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REPORT
GEOTECHNICAL INVESTIGATION

AP 007-201-12
OAK AVENUE
SAN ANSELMO, CA.

7 JUNE 2001
Rev. 1

SOILS

SALEM HOWES ASSOCIATES INC.
GEOTECHNICAL ENGINEERS AND GEOLOGISTS

07 June 2001
Rev. 1

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SUBJECT: Report
Geotechnical Investigation,
Residential Building Sites
535 Oak Avenue, San Anselmo
A.P. 007-201-12

Contents

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Introduction and Summary

This report presents the results of our geotechnical investigation of the proposed residential building sites located above Oak Avenue, San Anselmo on the indicated assessor's parcel. 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 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.

The fieldwork consisted of reconnaissance mapping of exposed geologic features on the site and in the immediate surrounding area and the drilling of 12 test pits/borings. The borings were advanced using a portable hydraulic drill rig with 3-inch flight augers and sampled by Standard Penetration Tests [see "notes"]. Since soil depths are relatively shallow and the soil properties do not provide significant engineering design data, boring logs were omitted and only the depth to the top of rock is shown on the site plan. Fieldwork was conducted during the period of April 1999 through March of 2001. 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.

In summary, sandstone bedrock underlies the site at relatively shallow depths and is suitable for foundation construction utilizing either drilled pier or footing type construction. During our investigation we did not observe any local geologic hazards that would 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 building sites are located on the nose of a side hill ridge that is underlain by sandstone bedrock that has been mapped by others(1) as a Cretaceous member [Ks] of the Franciscan geologic formation. Rock resembling that described in the literature is exposed in the cut slope below the building sites and was encountered in all of the test borings.

This area has been mapped by Rice et al(1) as being on the edge of a block slump landslide; however, our detailed field mapping showed that the boulders are confined to the drainage swales and the side hill ridges are underlain by the sandstone bedrock. This area is at the top of the slump and only surface debris from the ancient slumping remains. The block slumps do not affect the stability of the underlying sandstone bedrock. The block slump area is hummocky and the banks of the drainage reveal it is underlain by cobbles to very large boulders in a silty matrix. The boulder debris consists of basaltic volcanic rocks, primarily greenstone, in contrast to the underlying sandstone bedrock. The top of Bald Hill, directly above the site, is capped by greenstone, which is more resistant to landsliding and erosion than the underlying sandstone. The landslides on the flanks of Bald Hill are the result of the sandstone slopes having been oversteepened and in collapsing, undermining some of the overlying greenstone. In the development area the debris falls are a veneer over the bedrock only filling the drainage swales. The underlying sandstone is exposed on the sidehill ridges between the drainages.

Rock of this formation has been classified (1) as highly stable on natural slopes and fresh sandstone 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 during our investigation. 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, wine cellars and swimming pools 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 or within a current Alquest-Priolo Special Studies Zone. 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 trace. 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, 10 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 Uniform Building Code design purposes this site is in a Seismic Risk Zone 4 with a Soil Profile Type S_B and located 11 kilometers from a type A fault [3].

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 between three and seven feet below the surface. 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 is generally soft and highly fractured and can normally be excavated by common means; however, hard massive areas may be required that could require the use of an excavator mounted "hoe ram". We have worked on five properties in the immediate vicinity of this one and there have not been any problems obtaining the required foundation or drilling depths. Rock slopes over six feet high will require shoring. 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 the night after excavation.

Design Recommendations

All foundations must bear on the unweathered sandstone bedrock by pier or 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 restrain 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. Shoring will be required on rock cuts over five feet.

Drilled Piers

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 on this project, 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 no more than 12 feet on center and connected by tie and grade beams in a grid like configuration. Isolated interior and deck piers should be avoided. Normally end bearing should be neglected (see conditions below).

Piers may be designed by the formula in section 1806.8.2.1, Uniform Building Code 1997 (UCB), 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'. ***Note*** that 'b' is the actual diameter of the pier not a multiple and 'h' is measured from the point of fixity.

Design Parameters

Depth of fixity below top of bedrock surface:	1.5 feet
Soil active pressure:	45 lbs/ft^3
Rock active pressure:	$K_a = 0.0$
Rock passive pressure:	800 $lbs/ft^2/ft$ to calculate S_1
Adhesion:	900 lbs/ft^2

Group action should be considered when the pier spacing in the direction of loading is less than 4 to 6 pier diameters in rock. Group action can be evaluated by reducing the effective coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R[4] as follows:

Pile Spacing in Direction of Loading D = Pile Diameter	Subgrade Reaction Reduction Factor R
4D	1.00
3D	0.80
2D	0.60

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 bottom 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:

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

Bearing may be increased 10 percent of the allowable value for each foot of depth extending below the rock surface. Lateral bearing for transitory loading on end bearing piers may be resisted by a rectangular distribution of 0.80-kips/ft^2 acting on 1.6 diameters.

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.

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 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 Table 18-I-C, section 1809 of the UBC or CBC except that the "Depth Below Undisturbed Ground Surface" in Table 18-I-C 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. In addition, the base of the footing should be below a 30 degree line projected upward from the toe of the closest slope. 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 can be determined by the following formula[4] :

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

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 135 lbs/ft^3 or a passive equivalent fluid pressure of 800 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.

Retaining Walls

All retaining walls should be supported on rock by piers or spread footing type foundations. Landscape structures on engineered fill may use 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 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). Since the backfill never truly provides rigid support that prevents mobilization of the active pressure, this value is appropriate for normal or restrained walls. The portion of any wall supporting a rock back slope may be designed for an equivalent fluid pressure of 35 lbs/ft^3, with a K_a equal to 0.25. Any wall where the backfill is subject to vehicular loads within an area defined by a 30-degree (form vertical) plane projected up from the base of the wall 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:

$$P_s = q' * (\text{height of wall}) * K_a$$

Allowable foundation bearing and lateral resistance to sliding should be obtained from the formulae in the respective sections on pier or footing foundations. When short rigid drilled piers are used in lieu of a keyway they may be designed as follows:

Passive resistance at ⅔ pier depth (D in ft.) acting on 1.6 pier diameters:

$$P_P = ½ * \gamma * D^2 * \tan^2 (45 + Ø/2) = \text{lbs/lft} * 1.66 * \text{pier diameter} = \text{lbs}$$

Properties
γ = 140 lbs/ft^3
Ø = 45°

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.

Piers for 'garden' type walls founded in the stiff soil may be designed using the criteria in section 1806.8.2.1 of the UBC, with an allowable lateral bearing pressure of 400 lbs/ft^2/ft of depth, or Marin County Standard Type A, B or C may be used (5).

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**. 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.

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 elevation. All waterproofing materials must be installed in strict compliance with the manufacturer's specifications. 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.

Tiebacks

The capacity of tiebacks should be determined by the methods in Table 1, Capacity of Anchor Rods in Fractured Rock[(4)]. 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.

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.

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 150 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 shown of Drawing B.

Reinforced Shotcrete

A minimum of six-inches of shotcrete (gunnite) should be applied to completely cover the reinforcing wire fabric. The shotcrete should extend over the top of the slope, be formed as a drainage ditch and keyed into the slope. The shotcrete should be applied by a contractor with prior experience and expertise in the application and use of this equipment. The shotcrete mix with additives should be submitted to the Engineer for approval.

Anchors should be one-inch threaded bars intended to for rock bolting, such as Williams Form Engineering Corp. R1H Hollow-core "spin Lock" mechanical rock bolts. The actual design and specification is highly site and application specific and should be designed in conjunction with your structural engineer. Frequently they are placed in a two-inch diameter hole drilled slightly downward (typically 15° from the horizontal), normally six–feet deep (the specific depth will be determined by our geologist when the excavation is exposed) that is backfilled with a 5000 psi sand-cement grout with expansive additives. The anchors should be installed a minimum of six-feet on center in a staggered pattern

Drainage behind shotcrete walls, that are not adjacent to living quarters, should consist of 12-inch wide Miradrain® HC strips placed vertically on the slope face at a maximum spacing of 3-feet. The strips should be connected to a solid drain line with Miradrain® "end connectors". The drainpipe should be compatible with the end connectors and equivalent to Schedule 40 in strength. The strips should extend to 1-foot of the top of the wall. Clean-outs should be provided for the drainline.

Call Mike at Water Components in San Rafael (451-1780) for product availability and installation information.

Wire fabric should be chain link fence, which conforms to the specifications in the Caltrans Standard Specifications, section 80.401 (1988). Sections of the fabric should be lapped only at a row or column of anchors and the overlapping sections should extend six inches each way past the anchor. The fabric should extend over the top of the slope past the drainage ditch.

Typical reinforced shotcrete details are shown on Drawing C

Shoring

For shoring only (soil 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 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.

Typical shoring details are on Drawing D.

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. Where migration of moisture vapor would be undesirable (e.g. under living spaces) a 10-mil impermeable membrane moisture vapor barrier should be provided. The membrane should be protected during construction from puncture. This is typically accomplished by placing a two-inch sand layer on the moisture barrier. The sand should be thoroughly dampened prior to placing concrete. *Drains and outlets should be provided from the slab drain rock.* (See Drawing E 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.

Geotechnical Drainage Considerations

The site should be graded to provide positive drainage away from the foundations. 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. A bentonite seal should be placed at the transition point between drainpipes and solid pipes.

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. 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 minimum of six inches below the adjacent interior grade and constructed in accordance with the attached Typical Drainage Details. All drainpipe 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. 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 discharged into natural drainages where water will have no detrimental downslope effects, or preferably into the municipal storm water system. The discharge point of all drainage systems should end in a 'T' and be protected by energy dissipating riprap. Typical outfall 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.

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.*

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. The ground should not be disturbed outside the immediate construction area. Prevention of erosion is emphasized over containment of silt. ***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. Periodic land maintenance should be performed to clean and maintain all drains and repair any sloughing or erosion before it becomes a major problem.

Before submitting the project drawings to us for review the architect and structural engineer should be sure the following 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
- No gravel under any drain pipe
- Upslope exterior foundation drains
- Bentonite seals at drainpipe transition to solid pipe
- Proper installation of the drainage panels.
- A reference to this report on drainage and pipe specifications

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. Otherwise, if directed by the Owner, these inspections will be performed on an "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.
- Observe drilling and observation of rock bolts/dowels placement.
- Observe tieback placement and proof loading, including lift off measurement.
- Observe placement of drainage panels and wire mesh prior to application of shotcrete.
- 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.
- 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 John Ducharme 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 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 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 investigate 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.
Geotechnical Engineers

Reviewed by:

E Vincent Howes
Geotechnical Engineer
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REGISTERED PROFESSIONAL ENGINEER
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Attachments: Drawing A, Site Plan and Location of Test Borings
Drawing B, Typical Tieback Installation
Drawing C, Typical Reinforced Shotcrete Application
Drawing D, Typical Shoring Installation
Drawing E, Typical Under-slab Drains
Typical Drain Detail
Typical Outfall Details
Table 1, Capacity of Anchor Rods in Fractured Rock
Plate 1, San Francisco Bay Region Earthquake Probabilities

References:

[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.

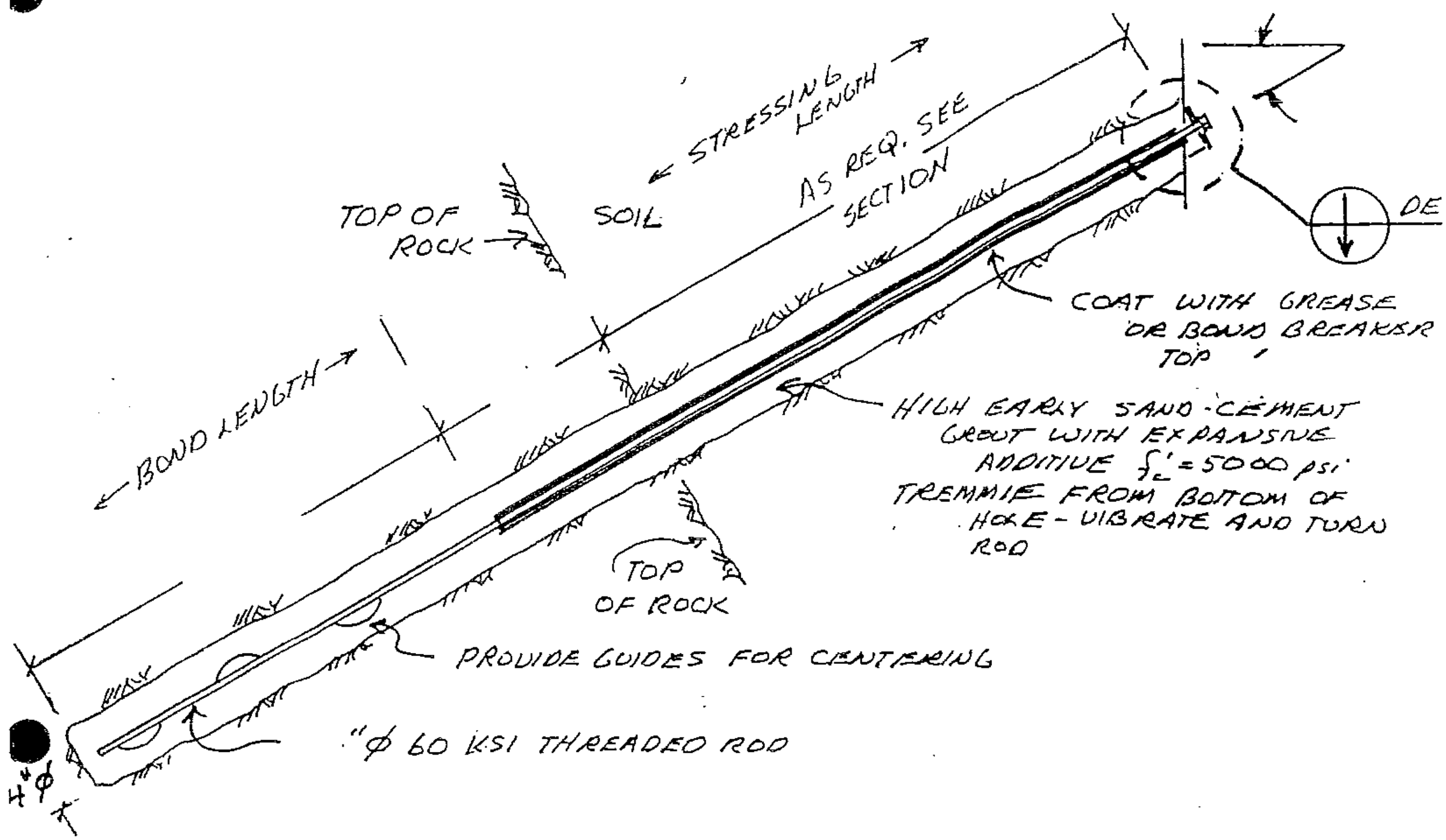
[2] U.S. Geological Survey, Probabilities of Large Earthquakes in the San Francisco Bay Region, 2000 to 2030, Open-File Report 99-517, 1999

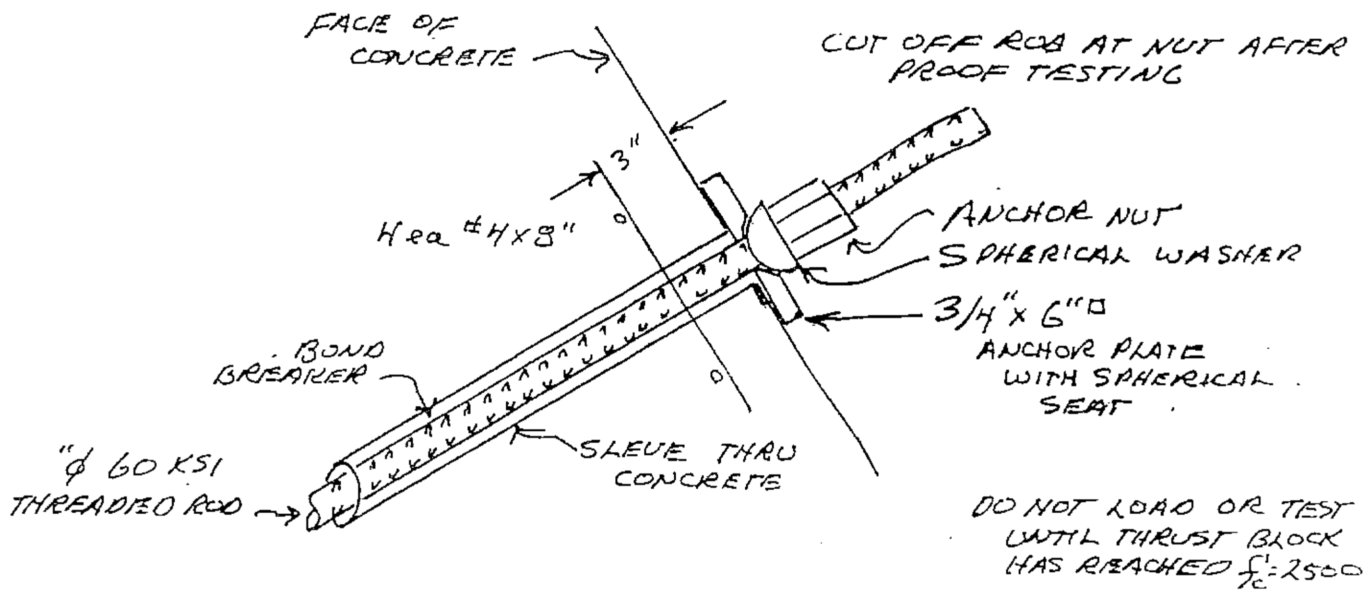
[3] California Department of Conservation, Division of Mines and Geology, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, February 1988, International conference of Building Officials.

[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

Aerial Photos, Pacific Aerial Surveys, Oakland
AV-21 40-03-23,24 5-03-82 1:12000
MRN AV 9549 122 63,64 5-03-00 1:12000
5-89 GS-CP 9-6-46 1:23600 AV-9-2-1 9-6-46

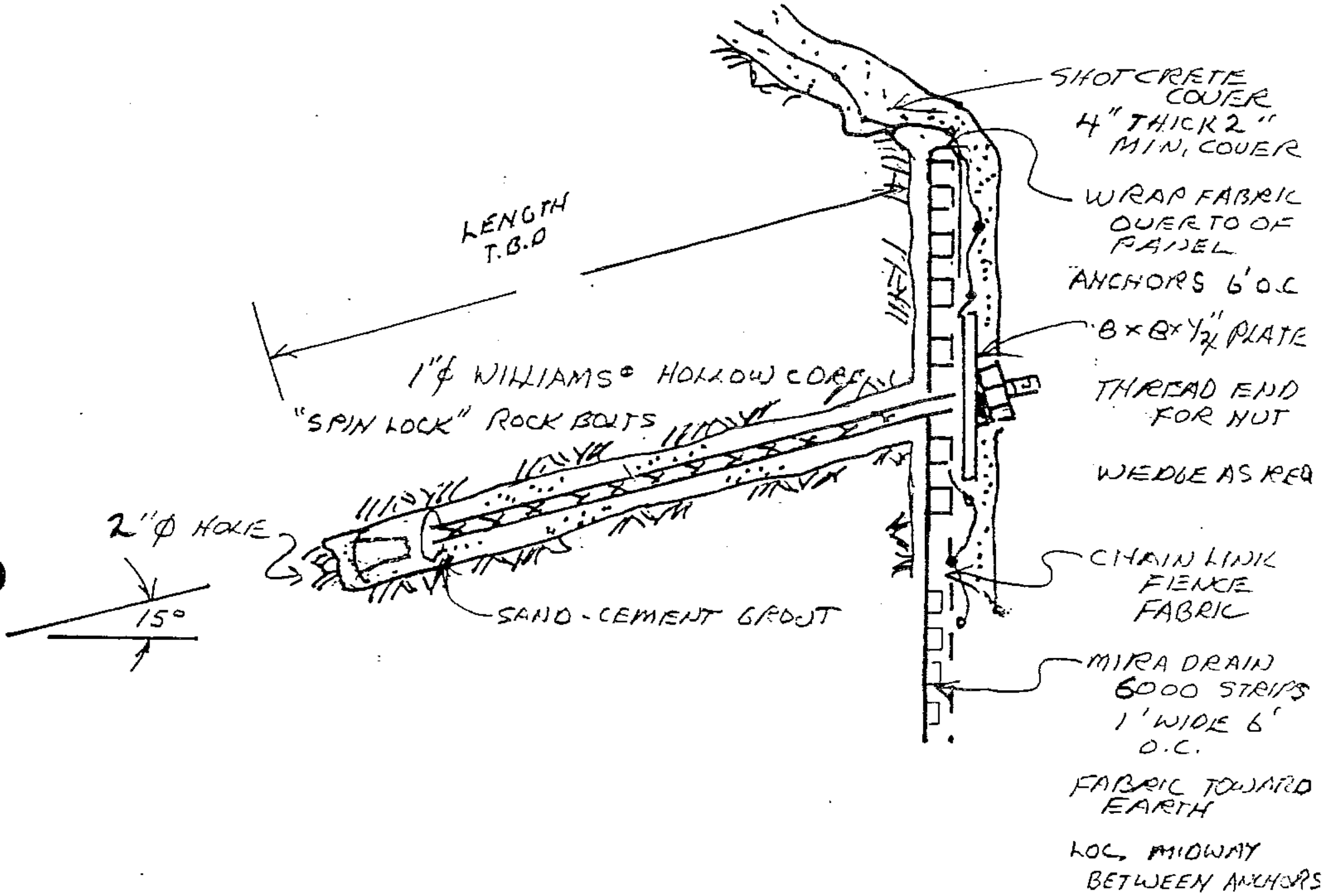




B

TYPICAL TIEBACK INSTALLATION

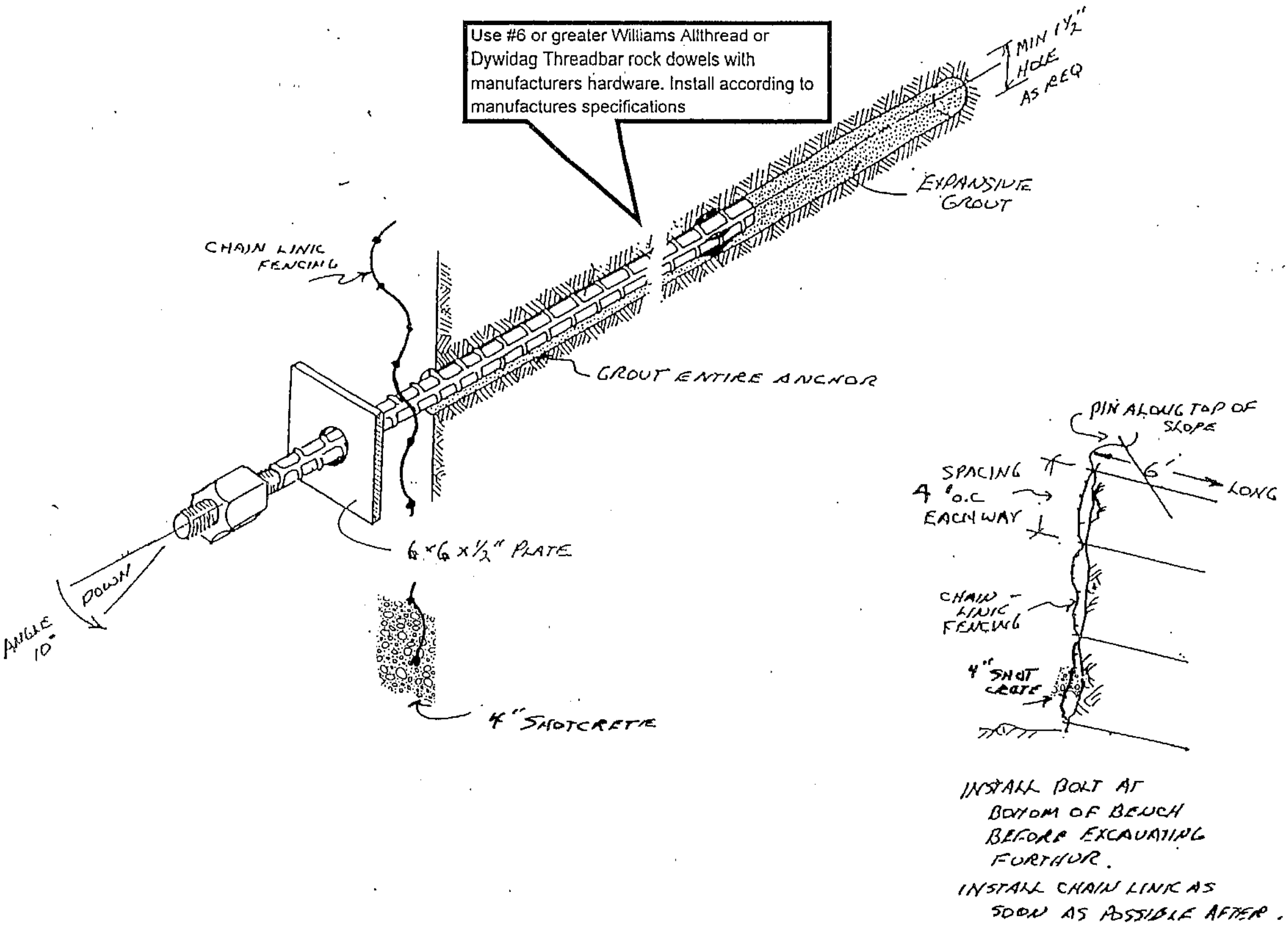
NO SCALE



C

TYPICAL REINFORCED SHOTCRETE APPLICATION

NO SCALE

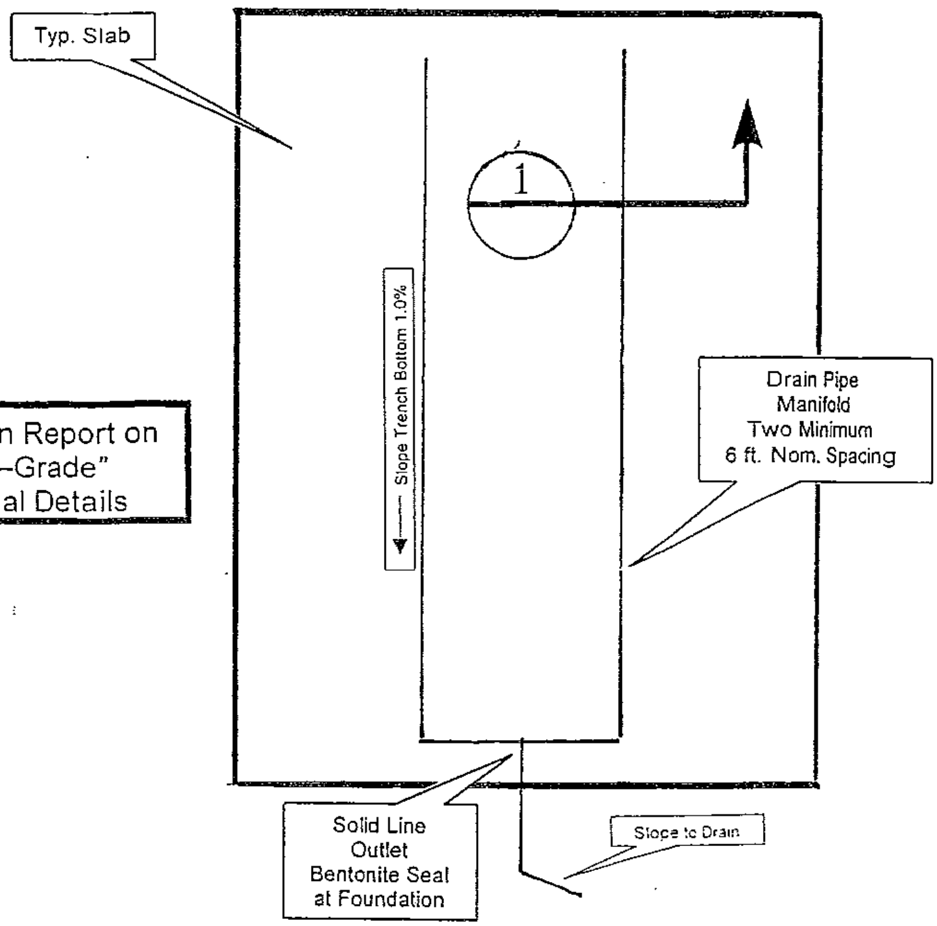


D

TYPICAL SHORING INSTALLATION

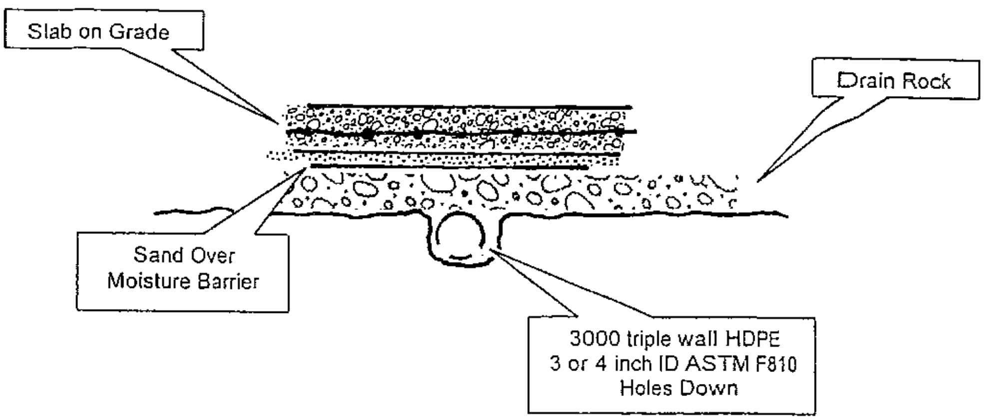
NO SCALE

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



PLAN VIEW

NO SCALE



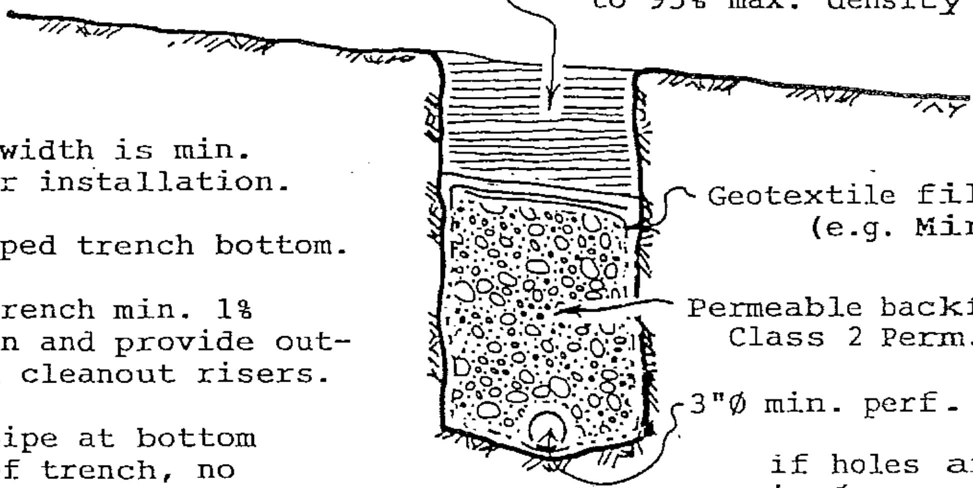
1 Typ. Section Thru Drain Line

No Scale

E **TYPICAL UNDER-SLAB 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).

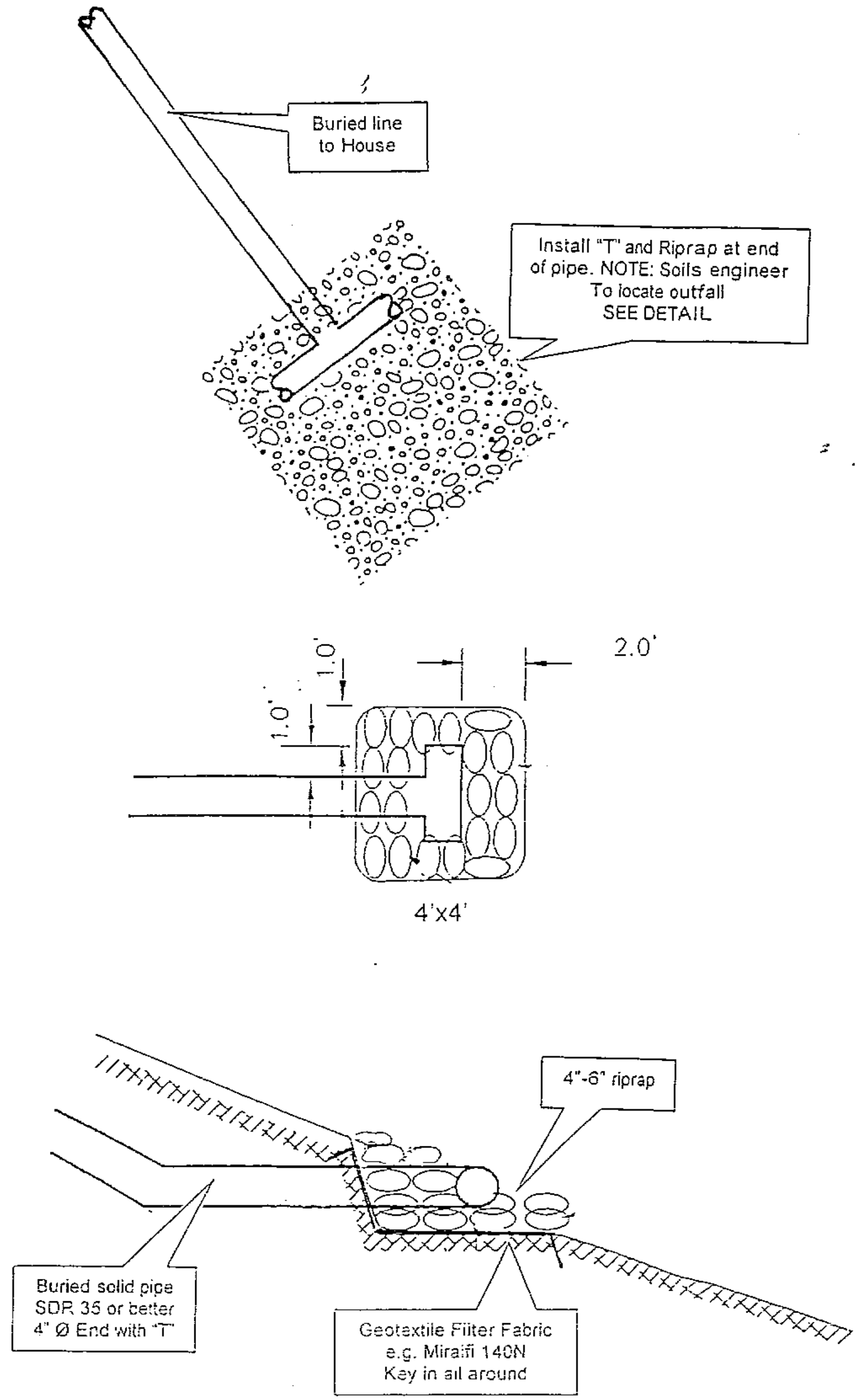
Permeable backfill (e.g. Caltrans Class 2 Perm.) Vibrate into place.

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.

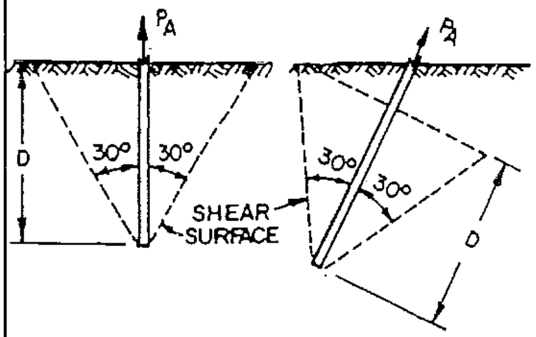
NOTE: We recommend rigid drainpipe 3000 triple wall HDPE, 3 or 4 inch ID, ASTM F810.

TYPICAL DRAIN DETAILS



OUTFALL DETAILS

NO SCALE



SINGLE BAR ANCHORAGES

P_A = ALLOWABLE ANCHOR PULL

D = EMBEDMENT DEPTH, MEASURED AS SHOWN

C_{all} = ALLOWABLE ROCK SHEAR STRESS

f_s = ALLOWABLE BAR STRESS, $0.66 f_y$

b_{rqd} = BOND STRESS ON BAR PERIMETER REQUIRED TO DEVELOP C_{all}

A = BAR CROSS-SECTION AREA

$P_A = (2.1) D^2 (C_{all})$ AND $P_A = A f_s$

$$b_{rqd} = \frac{P_A}{\text{BAR PERIMETER} \times D}$$

TESTS INDICATE THAT FOR BAR IN ORDINARY FRACTURED ROCK NEAR THE SURFACE:
MINIMUM D(FT) = $(1.25)\sqrt{P_A}$ (KIPS)
AT THIS DEPTH C_{all} = 0.3 KSF AND SHOULD NOT BE TAKEN GREATER THAN THIS VALUE WITHOUT PULLOUT TESTS
SPACING OF BARS IN PLAN SHOULD EXCEED 1.2D

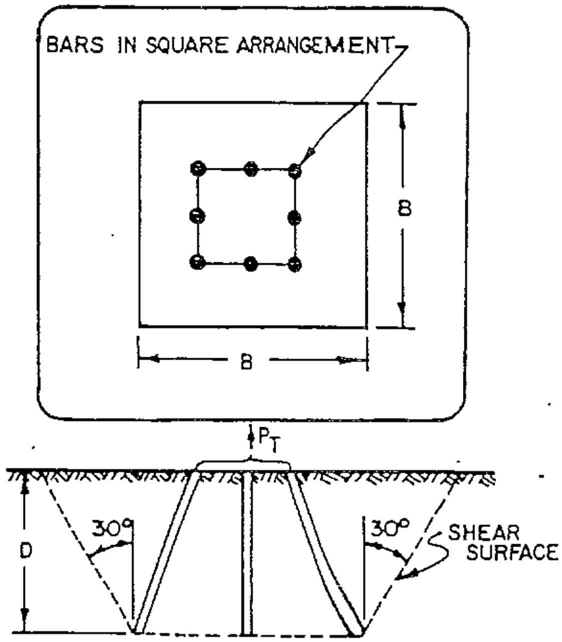
EXAMPLE:
GIVEN: P_A = 20K FOR 1 IN. SQUARE BAR
MINIMUM D = $1.25\sqrt{20}$ = 5.6 FT.
BAR SPACING = 1.2 (5.6) = 6.7 FT.

$$b_{rqd} = \frac{20{,}000}{4(5.6)(12)} = 74 \text{ PSI}$$

Not to exceed 100 psi.

(*) Minimum depth for any application is 6 feet, as measured above.

PLAN



SECTION

BAR GROUP ANCHORAGE

P_T = ALLOWABLE ANCHOR PULL FOR GROUP OF BARS.

N = NUMBER OF BARS IN SQUARE ARRANGEMENT

$P_T = 4.6D(B + 0.58D)\, C_{all}$ AND

$P_T = NA\, f_s$

$$b_{rqd} = \frac{P_T}{\text{BAR PERIMETER} \times ND}$$

TESTS INDICATE THAT FOR BAR GROUP IN ORDINARY FRACTURED ROCK NEAR THE SURFACE:
MINIMUM D(FT)

$$D = \frac{-4.6\, B\, C_{all} + \sqrt{21.2\, B^2 (C_{all})^2 + 10.7\, C_{all} \times NAf_s}}{5.34\, C_{all}}$$

AT THIS DEPTH C_{all} = 0.3 KSF AND SHOULD NOT BE TAKEN GREATER THAN THIS VALUE WITHOUT PULLOUT TESTS

EXAMPLE:
GIVEN P_T = 80K, USE 4 - 1 IN SQUARE BARS
B = 4.5 FT f_s = 20 KSI
MIN. D: WITHOUT TESTS:

$$D = \frac{-4.6 \times 4.5 \times 0.3 + \sqrt{21.2 \times 4.5^2 \times 0.3^2 + 10.7 \times 0.3 \times 4 \times 1 \times 20.0}}{5.34 \times 0.3}$$

= 6.9 FT

$$b_{rqd} = \frac{80{,}000}{(4)(4)(6.9)(12)} = 60 \text{ PSI}$$

Capacity of Anchor Rods in Fractured Rock

Table 1

From: Department of the Navy, Naval Facilities Engineering Command
Soil Mechanics, Design Manual 7.2,
(NAVFAC DM-7) May 1982, Pg 170 Figure 16.

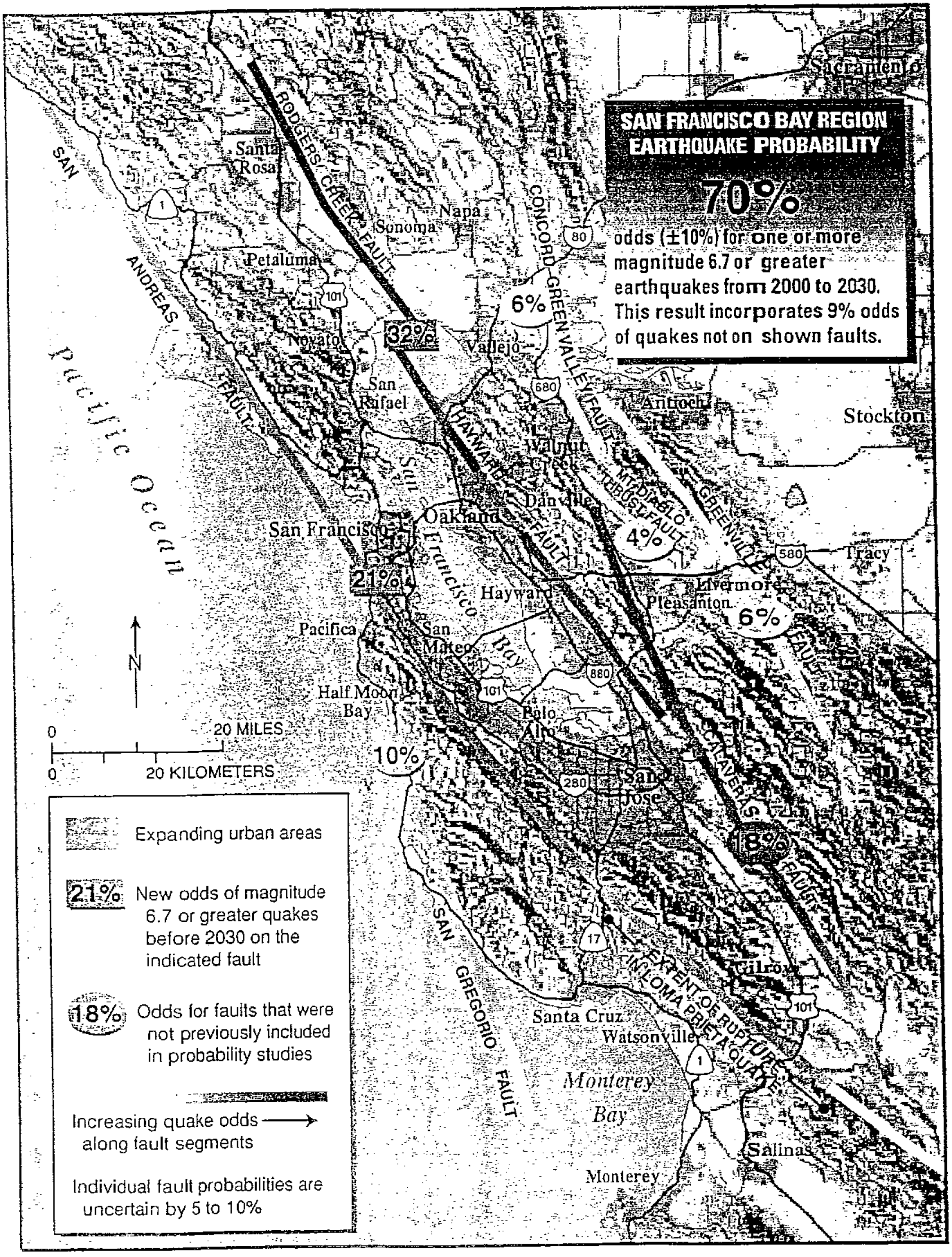


Plate 1, San Francisco Bay Region Earthquake Probabilities

From: U.S. Geological Survey Probabilities of Large Earthquakes in the San Francisco Bay Region, 2000 to 2030, Open-File Report 99-517, 1999