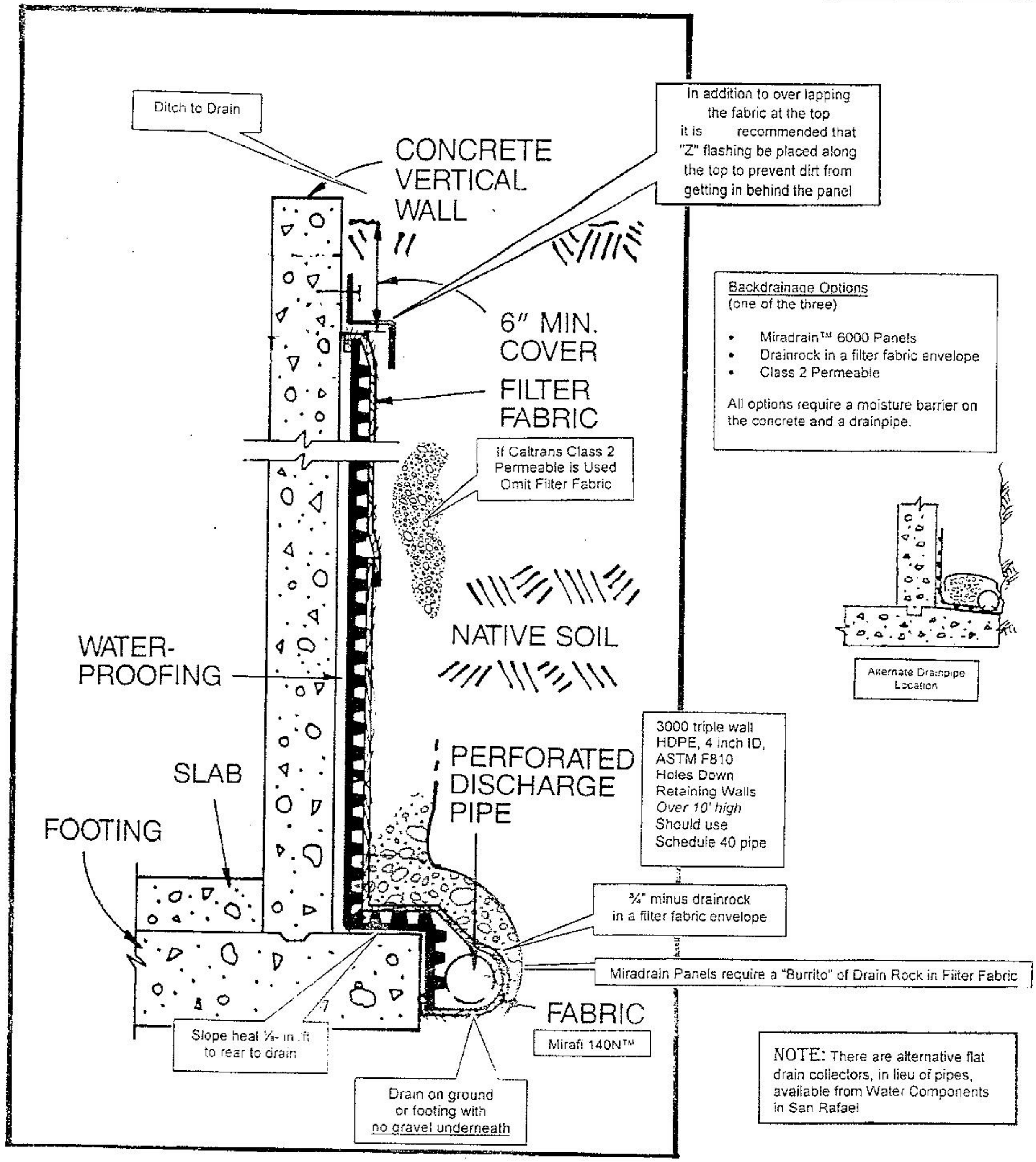
NOTE: We recommend rigid drainpipe 3000 triple wall HDPE, 3 or 4 inch ID, ASTM F810.

TYPICAL DRAIN DETAILS



SKETCH-TYPICAL DISPERSION FIELD DETAILS

NO SCALE



TYPICAL RETAINING WALL DRAINAGE DETAILS

PROJECT: 31 Lincoln Park	**BORING: A**
ENGINEER: E. V. Howes	**LOGGED BY: J. Gillis**
JOB # : 2110048	**DATE: 9 November 2021**

PLASTICITY INDEX (PI)	LIQUID LIMIT	SAMPLE TYPE	(N) Blows Per foot	DEPTH (feet)	WATER LEVEL	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				0-3.5		COLLUVIUM [Qc] 0.0'-3.5' topsoil landscaping horizon with trace rooting, dark brown silty clayey to clayey [ML-CL] soil. rocky horizon at 3.0' and is relatively uniform for this immediate area. below rocky horizon at 3.5' grades to residual soil		
		SPT	31			RESIDUAL SOIL 3.5'-6.0' reddish brown to grayish brown with depth. apparent colluvium texture as sheared shale matrix is highly weathered and inclusions are less weathered. slightly moist throughout with fine trace rooting at top of horizon		
		SPT	14					
		SPT	36			SHEARED SHALE [fm] 6.0'-7.5' stiff to hard, dark gray friable and weathered sheared shale with trace inclusions, slightly moist and no rooting		Top of rock 6.0' SHEARED SHALE [fm]
						End of Log		
								Ground water was not Encountered in boring

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

PROJECT: 31 Lincoln Park	BORING: B
ENGINEER: E. V. Howes	LOGGED BY: J. Gillis
JOB # : 2110048	DATE: 9 November 2021

PLASTICITY INDEX (PI)	LIQUID LIMIT	SAMPLE TYPE	(N) Blows Per foot	DEPTH (feet)	WATER LEVEL	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				0.0'-4.5'		COLLUVIUM [Qc] 0.0'-4.5' topsoil landscaping horizon with trace rooting, dark brown silty clayey to clayey [ML-CL] soil. rocky horizon absent. grays at 4.5'		
				4.5'-6.0'		RESIDUAL SOIL 4.5'-6.0' reddish brown to grayish brown with depth. apparent colluvium texture as sheared shale matrix is highly weathered and inclusions are less weathered. slightly moist throughout with fine trace rooting at top of horizon		
		SPT	20	6.0'-7.5'		SHEARED SHALE [fm] 6.0'-7.5' stiff to hard, dark gray friable and weathered sheared shale with trace inclusions, slightly moist and no rooting		Top of rock 6.0' SHEARED SHALE [fm]
						End of Log		

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

REMARKS
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.

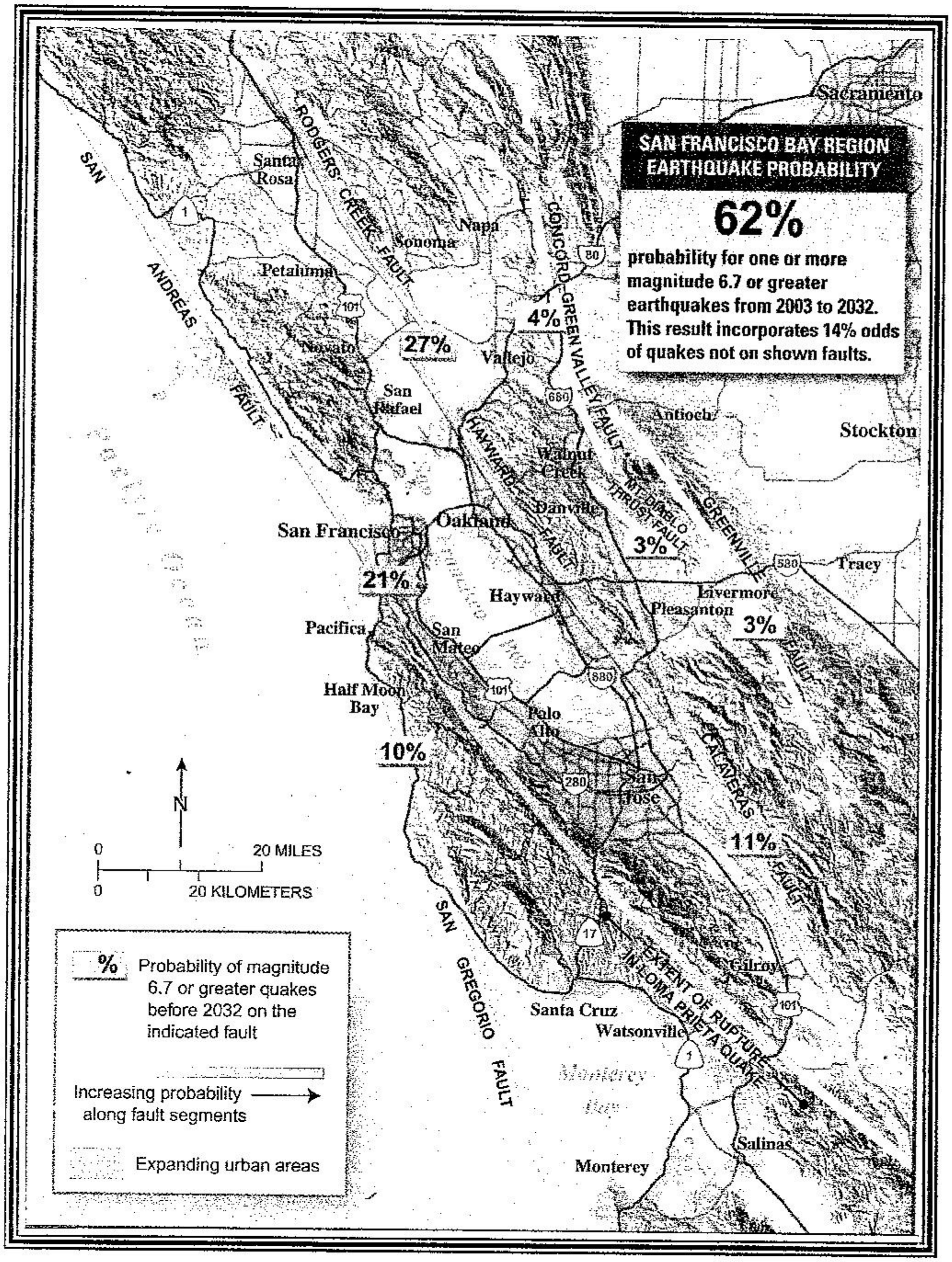
TV indicates in situ measurements made by a Torvane in kilograms per square centimeter.

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) fm = sheared shale bedrock

Topsoil: The fertile, dark-colored organic surface soil

Bedrock- The solid rock that underlies gravel, soil, or other superficial material. The top of the continuous rock deposits of the earth's mantle.



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