BYER GEOTECHNICAL, INC.

December 8, 2017
BG 22747

Camille and Elena Neagu
1544 Garden Street
Glendale, California 91201

Subject

Transmittal of Geologic and Soils Engineering Exploration
Proposed Lot Line Adjustments and Two Residences
Assessor's Parcel Nos. 5649-021-025 and 5649-008-005, -006, -012, and -013
601 Bohlig Road
Glendale, California

Gentlepersons:

Byer Geotechnical has completed our report dated December 8, 2017, which describes the geologic and soils engineering conditions with respect to the proposed project. The reviewing agency for this document is City of Glendale. Copies of the report have been distributed as follows:

(3) Addressee (E-mail and Pick Up)
(1) Addressee (E-mail and Mail)

It is our understanding that you or your representative will file the report with the City of Glendale. Please review the report carefully prior to submittal to the governmental agency. Questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

Giuseppe Cugno
Senior Project Geologist

BYER GEOTECHNICAL, INC.

GEOLOGIC AND SOILS ENGINEERING EXPLORATION

PROPOSED LOT LINE ADJUSTMENTS AND TWO RESIDENCES

ASSESSOR'S PARCEL NOS. 5649-021-025 AND 5649-008-005, -006, -012, AND -013

601 BOHLIG ROAD

GLENDALE, CALIFORNIA

FOR CAMILLE AND ELENA NEAGU

BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 22747

DECEMBER 8, 2017

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INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geologic and soils engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution, engineering properties, relative stability, and geologic structure of the earth materials underlying the site with respect to the proposed lot line adjustments and construction of two single-family residences. This report is intended to assist in the design and completion of the proposed project and to reduce geotechnical risks that may affect the project. The professional opinions and advice presented in this report are based upon commonly accepted exploration standards and are subject to the AGREEMENT with TERMS AND CONDITIONS, and the GENERAL CONDITIONS AND NOTICE section of this report. No warranty is expressed or implied by the issuing of this report.

PROPOSED PROJECT

The scope of the proposed project was determined from review of preliminary plans prepared by Malekian and Associates, Inc. The existing residence and appurtenant structures will be demolished. The project consists of lot line adjustments to create two buildable parcels and to construct a residence and a detached buried garage on each of the lots. Grading will consist of excavations to create the desired residence and garage floor grades. Retaining walls up to a maximum of 25 feet high are planned to support excavations for the proposed structures, including the subterranean garages, which are planned to be constructed as cut-and-cover (see Sections A, C, and E). The detached garage for the western lot will be accessed from Melwood Drive. Vehicular access to the garage on the eastern lot (Parcel 1) will be from Bohlig Road.

EXPLORATION

The scope of the field exploration was determined from consultation with Alen Malekian. The preliminary plans prepared by Malekian and Associates, Inc., were a guide to our work on this project. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the Geologic Map and cross sections. The scope of this exploration did not include an assessment of general site environmental conditions for the presence of contaminants in the earth materials and groundwater. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Exploration was conducted on October 10, 2017, with the aid of hand labor. It included excavating nine test pits to depths of ¾ foot to 9 feet. Samples of the earth materials were obtained and delivered to our soils engineering laboratory for testing and analysis.

Office tasks included laboratory testing of selected soil samples, review of published maps and photos for the area, review of our files, preparation of cross sections, preparation of the Geologic

Map, slope stability calculations, engineering analysis, and preparation of this report. Earth materials exposed in the test pits are described on the enclosed Log of Test Pits. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project, surface geologic conditions, and the locations of the test pits are shown on the Geologic Map. Subsurface distribution of the earth materials, projected geologic structure, and the proposed project are shown on Sections A through E. Section B forms the basis for the slope stability calculations.

SITE DESCRIPTION

The subject property consists of a partially-graded hillside property in the southern foothills of the Verdugo Mountains in the city of Glendale, California (34.1687° N Latitude, 118.2472° W Longitude). It is located on the crest and west flank of a southwest-northeast-trending ridge on the north side of Bohlig Road cul-de-sac, approximately one-eighth of a mile north of East Mountain Street and one-half of a mile east of North Brand Boulevard. The site is developed with a two-story single-family residence and garage. The surrounding area has been developed with single-family residences. Vehicular access to the residence and garage is provided by a driveway that ascends approximately 25 feet north of the Bohlig Road cul-de-sac.

Past grading on the site has consisted of cutting into the ridge-flank during the improvement of Bohlig Road and placement of minor fill on the downhill side of the road. Grading associated with the site development consisted in cutting into the ridge-flank, upslope of Bohlig Road, to create the access driveway and grading along the ridge-crest to create the existing garage and residence pads. Excavation material was partially placed over the west-facing slope that descends to Melwood Road to the west. Slopes descend 55 to 63 feet below (west of) the existing residence pad to the offsite driveway and to Melwood Drive to the west at gradients ranging from 2:1 to as steep as 1¼:1. Cut slopes as steep as 1:1 ascend approximately 25 feet above and northeast of the graded pad to the adjacent properties to the northeast. Cut slopes as steep as 1:1 descend 15 to 20 feet below the driveway and graded pad to Bohlig Road.

Vegetation on the site consists of shrubs along Bohlig Road. Mature oak trees are scattered on the descending slope west of the residence and surrounding property lines on the north and east sides.

Pad drainage is by sheetflow runoff down the contours of the land to Bohlig Road. Roof drainage freefalls to the pad. Drainage along the western portion of the lot sheetflows down the contours towards the adjacent driveway and Melwood Drive.

GROUNDWATER

Groundwater was not encountered in the test pits which were excavated to a maximum depth of nine feet. Seasonal fluctuations in groundwater levels occur due to variations in climate, irrigation, development, and other factors not evident at the time of the exploration. Groundwater levels may also differ across the site. Groundwater can saturate earth materials causing subsidence or instability of slopes.

EARTH MATERIALS

Fill

Fill, associated with previous site grading and development, blankets the western portion of the graded level pad to a maximum observed depth of seven feet in Test Pits 5 and 6. Greater depths of fill may occur. The fill consists of silty sand and sandy silt that is brown to medium brown, dry to slightly moist and loose to medium dense.

Soil

Natural residual soil blankets the site and was encountered in Test Pits 1 through 7. The soil consists of silty sand that is brown, slightly moist, slightly to medium dense. The soil layer observed varies from less than one foot to two feet thick.

Bedrock

Bedrock underlying the site and encountered in the test pits consists of granitic rock mapped as quartz-diorite by Byer (1968) and gneissoid quartz diorite by Thomas W. Dibblee, Jr. (1989). For ease of description, the bedrock underlying the site will be referred to as "granite." The bedrock is also exposed in cut slopes northeast of the building pad and along the road and driveway cut slopes. The bedrock is tan to light yellowish-brown, massive, slightly to moderately weathered, moderately hard to hard, and slightly fractured.

GEOLOGIC STRUCTURE

The bedrock described above is common to this area of Glendale. The bedrock is generally massive with steeply-dipping and discontinuous joint and foliation planes. The generally-massive nature of the bedrock is favorable for the gross stability of the site and proposed project.

GENERAL SEISMIC CONSIDERATIONS

The subject property is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Geological Survey (CGS), private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies now require earthquakeresistant structures. The purpose of the code seismic-design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

The *Fault Activity Map of California* (Jennings and Bryant, 2010) classifies fault activity based on the most recent age of fault movement, and distinguishes between "historic faults" (displacement within the last 200 years); "Holocene faults" (displacement within the last 11,700 years); "Late Quaternary faults" (surface rupture within the last 700,000 years); "Quaternary faults" (displacement within the last 1.6 million years); and "pre-Quaternary faults" (no displacement within the last 1.6 million years).

The mapped fault closest to the project site is the Verdugo Fault, a Holocene to Late Quaternary fault, located approximately 1,700 feet southwest of the site according to Dibblee and approximately 1,550 feet southwest of the site according to Jennings and Bryant, 2010.

In addition to the faults shown on the *Fault Activity Map of California*, blind-thrust faults, which, by definition, do not reach the surface, are known to underlie the greater Los Angeles area. For example, the Northridge Earthquake of January 17, 1994, magnitude 6.7, and the Whittier Earthquake of October 1, 1987, magnitude 5.9, occurred on previously-unrecognized blind-thrust faults. The Elysian Park blind thrust, which is generally considered responsible for generation of the anticline that forms the hills in the Silver Lake - Elysian Park area, is located below the Silver Lake area. In general, the seismic coefficients provided below incorporate the effects of all known seismogenic sources, including blind thrusts.

The following table lists the applicable seismic coefficients for the project based on the California Building Code:

SEISMIC COEFFICIENTS (2016 California Building Code - Based on ASCE Standard 7-10)		
Latitude = 34.1687° N Longitude = 118.2472° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 20.3-1, ASCE Standard 7-10	Bedrock - C	
Mapped Spectral Accelerations from Figures 1613.3.1 (1) and 1613.3.1 (2) and USGS	S_s = 2.903 (g)	S_1 = 1.003 (g)
Site Coefficients from Tables 1613.3.3 (1) and 1613.3.3 (2) and USGS	F_A = 1.0	F_V = 1.3
Maximum Considered Spectral Response Accelerations from Equations 16-37 and 16-38, 2013 CBC	S_{MS} = 2.903 (g)	S_{M1} = 1.304 (g)
Design Spectral Response Accelerations from Equations 16-39 and 16-40, 2013 CBC	S_{DS} = 1.935 (g)	S_{D1} = 0.869 (g)
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	PGA_M = 1.081 (g)	

Reference: U.S. Geological Survey, **Geologic Hazards Science Center, U. S. Seismic Design Maps,** http://earthquake.usgs.gov/designmaps/us/application.php

The Occupancy Category for a residence is II. The mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is greater than 0.75g. Therefore, the project is considered to be in Seismic Design Category E.

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear panels, moment frames, and reinforcement. Additional precautions may be taken, including strapping water heaters and securing furniture to walls and floors. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

The property is more than 18 miles from the shoreline and its elevation varies from 815 to 900 feet above sea level. The risk from a tsunami is nil.

Ground Motion

To determine the ground motion for the project site, a probabilistic seismic deaggregation analysis was performed, using the USGS 2008 Interactive Deaggregation application available online (http://geohazards.usgs.gov/deaggint/2008/) for a 10 percent probability of exceedance in 50 years (475-year return period), and using a shear-wave velocity estimate of 760 meters-per-second. The results are shown on the enclosed "PSH Deaggregation Chart." The analysis indicates a peak ground acceleration (PGA) of 0.59g, a modal earthquake magnitude (M_w) of 6.47, and a modal fault distance of 6.5 kilometers.

Liquefaction

The CGS has not mapped the site within an area where historic occurrence of liquefaction or geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693 (c) would be required. The subject property is underlain by bedrock which is not subject to liquefaction.

SLOPE STABILITY

Gross Stability

The CGS has not designated the property within a state zone requiring seismic landslide investigation per Public Resources Code, Section 2693 (c). Slopes analyzed for stability include an existing approximately 25-foot-high, 1:1 cut slope and a 65-foot-high natural slope. The gross stability of the slopes was analyzed using a computerized version of Bishop's simplified method.

The analysis shows that the existing and proposed slopes will be grossly stable with a factor of safety in excess of 1.5. The calculations use the shear tests of samples believed to be representative of the strength of the bedrock encountered during exploration. The cross section used is the most critical for the slopes analyzed.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, nine test pits, field geologic mapping, research of available records, laboratory testing, engineering analysis, and years of experience performing similar studies on similar sites. It is the finding of Byer Geotechnical, Inc., that construction of the proposed project is feasible from a geologic and soils engineering standpoint, provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material is bedrock. Conventional foundations may be used to support portions of the residence provided the footings are adequately setback from descending slopes and from retaining walls. Deepened foundations are recommended for the southern portions of the residence on the easterly lot (Parcel 1) to ensure embedment below a 1:1 plane from the bottom of the subterranean garage. Soils to be exposed at finished grade will be in the non-expansion range. Geotechnical issues affecting the project include the presence of old uncertified fill and deep excavations.

Shoring, consisting of soldier piles, will be required to support the majority of the temporary excavations for the garages (see Sections B, C, and E) and for portions of the residence with basement where slope trimming is not feasible or desired (see Sections B and C). The remaining portion of the existing road cut should be trimmed back to a gradient no steeper than 1¼:1. Any fill mantling the descending slope should be removed or trimmed to a gradient no steeper than 2:1. The garage roofs should be designed as structural slabs capable of supporting the future backfill (see Sections B, C, and E).

SITE PREPARATION - REMOVALS

The following general grading specifications may be used in preparation of the grading plan and job specifications for the placement of compacted fill over the garage roofs. Byer Geotechnical would appreciate the opportunity of reviewing the plans to ensure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The garage roofs should be provided with adequate waterproofing system designed by the structural engineer to remove any subsurface water and prevent infiltration into the garage. A subdrain system consisting of perforated drain pipes covered with a 12-inch-thick gravel blanket is recommended.

B. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts, moistened as required, and compacted in six-inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

C. The moisture content of the fill should be near the optimum moisture content. When the moisture content of the fill is too wet or dry, the fill shall be moisture conditioned and mixed until the proper moisture is attained.

D. The fill shall be compacted to at least 90 percent of the maximum laboratory dry density for the material used. The maximum dry density shall be determined by ASTM D 1557-12 or equivalent.

E. Field observation and testing shall be performed by the soils engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until 90 percent relative compaction is obtained. A minimum of one compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

Excavation Characteristics

Excavation difficulty is a function of the degree of weathering and amount of fracturing within the bedrock. The bedrock generally becomes harder and more difficult to excavate with increasing depth. Hard, cemented layers are also known to occur at random locations and depths and may be encountered during foundation excavation. Should a hard, cemented layer be encountered, coring or the use of jackhammers may be necessary.

FOUNDATION DESIGN

Spread Footings

Continuous and/or pad footings may be used to support the proposed structures, provided they are founded in bedrock and setback from any descending slopes in accordance with the "Foundation Setback" section of this report. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24-inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Bedrock	12	6,000	0.45	500	6,000

The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

Footings adjacent to retaining walls should be deepened below a 1:1 plane from the bottom of the lower retaining wall, or the footings should be designed as grade beams to bridge from the wall to the 1:1 plane.

All continuous footings should be reinforced with a minimum of four #4 steel bars: two placed near the top and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks, and approved by the geologist prior to placing forms, steel, or concrete.

Deepened Foundations - Friction Piles

Cast-in-place, concrete friction piles are recommended to support the portions of the residences adjacent to the descending slope and future retaining walls to satisfy setback requirements. Piles should be a minimum of 24 inches in diameter and a minimum of 8 feet into bedrock and below a 1:1 plane projected from the bottom of any retaining wall. Piles may be assumed fixed at 4 feet into bedrock and below a 1:1 plane projected from the bottom of any retaining wall. The piles may be designed for a skin friction of 700 pounds-per-square-foot for that portion of pile in contact with the bedrock and below a 1:1 plane projected from the bottom of any retaining wall. The structural engineer may design piles that are deeper or larger in diameter depending on final loads. All piles should be tied in two horizontal directions with grade beams. Grade beams parallel to the slope should be designed to resist an equivalent fluid pressure of 43 pounds-per-cubic-foot. Grade beams supporting future compacted fill should be embedded a minimum of 18 inches into bedrock as measured on the downhill side.

Lateral Design

The existing fill and soil on the site are subject to downhill creep. Pile shafts are subject to lateral loads due to the creep forces. Pile shafts should be designed for a lateral load of 1,000 pounds-per-linear-foot for each foot of shaft exposed to the existing fill and soil.

The friction value is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by passive earth pressure within the bedrock.

Passive earth pressure may be computed as an equivalent fluid having a density of 500 pounds-per-cubic-foot. The maximum allowable earth pressure is 6,000 pounds-per-square-foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½-pile diameters on center may be considered isolated.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A total settlement of one-fourth to one-half of an inch may be anticipated. Differential settlement should not exceed one-fourth of an inch.

Foundation Setback

The California Building Code requires that foundations be a sufficient depth to provide a horizontal setback from a descending slope steeper than 3:1. The required setback is one-third the height of the slope, with a maximum of 40 feet, measured horizontally, from the base of the foundation to the slope face. The required setback for a swimming pool is one-sixth the height of the slope, with a minimum of five feet and a maximum of 20 feet, measured horizontally, from the bottom of the pool to the slope face.

Toe of Slope Clearance

The building code requires a level rear-yard setback, between the toe of an ascending slope steeper than 3:1 and the proposed structure, of one-half the slope height to a maximum 15-foot clearance. For retained slopes, the face of the retaining wall is considered the toe of the slope.

SWIMMING POOL

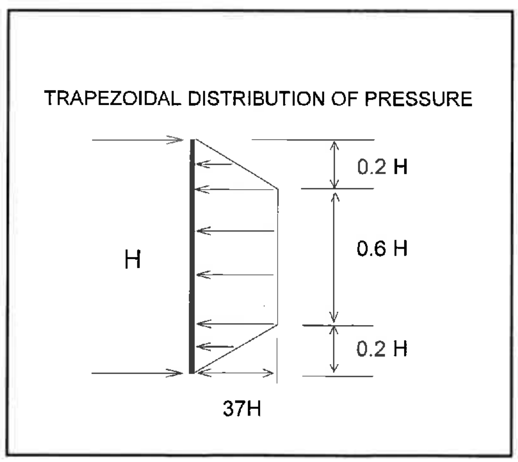
The proposed swimming pool may be constructed using a freestanding design. Pool walls should be designed for an inward pressure of 43 pounds-per-cubic-foot. The pool should derive support entirely from the bedrock. This may require the use of a deepened foundation system for the southern portion adjacent to Bohlig Road.

RETAINING WALLS

General Design

Retaining walls up to 25 feet high, with a level backslope, may be designed for an equivalent fluid pressure of 43 pounds-per-cubic-foot, per the enclosed calculations. Rear-yard retaining walls up to 20 feet high and with a backslope as steep as 1½:1 should be designed for an equivalent fluid pressure of 55 pounds-per-cubic-foot, per the enclosed calculations. Retaining walls with a backslope as steep as 40 degrees (approximately 1¼:1) should be designed for an equivalent of 63 pounds per-cubic-foot. Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel.

Proposed basement walls, which will be restrained, should be designed for an at-rest lateral earth pressure of 37H, where H is the height of the wall. The diagram illustrates the trapezoidal distribution of earth pressure. The design earth pressures assume that the walls are freedraining. Basement walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel.



Seismic Loading

The seismic loading on the proposed retaining walls was calculated using a horizontal pseudo-static seismic coefficient (k_h) equal to one-third PGA_M = 0.36g. The calculations indicate that the recommended static design pressures are sufficient to support seismic loading.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557-12, or equivalent. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾-inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted-fill blanket to the surface. Restrained walls should not be backfilled until the restraining system is in place.

Foundation Design

Retaining wall footings may be sized per the "Deepened Foundations" and "Spread Footings" sections of this report.

Retaining Wall Deflection

It should be noted that non-restrained retaining walls can deflect up to one percent of their height in response to loading. This deflection is normal and results in lateral movement and settlement of the backfill toward the wall. The zone of influence is within a 1:1 plane from the bottom of the wall. Hard surfaces or footings placed on the retaining wall backfill should be designed to avoid the effects of differential settlement from this movement. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

Freeboard

Retaining walls surcharged by a sloping condition should be provided with a minimum of 18 inches of freeboard for slough protection. An open "V" drain should be placed behind the wall so that all upslope flows are directed around the structure to the street.

TEMPORARY EXCAVATIONS

Temporary excavations will be required to construct the proposed residence, garage, and retaining walls. The excavations for the proposed subterranean garages and for the residence will be up to 25 feet in height and will expose thin fill and soil over bedrock. The bedrock is capable of maintaining vertical excavations up to 10 feet, per the enclosed calculations. Where vertical excavations in the bedrock exceed 10 feet in height, the upper portion should be trimmed to 1:1 (45 degrees). Temporary excavations for the proposed residence and garage where slope trimming is not feasible will require the use of shoring consisting of drilled, cast-in-place concrete soldier piles. The piles may be incorporated into the permanent retaining walls.

Temporary Shoring/Soldier Piles

Drilled, cast-in-place concrete soldier piles are recommended as shoring to support the temporary excavations where slope trimming is not feasible or not desired. Soldier piles should be a minimum of 24 inches in diameter and a minimum of eight feet into bedrock below the base of the excavation. Piles may be assumed fixed at three feet into bedrock below the base of the excavation. The piles may be designed for a skin friction of 700 pounds-per-square-foot for that portion of the pile in contact with the bedrock below the base of the excavation. Soldier piles should be spaced a maximum of 10 feet on center. Soldier piles up to 25 feet high with a level backslope may be designed for an equivalent fluid pressure of 30 pounds-per-cubic-foot, per the enclosed calculations. Soldier piles up to 20 feet high with a 1½:1 backslope may be designed for an equivalent fluid pressure of 35 pounds-per-cubic-foot.

Lateral Design - Soldier Piles

The friction values are for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by passive earth pressure within the bedrock.

Passive earth pressure may be computed as an equivalent fluid having a density of 500 pounds-per-cubic-foot. The maximum allowable earth pressure is 6,000 pounds-per-square-foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced at least 2½-pile diameters on center may be considered isolated.

Tieback Anchors

Tieback anchors may be used to resist lateral loads for the temporary shoring piles. Conventional, drilled friction anchors or pressure-grouted anchors may be used. The active wedge adjacent to the shoring is defined by a plane drawn at 33 degrees with the vertical through the bottom of the excavation. The friction anchors should extend at least 15 feet beyond the active wedge or to a greater length if necessary to develop the desired resistance. For design purposes, it is estimated that drilled friction anchors a minimum of 10 feet beyond the active wedge will develop an average friction value of 750 pounds-per-square-foot. Only the frictional resistance developed beyond the active wedge will be effective in resisting lateral loads. If anchors are spaced no closer than six feet, on center, no reduction in the capacity of the anchors is necessary. The anchors may be installed at angles of 25 to 40 degrees from the horizontal. Tieback anchors should be tested during installation in accordance with the specifications of the shoring engineer.

Rakers

Rakers or struts may be used to internally brace the soldier piles. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. For design of temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 6,000 pounds-per-square-foot may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade.

Lagging

Continuous lagging should be anticipated between the soldier piles. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. Lagging should be designed for the recommended earth pressure, but may be limited to a maximum value of 400 pounds-per-square-foot.

Deflection

Some deflection of the shored embankment should be anticipated. Where shoring is planned adjacent to existing structures, it is recommended that lateral deflection not exceed one-half of an inch. For shoring not surcharged by a structure, the allowable deflection is deferred to the structural engineer. If greater deflection occurs during construction, additional bracing or anchors may be necessary to minimize deflection. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

FLOOR SLABS

Floor slabs should be cast over bedrock or approved compacted fill and reinforced with a minimum of #4 bars on 16-inch centers, each way. Slabs that will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be sandwiched between the layers of sand, about two inches each, to prevent punctures and aid in the concrete cure. A low-slump concrete may be used to minimize possible curling of the slab. The concrete should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

It should be noted that cracking of concrete slabs is common. The cracking occurs because concrete shrinks as it cures. Control joints, which are commonly used in exterior decking to control such cracking, are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the performance of the slab. The minor shrinkage cracks, which often form in interior slabs, generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile.

EXTERIOR CONCRETE DECKS

Decking should be cast over bedrock or approved compacted fill and reinforced with a minimum of #3 bars placed 24 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill. The subgrade should be moistened prior to placing concrete.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Roof gutters are recommended. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Planters located next to raised-floor-type construction also should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Low-Impact Development (LID) Requirements

Typically, infiltration systems are utilized in areas underlain by pervious granular earth materials that have high percolation characteristics. In addition, infiltration systems are normally planned at least 10 feet from adjacent property lines or public right-of-way, and 15 feet from a 1:1 plane projected from the bottom of adjacent structural foundations. Due to the presence of hard, relatively impermeable bedrock, infiltration pits are not recommended for the subject site.

As an alternative, a flow-through biofiltration system may be installed on the site in accordance with the City of Los Angeles, Best Management Practices. A planter box may be used to capture and treat storm-water runoff through different fill layers before discharging water to the street or storm drain. The planter box should be impermeable, and may be situated above ground and placed adjacent to buildings. Planter boxes should be designed as freestanding and for an inward equivalent fluid pressure of 43 pounds-per-cubic-foot. This fluid pressure includes possible vehicular surcharge. Byer Geotechnical, Inc., should be provided with the final plans to verify the location of the planter boxes.

Irrigation

Control of irrigation water is a necessary part of site maintenance. Soggy ground and perched water may result if irrigation water is excessively applied. Irrigation systems should be adjusted to provide the minimum water needed. Adjustments should be made for changes in climate and rainfall.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage, and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾-inch crushed gravel to help the collection of water. Landscape areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by Byer Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The building department requires that the geotechnical engineer provide site observations during grading and construction. Foundation excavations should be observed and approved by the geotechnical engineer or geologist prior to placing steel, forms, or concrete. The engineer/geologist should observe bottoms for fill, compaction of fill, temporary slopes, permanent cut slopes, and subdrains. All fill that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

Please advise Byer Geotechnical, Inc., at least 24 hours prior to any required site visit. The building department stamped plans, the permits, and the geotechnical reports should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site with the findings. This notice should be given to the agency inspector.

FINAL REPORTS

The geotechnical engineer will prepare interim and final compaction reports upon request. The geologist will prepare reports summarizing pile excavations.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. The area should be fenced and warning signs posted. All excavations must be covered and secured. Soil generated by foundation excavations should be either removed from the site or placed as compacted fill. Soil should not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep. Water shall not be allowed to saturate open footing trenches.

GENERAL CONDITIONS AND NOTICE

This report and the exploration are subject to the following conditions. Please read this section carefully; it limits our liability.

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by Byer Geotechnical, Inc., and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein have been projected from test excavations on the site and may not reflect any variations that occur between these test excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report, issued and made for the sole use and benefit of the client, is not transferable. Any liability in connection herewith shall not exceed the Phase I fee for the exploration and report or a negotiated fee per the Agreement. No warranty is expressed, implied, or intended in connection with the exploration performed or by the furnishing of this report.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

Giuseppe Cugno
E. G. 1804

CERTIFIED ENGINEERING GEOLOGIST
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GEOTECHNICAL
STATE OF CALIFORNIA

GC:RSB:mh
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Enc: List of References
- Appendix I - Laboratory Testing and Log of Test Pits
  - Laboratory Testing (2 Pages)
  - Shear Diagrams (2 Pages)
  - Log of Test Pits 1 - 9 (3 Pages)
- Appendix II - Calculations and Figures
  - Seismic Hazard Deaggregation Chart
  - Temporary Excavation Height Calculation Sheet
  - Shoring Pile Calculation Sheets (2 Pages)
  - Retaining Wall Calculation Sheets (6 Pages)
  - Slope Stability Calculation Sheets (7 Pages)
  - Aerial Photo
  - Local Topographic Map
  - Regional Geologic Maps #1 and #2
  - Regional Fault Map
  - Seismic Hazard Zones Map
  - Sections A, B, and C (2 Sheets)
  - Geologic Map

xc: (3) Addressee (E-mail and Pick Up)
(1) Addressee (E-mail and Mail)

REFERENCES

California Building Standards Commission (2016), **2016 California Building Code,** Based on the 2015 International Building Code (IBC), Title 24, Part 2, Vol. 1 and 2.

California Department of Conservation (1999), **State of California, Seismic Hazard Zones, Hollywood Quadrangle**, Official Map, Division of Mines and Geology.

California Department of Conservation (1999a), **State of California, Earthquake Zones of Required Investigation, Pasadena Quadrangle**, Official Map, California Geological Survey.

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Dibblee, T. W. (1991), **Geologic Map of the Hollywood and Burbank (south ½) Quadrangle, Los Angeles County, California**, 1:24,000 scale, Dibblee Foundation, Santa Barbara, California, Map DF-30.

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Hoots, H. W. (1931), **Geology of the Eastern Part of the Santa Monica Mountains, Los Angeles County, California**, U. S. Geological Survey Professional Paper 165-C.

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Software

Slide 7.017, Rocscience, Inc., 2016.

APPENDIX I

Laboratory Testing and Log of Test Pits

LABORATORY TESTING

Undisturbed and bulk samples of the fill, soil, and bedrock were obtained from the test pits and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring-lined, barrel sampler conforming to ASTM D 3550-01 with successive drops of the sampler. Experience has shown that sampling causes some disturbance of the sample. However, the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height and stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D 2937-10. The moisture content of the samples was determined using the procedures outlined in ASTM D 2216-10. The results are shown on the enclosed Log of Test Pits.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill were determined using the procedures outlined in ASTM D 1557-12, a five-layer standard.

Test Pit	Depth (Feet)	Earth Material	Color and Soil Type	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
1	3 - 5	Bedrock	Medium Brown Silty Sand	121.0	12.0	29 - Low

Expansion Test

To find the expansiveness of the soil, a swell test was performed using the procedures outlined in ASTM D 4829-11. Based upon the testing, the earth materials are expected to exhibit a low expansion potential.

LABORATORY TESTING (Continued)

Shear Tests

Shear tests were performed on samples of bedrock using the procedures outlined in ASTM D 3080-11 and a strain controlled, direct-shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inch per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the enclosed Shear Test Diagram.

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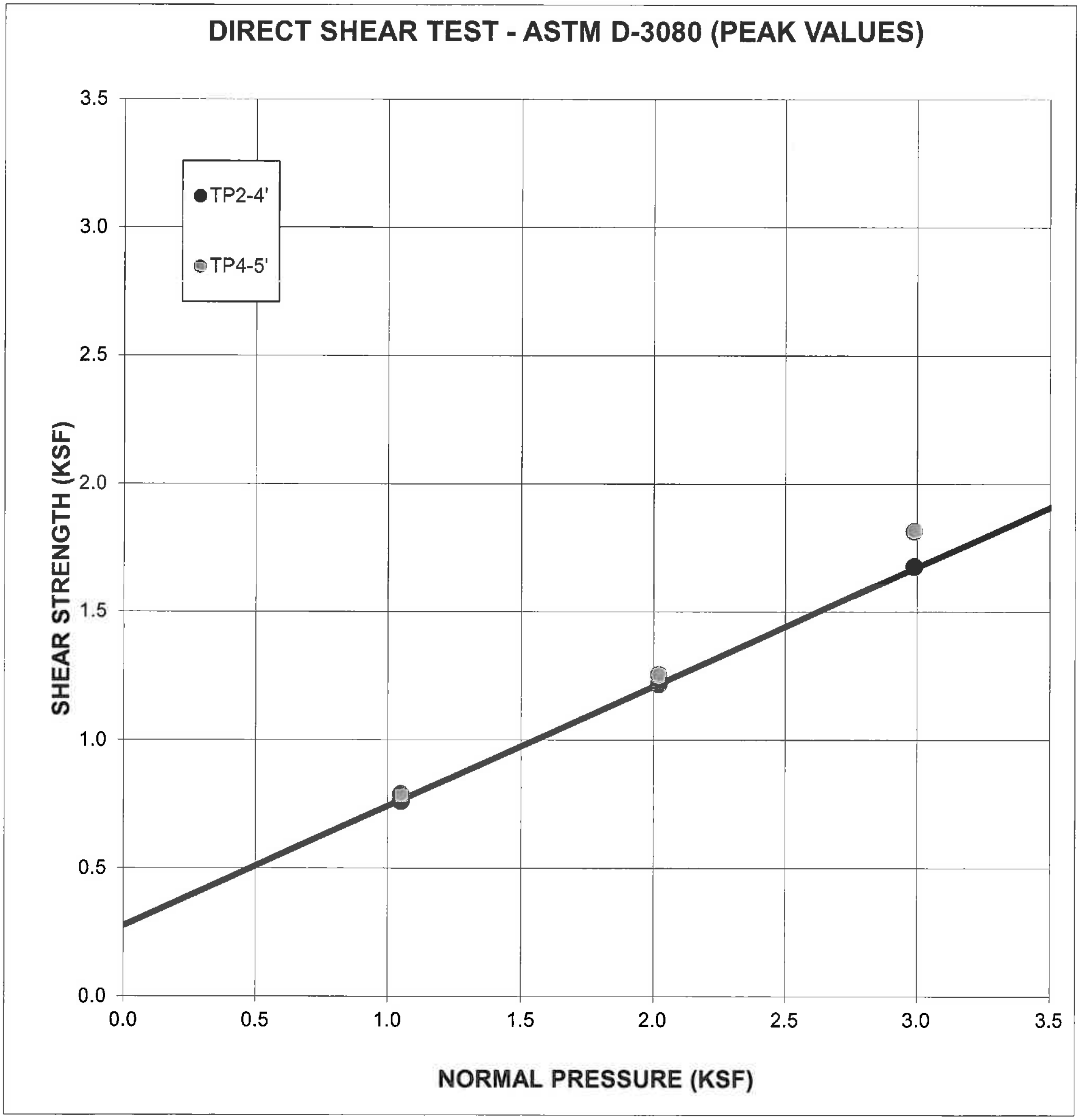
SHEAR DIAGRAM #1

BG 22747 CONSULTANT: MP/GC
CLIENT: NEAGU

EARTH MATERIAL: SOIL

Phi Angle =	**25 degrees**	**Average Mostiure Content**	**24.8%**
Cohesion =	**275 psf**	**Average Dry Density (pcf)**	**99.30%**
		Average Saturation	**98.5%**



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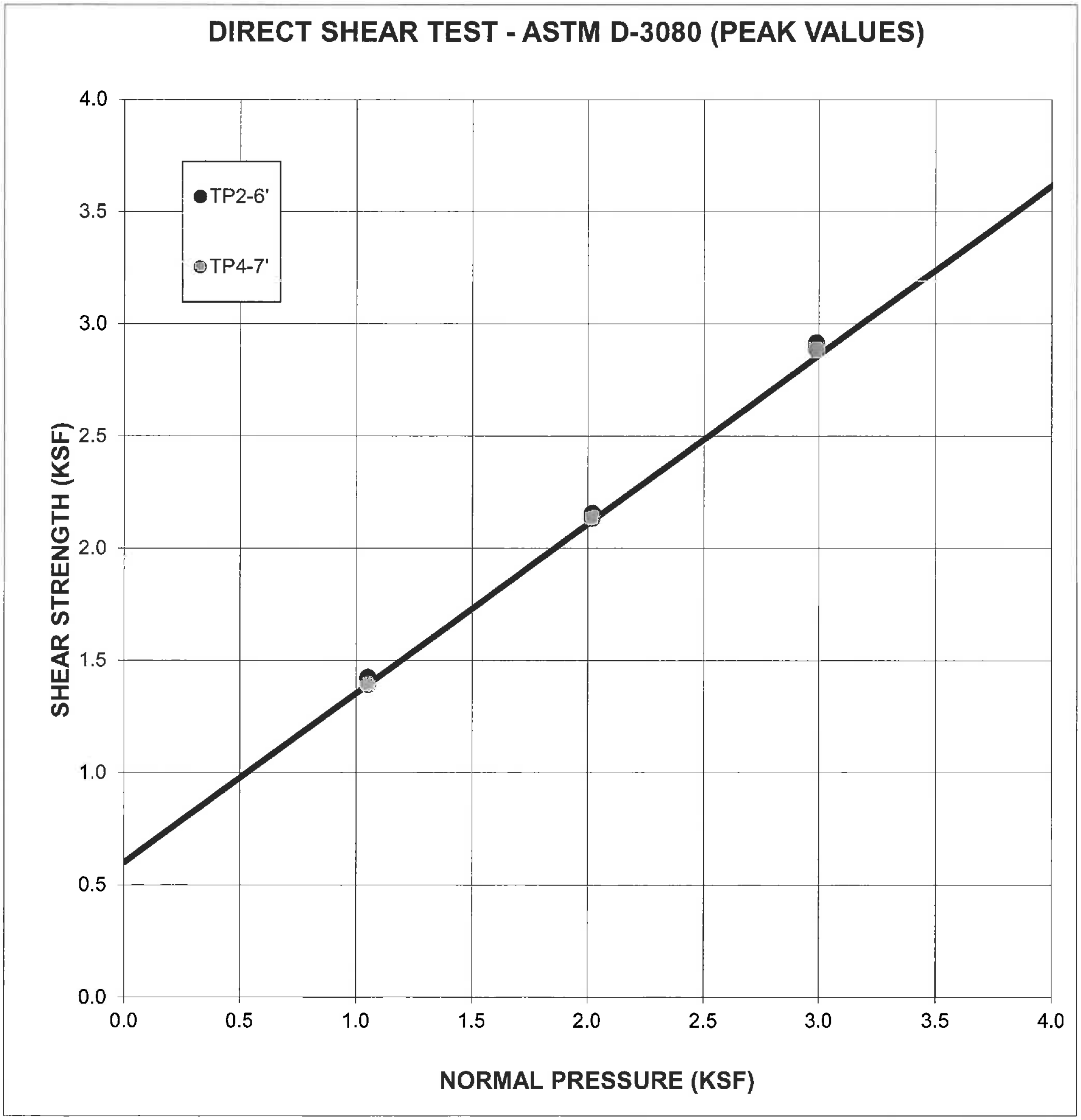
SHEAR DIAGRAM #2

BG 22747 CONSULTANT: MP/GC
CLIENT: NEAGU

EARTH MATERIAL: BEDROCK

Phi Angle =	**37 degrees**	**Average Mostiure Content**	**19.7%**
Cohesion =	**600 psf**	**Average Dry Density (pcf)**	**108.10%**
		Average Saturation	**98.5%**



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LOG OF TEST PITS

CLIENT: NEAGU

MAPPED BY: MP/GC BG: 22747

REPORT DATE: 12/8/17 DATE LOGGED: 10/12/17

SAMPLE DEPTH (feet)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DEPTH INTERVAL (feet)	EARTH MATERIAL	LITHOLOGIC DESCRIPTION
TEST PIT #1			Surface Conditions: Slope, southwest of residence		
			0 - 2	**FILL:**	Sandy SILT, medium brown, dry, loose
2	9.2	83.6	2 - 4	**SOIL:**	Silty SAND, brown, slightly moist, slightly to medium dense
4	7.1	114.7	4 - 5½	**BEDROCK:**	Granite, tan, light yellowish-brown, salt and pepper texture, moderately hard to hard, massive, slightly fractured
End at 5½ Feet; No Water; No Caving; Fill to 2 Feet.					
TEST PIT #2			Surface Conditions: Slope, west of residence		
			0 - 3	**FILL:**	Sandy SILT, medium brown, dry, loose
4	5.8	98.7	3 - 4½	**SOIL:**	Silty SAND, brown, slightly moist, medium dense
6	10.4	106.9	4½ - 6	**BEDROCK:**	Granite, tan, light yellowish-brown, moderately hard to hard, massive, slightly fractured
End at 6 Feet; No Water; No Caving; Fill to 3 Feet.					
TEST PIT #3			Surface Conditions: Slope, northwest of residence		
			0 - 2	**FILL:**	Sandy SILT, medium brown, dry, loose
					at 20 inches: thin layer of black material
2	6.5	92.4	2 - 3½	**SOIL:**	Silty SAND, brown, slightly moist, slightly dense
5	4.5	120.8	3½ - 5	**BEDROCK:**	Granite, tan, light yellowish-brown, moderately hard, slightly weathered, slightly fractured
End at 5 Feet; No Water; No Caving; Fill to 2 Feet.					

NOTE: The stratification depths shown on the Log of Test Pits are approximate and are based upon visual classification of samples and cuttings. The actual depths may vary. Variations between test pits may also occur.

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LOG OF TEST PITS

CLIENT: NEAGU

MAPPED BY: MP/GC BG: 22747

REPORT DATE: 12/8/17 DATE LOGGED: 10/27/17

SAMPLE DEPTH (feet)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DEPTH INTERVAL (feet)	EARTH MATERIAL	LITHOLOGIC DESCRIPTION
TEST PIT #4			Surface Conditions: Top of slope, northwest of residence		
2	4.2	101.5	0 - 5	**FILL:**	Sandy SILT/Silty SAND, brown, dry, loose
					at 4½ feet: rock fragments up to 4" diameter
5	5.5	99.8	5 - 6	**SOIL:**	Silty SAND, brown, slightly moist, medium dense
7	5.9	109.3	6 - 7	**BEDROCK:**	Granite, tan, brown, moderately hard to hard
End at 7 Feet; No Water; No Caving; Fill to 5 Feet.					
TEST PIT #5			Surface Conditions: Top of slope, west of residence		
2	3.7	113.4	0 - 7	**FILL:**	Sandy SILT/Silty SAND, medium brown, dry, loose
			7 - 8	**SOIL:**	Silty SAND, brown, slightly moist, medium dense
			8 - 8½	**BEDROCK:**	Granite, tan, brown, salt and pepper texture, moderately hard to hard
End of Test Pit at 5 Feet; Hand-Auger from 5 Feet to 8½ Feet; No Water; No Caving; Fill to 7 Feet.					
TEST PIT #6			Surface Conditions: Top of slope, southwest of residence		
4	4.4	107.8	0 - 7	**FILL:**	Sandy SILT/Silty SAND, medium brown, dry, loose
			7 - 8	**SOIL:**	Silty SAND, brown, slightly moist, slightly to medium dense
			8 - 9	**BEDROCK:**	Granite, tan, brown, salt and pepper texture, moderately hard to hard
End of Test Pit at 5 Feet; Hand-Auger from 5 Feet to 9 Feet; No Water; No Caving; Fill to 7 Feet.					

NOTE: The stratification depths shown on the Log of Test Pits are approximate and are based upon visual classification of samples and cuttings. The actual depths may vary. Variations between test pits may also occur.

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LOG OF TEST PITS

CLIENT: NEAGU

MAPPED BY: MP/GC BG: 22747

REPORT DATE: 12/8/17 DATE LOGGED: 10/12/17

SAMPLE DEPTH (feet)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DEPTH INTERVAL (feet)	EARTH MATERIAL	LITHOLOGIC DESCRIPTION
TEST PIT #7			Surface Conditions: Slope, near driveway, south of residence		
			0 - 18"	**FILL:**	Sandy SILT, medium brown, dry, loose
			18" - 2	**SOIL:**	Silty SAND, brown, slightly moist, slightly to medium dense
3	2.4	118.8	2 - 3	**BEDROCK:**	Granite, tan, light yellowish-brown, salt and pepper texture, moderately hard to hard
End at 3 Feet; No Water; No Caving; Fill to 18 Inches.					
TEST PIT #8			Surface Conditions: Slope, southeast of residence		
			0 - 3	**FILL:**	Silty SAND, brown to medium brown, slightly moist, medium dense
			3 - 4	**BEDROCK:**	Granite, tan, light yellowish-brown, moderately hard, massive
End at 4 Feet; No Water; No Caving; Fill to 3 Feet.					
TEST PIT #9			Surface Conditions: Planter area near driveway, east of residence		
			0 - ¾	**FILL:**	Silty SAND, medium grown, dry, loose
			¾ - 1	**BEDROCK:**	Granite, tan, light yellowish-brown, moderately hard to hard, massive,
End at 1 Foot; No Water; No Caving; Fill to ¾ Foot.					

NOTE: The stratification depths shown on the Log of Test Pits are approximate and are based upon visual classification of samples and cuttings. The actual depths may vary. Variations between test pits may also occur.

APPENDIX II

Calculations and Figures

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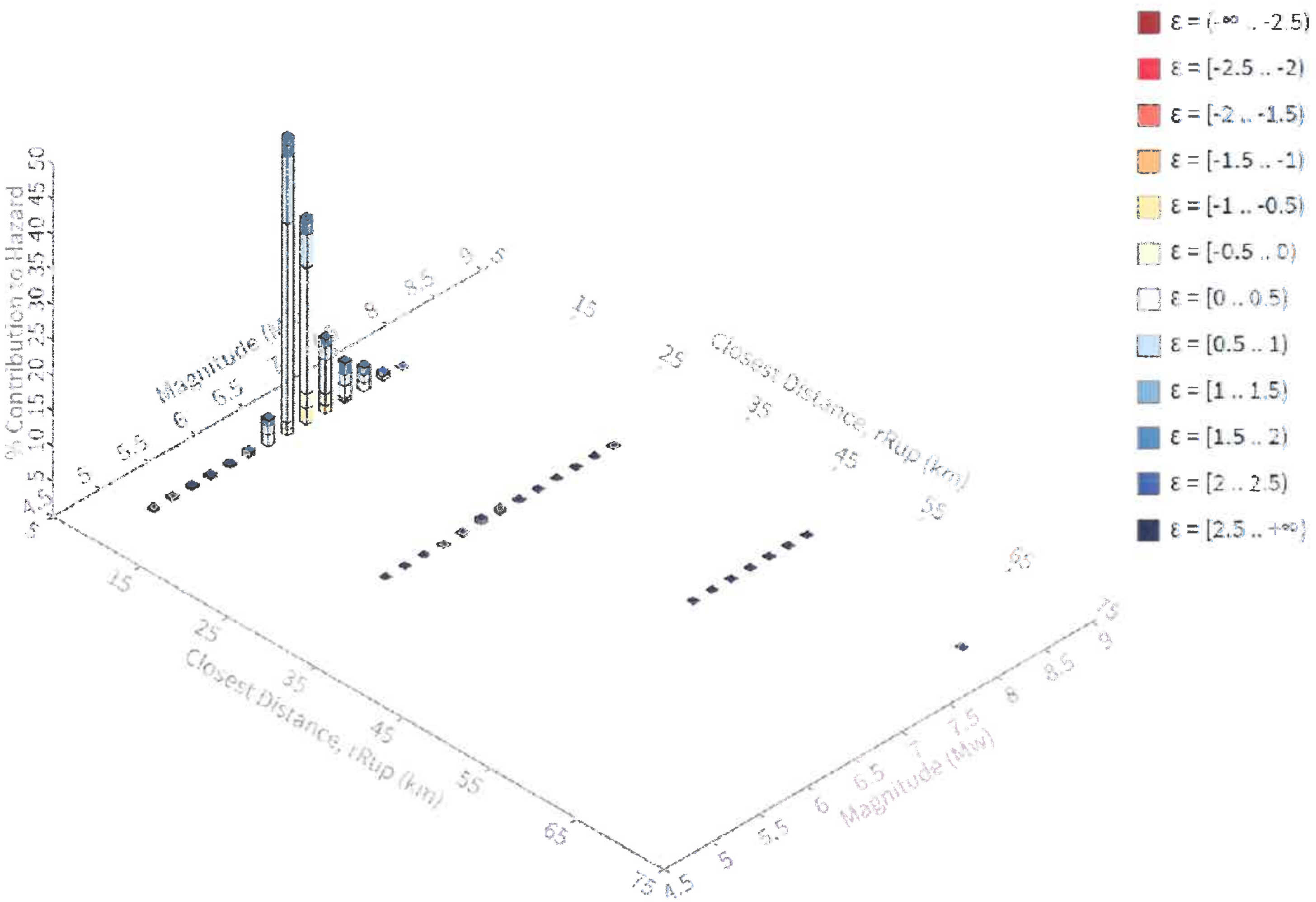
SEISMIC HAZARD DEAGGREGATION CHART
(Probability of Exceedance: 10% in 50 years)

BG: 22747 CLIENT: NEAGU

ENGINEER: GC

REFERENCE: USGS, 2017, Earthquake Hazards Program, Beta - Unified Hazard Tool, Seismic Hazard Deaggregation, Conterminous U.S. 2008 (v3.3.0) Edition, https://earthquake.usgs.gov/hazards/interactive/index.php.



Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.59010514 g

Recovered targets

Return period: 485.04959 yrs
Exceedance rate: 0.0020616449 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.09 %

Mean (for all sources)

r: 6.88 km
m: 6.66
ε₀: 0.49 σ

Mode (largest r-m bin)

r: 6.47 km
m: 6.51
ε₀: 0.44 σ
Contribution: 42.15 %

Mode (largest ε₀ bin)

r: 6.73 km
m: 6.51
ε₀: 0.25 σ
Contribution: 28.21 %

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TEMPORARY EXCAVATION HEIGHT

BG: **22747** CONSULTANT: **GC**
CLIENT: **NEAGU**

CALCULATION SHEET # 1

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST). THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT:	10 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	45 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	420.0 psf	FINAL TENSION CRACK:	10 feet

PHID = ATAN(TAN(PHI)/FS) = 29.3 degrees

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	57 degrees
AREA OF TRIAL FAILURE WEDGE	9.7 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	1362.2 pounds
NUMBER OF TRIAL WEDGES ANALYZED	410 trials
LENGTH OF FAILURE PLANE	1.8 feet
DEPTH OF TENSION CRACK	9.5 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	1.0 feet
CALCULATED HORIZONTAL THRUST	**-43.7 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-0.9 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**10.0 feet**

CONCLUSIONS:

THE CALCULATION INDICATES THAT TEMPORARY EXCAVATIONS UP TO 10 FEET HIGH IN BEDROCK WITH A 1:1 BACKSLOPE HAVE A NEGATIVE THRUST AND ARE TEMPORARILY STABLE.

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SHORING PILE

BG: **22747** CONSULTANT: **GC**
CLIENT: **NEAGU**

CALCULATION SHEET # **2**

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	RETAINED LENGTH	25 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	350.0 psf	FINAL TENSION CRACK:	10 feet

PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	59 degrees
AREA OF TRIAL FAILURE WEDGE	166.8 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	23350.0 pounds
NUMBER OF TRIAL WEDGES ANALYZED	410 trials
LENGTH OF FAILURE PLANE	19.4 feet
DEPTH OF TENSION CRACK	8.4 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	10.0 feet
CALCULATED THRUST ON PILE	**8310.4 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**26.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**30.0 pcf**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED SHORING PILES TO A HEIGHT OF 25 FEET WITH A LEVEL BACKSLOPE MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 30 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

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SHORING PILE

BG: **22747** CONSULTANT: **GC**
CLIENT: **NEAGU**

CALCULATION SHEET # **3**

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	RETAINED LENGTH	20 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	34 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	350.0 psf	FINAL TENSION CRACK:	10 feet

PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	57 degrees
AREA OF TRIAL FAILURE WEDGE	156.7 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	21942.5 pounds
NUMBER OF TRIAL WEDGES ANALYZED	410 trials
LENGTH OF FAILURE PLANE	18.4 feet
DEPTH OF TENSION CRACK	11.3 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	10.0 feet
CALCULATED THRUST ON PILE	**6834.0 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**34.2 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**35.0 pcf**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED SHORING PILES TO A HEIGHT OF 20 FEET WITH A 1½:1 BACKSLOPE (34 Degrees) MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 35 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

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RETAINING WALL

BG: **22747** CONSULTANT: **GC**
CLIENT: **NEAGU**

CALCULATION SHEET # **4**

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT	25 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	350.0 psf	FINAL TENSION CRACK:	50 feet

PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	57 degrees
AREA OF TRIAL FAILURE WEDGE	181.8 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	25457.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	2050 trials
LENGTH OF FAILURE PLANE	20.2 feet
DEPTH OF TENSION CRACK	8.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	11.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**8341.9 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**26.7 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**43.0 pcf**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL TO A HEIGHT OF 25 FEET MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 43 POUNDS PER CUBIC FOOT.

BYER
GEOTECHNICAL
INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL

BG: **22747** CONSULTANT: **GC**
CLIENT: **NEAGU**

CALCULATION SHEET # **4s**

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT	25 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	0 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	525.0 psf	FINAL TENSION CRACK:	56 feet

PHID = ATAN(TAN(PHI)/FS) = 35.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.36 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	49 degrees
AREA OF TRIAL FAILURE WEDGE	245.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	34381.7 pounds
NUMBER OF TRIAL WEDGES ANALYZED	2296 trials
LENGTH OF FAILURE PLANE	22.9 feet
DEPTH OF TENSION CRACK	7.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	15.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**10816.0 pounds**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL TO A HEIGHT OF 25 FEET WILL BE SUBJECT TO A THRUST OF 10816 POUNDS UNDER THE GIVEN SEISMIC CONDITION. SINCE THIS IS LESS THAT THE STATIC DESIGN FOR A 43 PCF WALL (13438 pounds) NO ADDITIONAL REINFORCEMENT IS NEEDED.

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RETAINING WALL

BG: 22747 CONSULTANT: GC
CLIENT: NEAGU

CALCULATION SHEET # 5

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT	20 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	34 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	350.0 psf	FINAL TENSION CRACK:	15 feet

PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	53 degrees
AREA OF TRIAL FAILURE WEDGE	226.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	31722.6 pounds
NUMBER OF TRIAL WEDGES ANALYZED	615 trials
LENGTH OF FAILURE PLANE	24.9 feet
DEPTH OF TENSION CRACK	10.2 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	15.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**7899.8 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**39.5 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**43.0 pcf**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL TO A HEIGHT OF 20 FEET AND A BACKSLOPE AS STEEP AS 1½:1 MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 43 POUNDS PER CUBIC FOOT.

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RETAINING WALL

BG: 22747 CONSULTANT: GC
CLIENT: NEAGU

CALCULATION SHEET # **5s**

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT	20 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	34 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	525.0 psf	FINAL TENSION CRACK:	15 feet

PHID = ATAN(TAN(PHI)/FS) = 35.0 degrees
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.36 %g
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	49 degrees
AREA OF TRIAL FAILURE WEDGE	246.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	34505.2 pounds
NUMBER OF TRIAL WEDGES ANALYZED	615 trials
LENGTH OF FAILURE PLANE	22.9 feet
DEPTH OF TENSION CRACK	12.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	15.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**10891.3 pounds**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL TO A HEIGHT OF 20 FEET WITH A 1½:1 BACKSLOPE WILL BE SUBJECT TO A THRUST OF 10,891 POUNDS UNDER THE GIVEN SEISMIC CONDITION. SINCE THIS IS LESS THAT THE STATIC DESIGN FOR A 55 EFP WALL (11,000 POUNDS) NO ADDITIONAL REINFORCEMENT IS NEEDED.

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RETAINING WALL

BG: 22747 CONSULTANT: GC
CLIENT: NEAGU

CALCULATION SHEET # 6

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT	20 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	40 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	350.0 psf	FINAL TENSION CRACK:	15 feet

PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	54 degrees
AREA OF TRIAL FAILURE WEDGE	239.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	33537.8 pounds
NUMBER OF TRIAL WEDGES ANALYZED	615 trials
LENGTH OF FAILURE PLANE	25.5 feet
DEPTH OF TENSION CRACK	11.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	15.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**9320.8 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**46.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**63.0 pcf**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL TO A HEIGHT OF 20 FEET AND A BACKSLOPE AS STEEP AS 40 DEGREES, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 63 POUNDS PER CUBIC FOOT.

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RETAINING WALL

BG: **22747** CONSULTANT: **GC**
CLIENT: **NEAGU**

CALCULATION SHEET # **6s**

CALCULATE THE DESIGN EQUIVALENT FLUID PRESSURE FOR PROPOSED RETAINING WALL. THE RETAINED HEIGHT, BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	WALL HEIGHT	20 feet
SHEAR DIAGRAM:	1	BACKSLOPE ANGLE:	40 degrees
COHESION:	525 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	140 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	525.0 psf	FINAL TENSION CRACK:	15 feet

PHID = ATAN(TAN(PHI)/FS) = 35.0 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.36 %g

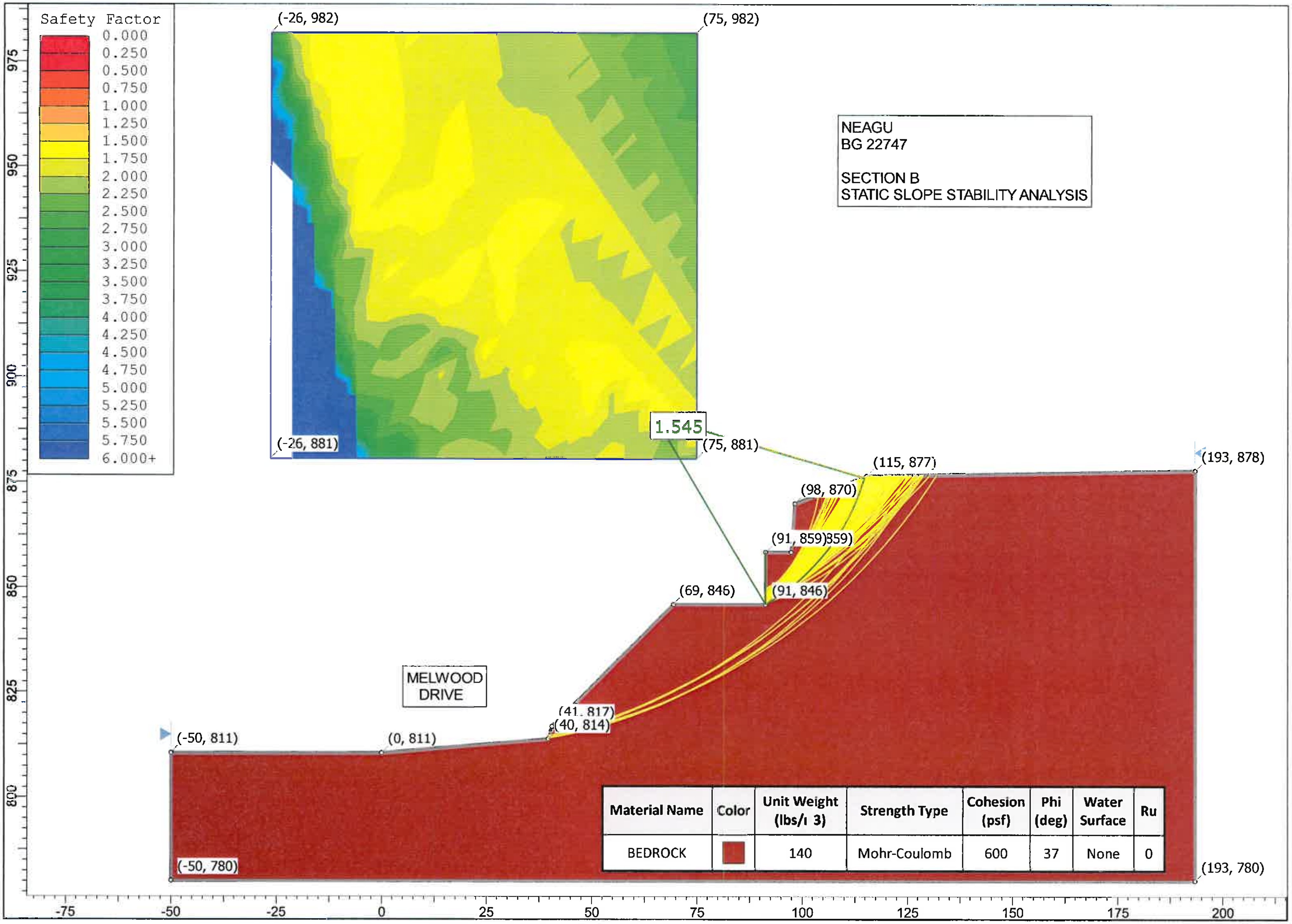
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50 degrees
AREA OF TRIAL FAILURE WEDGE	260.3 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	36445.7 pounds
NUMBER OF TRIAL WEDGES ANALYZED	615 trials
LENGTH OF FAILURE PLANE	23.3 feet
DEPTH OF TENSION CRACK	14.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	15.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**12496.3 pounds**

Conclusions:

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL TO A HEIGHT OF 20 FEET WITH A 40 DEGREE BACKSLOPE WILL BE SUBJECT TO A THRUST OF 12,496.3 POUNDS UNDER THE GIVEN SEISMIC CONDITION. SINCE THIS IS LESS THAT THE STATIC DESIGN FOR A 63 EFP WALL (12,600 POUNDS) NO ADDITIONAL REINFORCEMENT IS NEEDED.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
BEDROCK		140	Mohr-Coulomb	600	37	None	0

Slide Analysis Information

SLIDE - An Interactive Slope Stability Program

Project Summary

File Name:	SECTION B STATIC
Slide Modeler Version:	7.024
Project Title:	SLIDE - An Interactive Slope Stability Program
Date Created:	12/8/2017, 9:15:10 AM

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Failure Direction:	Right to Left
Data Output:	Standard
Maximum Material Properties:	20
Maximum Support Properties:	20

Analysis Options

Slices Type:	Vertical

Analysis Methods Used

Bishop simplified

Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Groundwater Analysis

Groundwater Method:	Water Surfaces
Pore Fluid Unit Weight [lbs/ft3]:	62.4
Use negative pore pressure cutoff:	Yes
Maximum negative pore pressure [psf]:	0
Advanced Groundwater Method:	None

Random Numbers

Pseudo-random Seed:	10116
Random Number Generation Method:	Park and Miller v.3

Surface Options

Surface Type:	Circular
Search Method:	Grid Search
Radius Increment:	10
Composite Surfaces:	Disabled
Reverse Curvature:	Invalid Surfaces
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Material Properties

Property	BEDROCK
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	140
Cohesion [psf]	600
Friction Angle [deg]	37
Water Surface	None
Ru Value	0

Global Minimums

Method: bishop simplified

FS	1.545460
Center:	64.750, 890.840
Radius:	51.994
Left Slip Surface Endpoint:	91.200, 846.077
Right Slip Surface Endpoint:	114.656, 876.255
Left Slope Intercept:	91.200 858.500
Right Slope Intercept:	114.656 876.255
Resisting Moment:	2.21691e+006 lb-ft
Driving Moment:	1.43447e+006 lb-ft
Total Slice Area:	276.717 ft2
Surface Horizontal Width:	23.4563 ft
Surface Average Height:	11.7971 ft

Valid / Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces:	4338
Number of Invalid Surfaces:	513

Error Codes:

Error Code -103 reported for 108 surfaces
Error Code -106 reported for 82 surfaces
Error Code -108 reported for 323 surfaces

Error Codes

The following errors were encountered during the computation:

-103 = Two surface / slope intersections, but one or more surface / nonslope external polygon intersections lie between them. This usually occurs when the slip surface extends past the bottom of the soil region, but may also occur on a benched slope model with two sets of Slope Limits.
-106 = Average slice width is less than 0.0001 * (maximum horizontal extent of soil region). This limitation is imposed to avoid numerical errors which may result from too many slices, or too small a slip region.
-108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.54546

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [degrees]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	0.469127	806.709	30.8795	BEDROCK	600	37	949.785	1467.86	1151.68	0	1151.68	1719.65	1719.65
2	0.469127	788.062	31.4837	BEDROCK	600	37	929.722	1436.85	1110.54	0	1110.54	1679.91	1679.91
3	0.469127	768.967	32.0919	BEDROCK	600	37	909.422	1405.47	1068.9	0	1068.9	1639.2	1639.2
4	0.469127	749.414	32.7042	BEDROCK	600	37	888.883	1373.73	1026.78	0	1026.78	1597.52	1597.52
5	0.469127	729.394	33.3207	BEDROCK	600	37	868.104	1341.62	984.163	0	984.163	1554.85	1554.85
6	0.469127	708.898	33.9416	BEDROCK	600	37	847.082	1309.13	941.05	0	941.05	1511.16	1511.16
7	0.469127	687.915	34.5671	BEDROCK	600	37	825.818	1276.27	897.438	0	897.438	1466.43	1466.43
8	0.469127	666.435	35.1973	BEDROCK	600	37	804.307	1243.03	853.323	0	853.323	1420.64	1420.64
9	0.469127	644.444	35.8325	BEDROCK	600	37	782.55	1209.4	808.7	0	808.7	1373.77	1373.77
10	0.469127	621.932	36.4727	BEDROCK	600	37	760.543	1175.39	763.567	0	763.567	1325.78	1325.78
11	0.469127	598.885	37.1183	BEDROCK	600	37	738.286	1140.99	717.918	0	717.918	1276.65	1276.65
12	0.469127	575.289	37.7695	BEDROCK	600	37	715.775	1106.2	671.753	0	671.753	1226.35	1226.35
13	0.469127	551.13	38.4264	BEDROCK	600	37	693.01	1071.02	625.063	0	625.063	1174.86	1174.86
14	0.469127	722.111	39.0894	BEDROCK	600	37	815.698	1260.63	876.685	0	876.685	1539.33	1539.33
15	0.469127	1089.29	39.7587	BEDROCK	600	37	1081.67	1671.68	1422.16	0	1422.16	2322.06	2322.06
16	0.469127	1237.19	40.4345	BEDROCK	600	37	1182.76	1827.91	1629.49	0	1629.49	2637.32	2637.32
17	0.469127	1222.33	41.1172	BEDROCK	600	37	1163.52	1798.17	1590.02	0	1590.02	2605.64	2605.64
18	0.469127	1206.81	41.8071	BEDROCK	600	37	1143.82	1767.73	1549.63	0	1549.63	2572.57	2572.57
19	0.469127	1190.63	42.5045	BEDROCK	600	37	1123.65	1736.56	1508.27	0	1508.27	2538.07	2538.07
20	0.469127	1173.75	43.2097	BEDROCK	600	37	1103.02	1704.67	1465.94	0	1465.94	2502.1	2502.1
21	0.469127	1156.15	43.9232	BEDROCK	600	37	1081.89	1672.02	1422.62	0	1422.62	2464.59	2464.59
22	0.469127	1137.81	44.6454	BEDROCK	600	37	1060.27	1638.6	1378.27	0	1378.27	2425.49	2425.49
23	0.469127	1118.7	45.3767	BEDROCK	600	37	1038.13	1604.39	1332.87	0	1332.87	2384.74	2384.74
24	0.469127	1098.78	46.1176	BEDROCK	600	37	1015.48	1569.38	1286.41	0	1286.41	2342.29	2342.29
25	0.469127	1078.03	46.8685	BEDROCK	600	37	992.286	1533.54	1238.85	0	1238.85	2298.06	2298.06
26	0.469127	1056.41	47.6302	BEDROCK	600	37	968.545	1496.85	1190.16	0	1190.16	2251.97	2251.97
27	0.469127	1033.88	48.4031	BEDROCK	600	37	944.238	1459.28	1140.31	0	1140.31	2203.94	2203.94
28	0.469127	1010.4	49.1879	BEDROCK	600	37	919.351	1420.82	1089.26	0	1089.26	2153.89	2153.89
29	0.469127	985.918	49.9854	BEDROCK	600	37	893.865	1381.43	1036.99	0	1036.99	2101.71	2101.71
30	0.469127	960.393	50.7964	BEDROCK	600	37	867.76	1341.09	983.458	0	983.458	2047.3	2047.3
31	0.469127	933.766	51.6216	BEDROCK	600	37	841.02	1299.76	928.616	0	928.616	1990.54	1990.54
32	0.469127	905.976	52.4622	BEDROCK	600	37	813.619	1257.42	872.421	0	872.421	1931.3	1931.3
33	0.469127	876.955	53.3192	BEDROCK	600	37	785.538	1214.02	814.828	0	814.828	1869.44	1869.44
34	0.469127	846.629	54.1937	BEDROCK	600	37	756.748	1169.52	755.785	0	755.785	1804.8	1804.8
35	0.469127	814.913	55.0871	BEDROCK	600	37	727.226	1123.9	695.235	0	695.235	1737.19	1737.19
36	0.469127	781.711	56.001	BEDROCK	600	37	696.939	1077.09	633.122	0	633.122	1666.42	1666.42
37	0.469127	746.917	56.9371	BEDROCK	600	37	665.857	1029.06	569.376	0	569.376	1592.25	1592.25
38	0.469127	710.407	57.8973	BEDROCK	600	37	633.947	979.739	503.931	0	503.931	1514.42	1514.42
39	0.469127	672.041	58.8838	BEDROCK	600	37	601.168	929.081	436.705	0	436.705	1432.64	1432.64
40	0.469127	631.655	59.8994	BEDROCK	600	37	567.481	877.019	367.616	0	367.616	1346.55	1346.55
41	0.469127	589.06	60.9471	BEDROCK	600	37	532.839	823.482	296.57	0	296.57	1255.75	1255.75
42	0.469127	544.029	62.0305	BEDROCK	600	37	497.194	768.394	223.467	0	223.467	1159.76	1159.76
43	0.469127	496.292	63.1541	BEDROCK	600	37	460.492	711.672	148.193	0	148.193	1058	1058
44	0.469127	445.523	64.323	BEDROCK	600	37	422.671	653.221	70.6271	0	70.6271	949.776	949.776
45	0.469127	391.319	65.5439	BEDROCK	600	37	383.667	592.942	-9.36594	0	-9.36594	834.228	834.228
46	0.469127	333.17	66.8251	BEDROCK	600	37	343.409	530.725	-91.9314	0	-91.9314	710.273	710.273
47	0.469127	270.421	68.1773	BEDROCK	600	37	301.82	466.451	-177.225	0	-177.225	576.513	576.513
48	0.469127	202.203	69.6147	BEDROCK	600	37	258.823	400	-265.409	0	-265.409	431.093	431.093
49	0.469127	127.315	71.1572	BEDROCK	600	37	214.342	331.257	-356.634	0	-356.634	271.452	271.452
50	0.469127	44.0141	72.8333	BEDROCK	600	37	168.324	260.138	-451.011	0	-451.011	93.8764	93.8764

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.54546

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	91.2	846.077	0	0	0
2	91.6691	846.357	122.433	0	0
3	92.1383	846.645	239.492	0	0
4	92.6074	846.939	351.621	0	0
5	93.0765	847.24	459.288	0	0
6	93.5456	847.549	562.98	0	0
7	94.0148	847.864	663.206	0	0
8	94.4839	848.188	760.499	0	0
9	94.953	848.518	855.418	0	0
10	95.4221	848.857	948.549	0	0
11	95.8913	849.204	1040.51	0	0
12	96.3604	849.559	1131.94	0	0
13	96.8295	849.922	1223.51	0	0
14	97.2986	850.295	1315.96	0	0
15	97.7678	850.676	1364.47	0	0
16	98.2369	851.066	1316.81	0	0
17	98.706	851.466	1220.23	0	0
18	99.1752	851.875	1114.91	0	0
19	99.6443	852.295	1001.3	0	0
20	100.113	852.725	879.917	0	0
21	100.583	853.165	751.294	0	0
22	101.052	853.617	616.023	0	0
23	101.521	854.081	474.743	0	0
24	101.99	854.556	328.146	0	0
25	102.459	855.044	176.982	0	0
26	102.928	855.545	22.0673	0	0
27	103.397	856.059	-135.71	0	0
28	103.866	856.587	-295.379	0	0
29	104.336	857.131	-455.882	0	0
30	104.805	857.689	-616.056	0	0
31	105.274	858.264	-774.626	0	0
32	105.743	858.857	-930.187	0	0
33	106.212	859.467	-1081.19	0	0
34	106.681	860.097	-1225.9	0	0
35	107.15	860.748	-1362.42	0	0
36	107.619	861.42	-1488.6	0	0
37	108.089	862.115	-1602.04	0	0
38	108.558	862.836	-1700.03	0	0
39	109.027	863.584	-1779.49	0	0
40	109.496	864.361	-1836.89	0	0
41	109.965	865.17	-1868.2	0	0
42	110.434	866.015	-1868.7	0	0
43	110.903	866.898	-1832.9	0	0
44	111.372	867.825	-1754.25	0	0
45	111.842	868.801	-1624.9	0	0
46	112.311	869.832	-1435.27	0	0
47	112.78	870.928	-1173.43	0	0
48	113.249	872.1	-824.227	0	0
49	113.718	873.362	-367.756	0	0
50	114.187	874.737	223.045	0	0
51	114.656	876.255	0	0	0

List Of Coordinates

External Boundary

X	Y
-50	780
193.4	780
193.4	878
115.3	876.5
98.2	870
97.3	858.5
91.2	858.5
91.2	846
69.3	846
40.6	816.8
39.5	813.8
0	810.5
-50	810.5

BYER GEOTECHNICAL INC.

1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

AERIAL PHOTO

BG: 22747 CLIENT: NEAGU

GEOLOGIST: GC SCALE: 1"=30'

REF: LOS ANGELES COUNTY, DEPARTMENT OF REGIONAL PLANNING, GISNET3 PUBLIC

BYER GEOTECHNICAL INC.

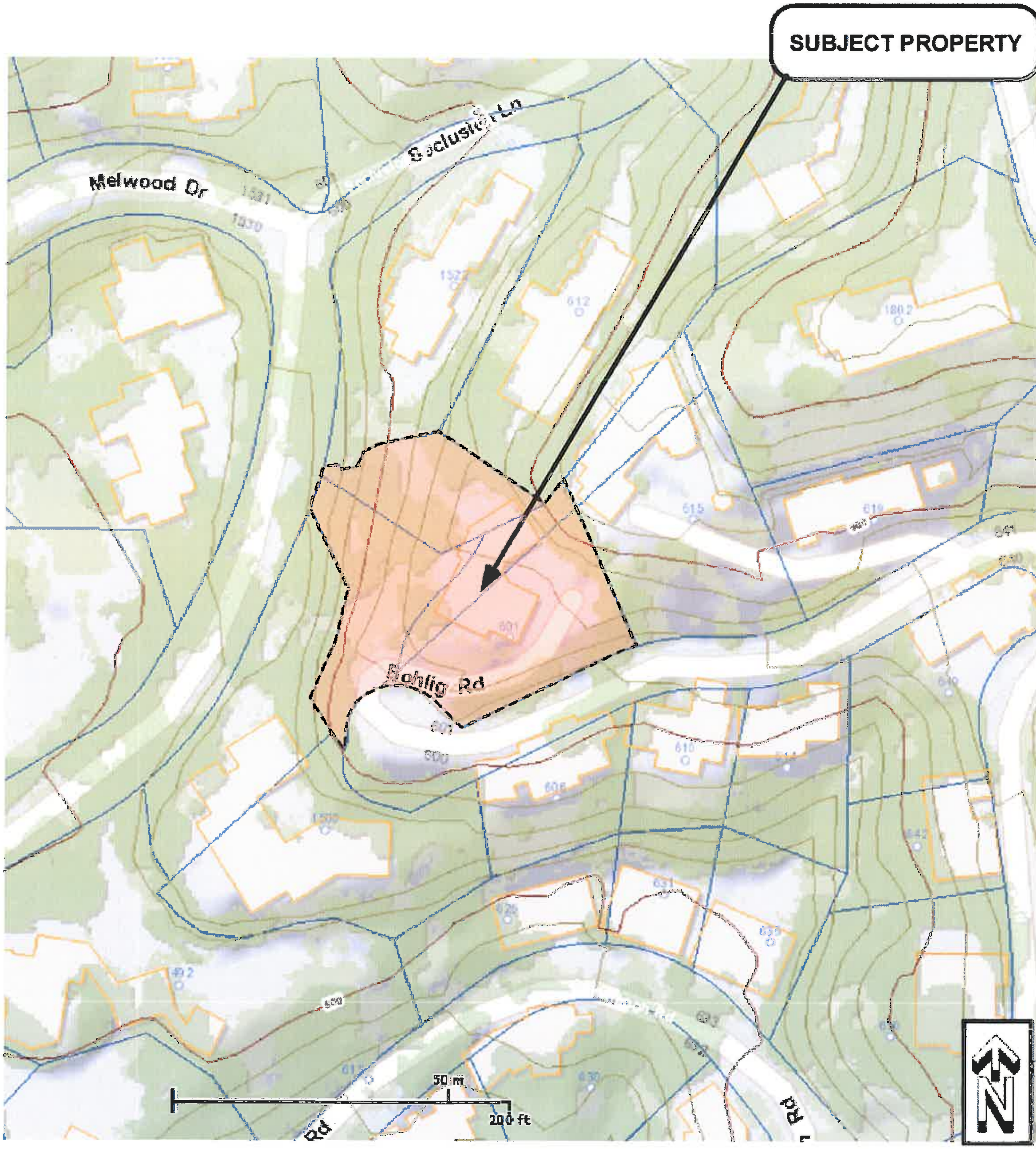
1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

LOCAL TOPOGRAPHIC MAP

BG: 22747 CLIENT: NEAGU

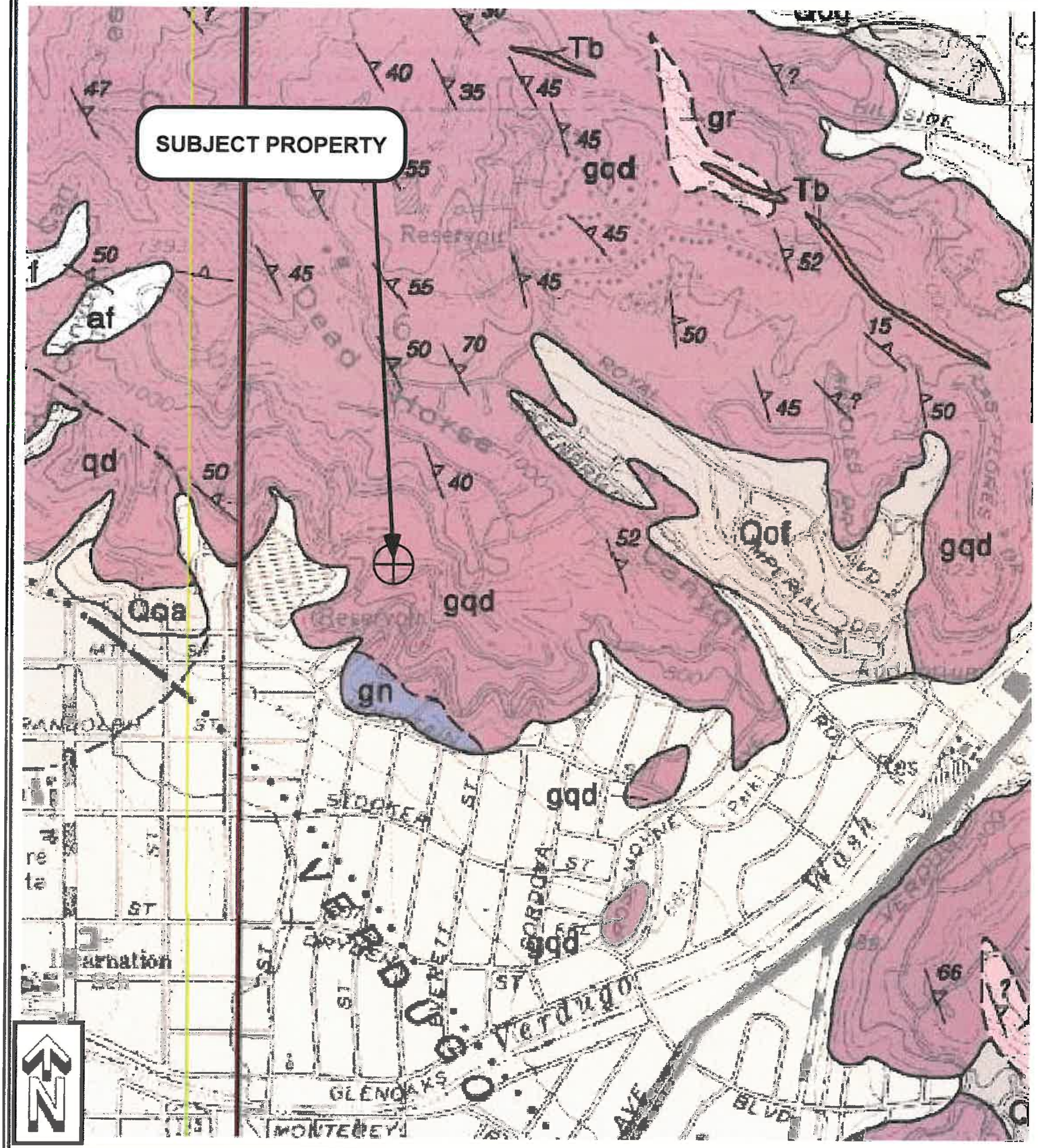
GEOLOGIST: GC SCALE: 1"=80'

REF.: Los Angeles County Department of Regional Planning. GIS-NET3 Public Web Mapping Application.



BYER GEOTECHNICAL INC.	REGIONAL GEOLOGIC MAP #1
1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206 tel 818.549.9959 fax 818.543.3747	BG: 22747 CLIENT: NEAGU GEOLOGIST: GC SCALE: 1"=1,000'

REFERENCE: Dibblee, T. W. (1991), Geologic Map of the Hollywood and Burbank (South 1/2) Quadrangles, Los Angeles County, California, Dibblee Geological Foundation, Santa Barbara, Map No. 30



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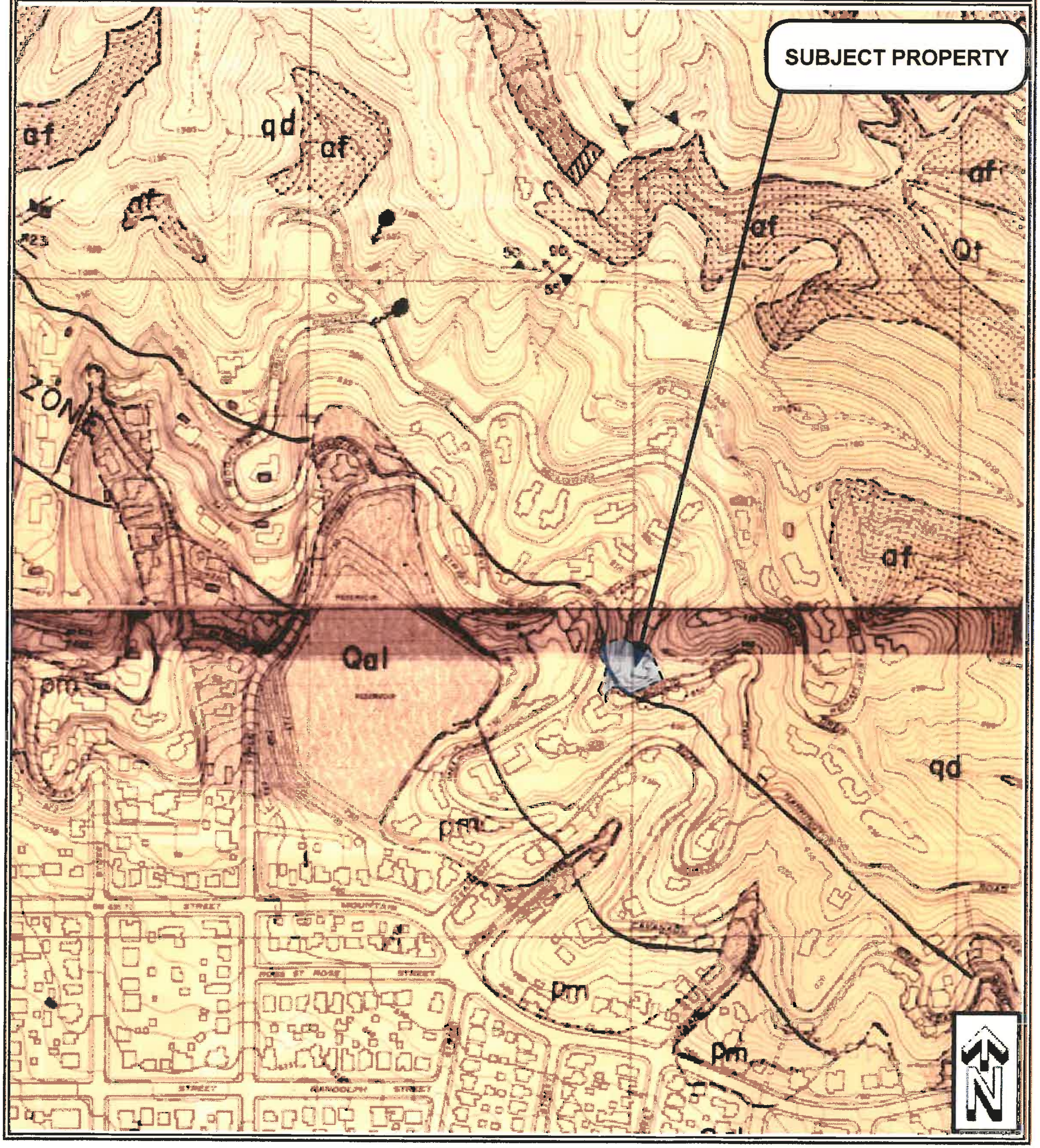
tel 818.549.9959 fax 818.543.3747

REGIONAL GEOLOGIC MAP #2

BG: 22747 CLIENT: NEAGU

GEOLOGIST: GC SCALE: 1"=400'

REF: GEOLOGIC MAP OF A PORTION OF THE VERDUGO MOUNTAINS, GLENDALE, CALIFORNIA (BYER 1968)



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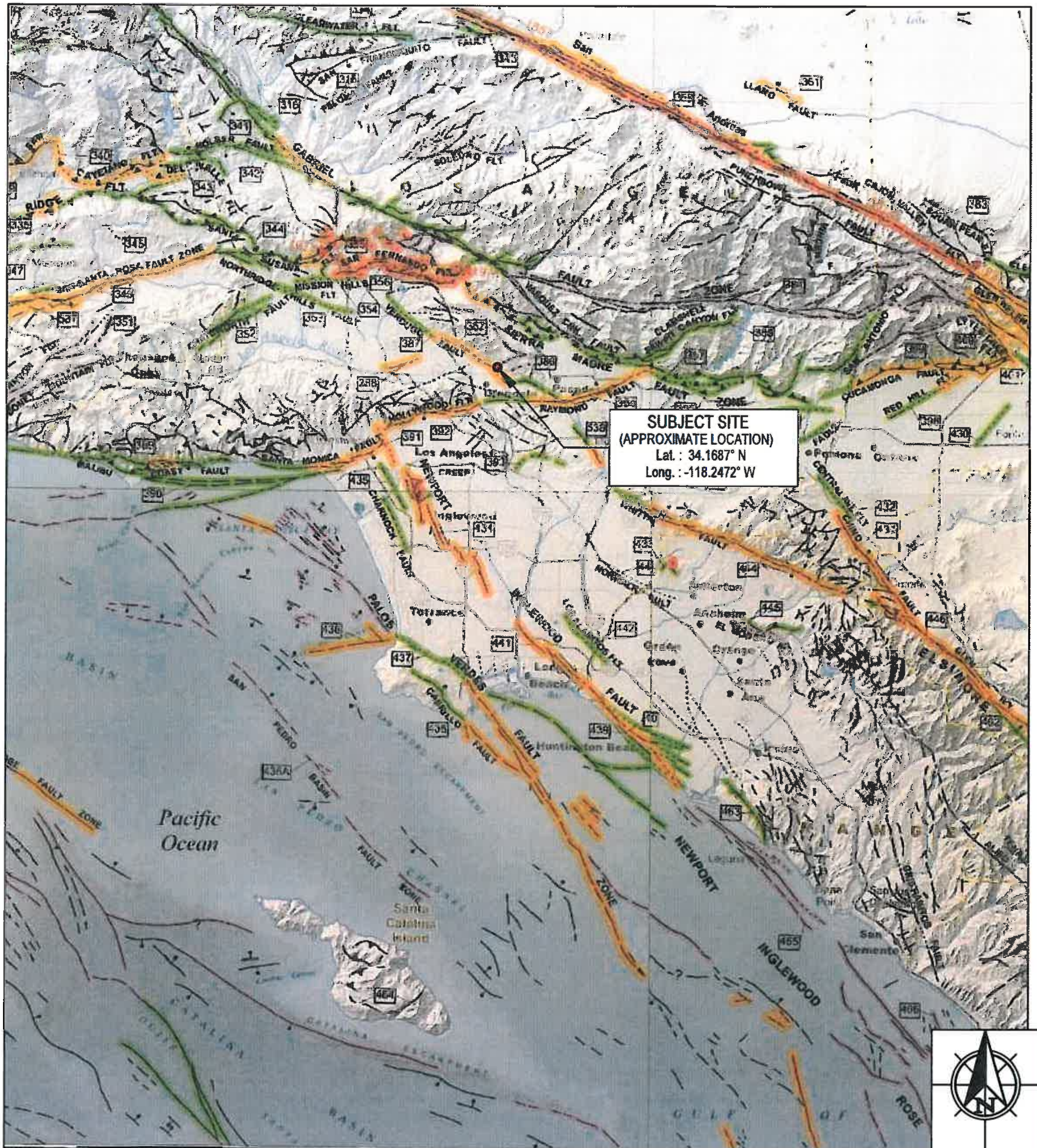
REGIONAL FAULT MAP

BG:	22747 NEAGU		
CONSULTANT:	GC/MP	SCALE:	1" = 12 MILES

REFERENCE: JENNINGS, C.W., AND BRYANT, W.A.,2010, FAULT ACTIVITY MAP OF CALIFORNIA GEOLOGICAL SURVEY, 150th ANNIVERSARY, MAP No 6.



BYER GEOTECHNICAL INC.

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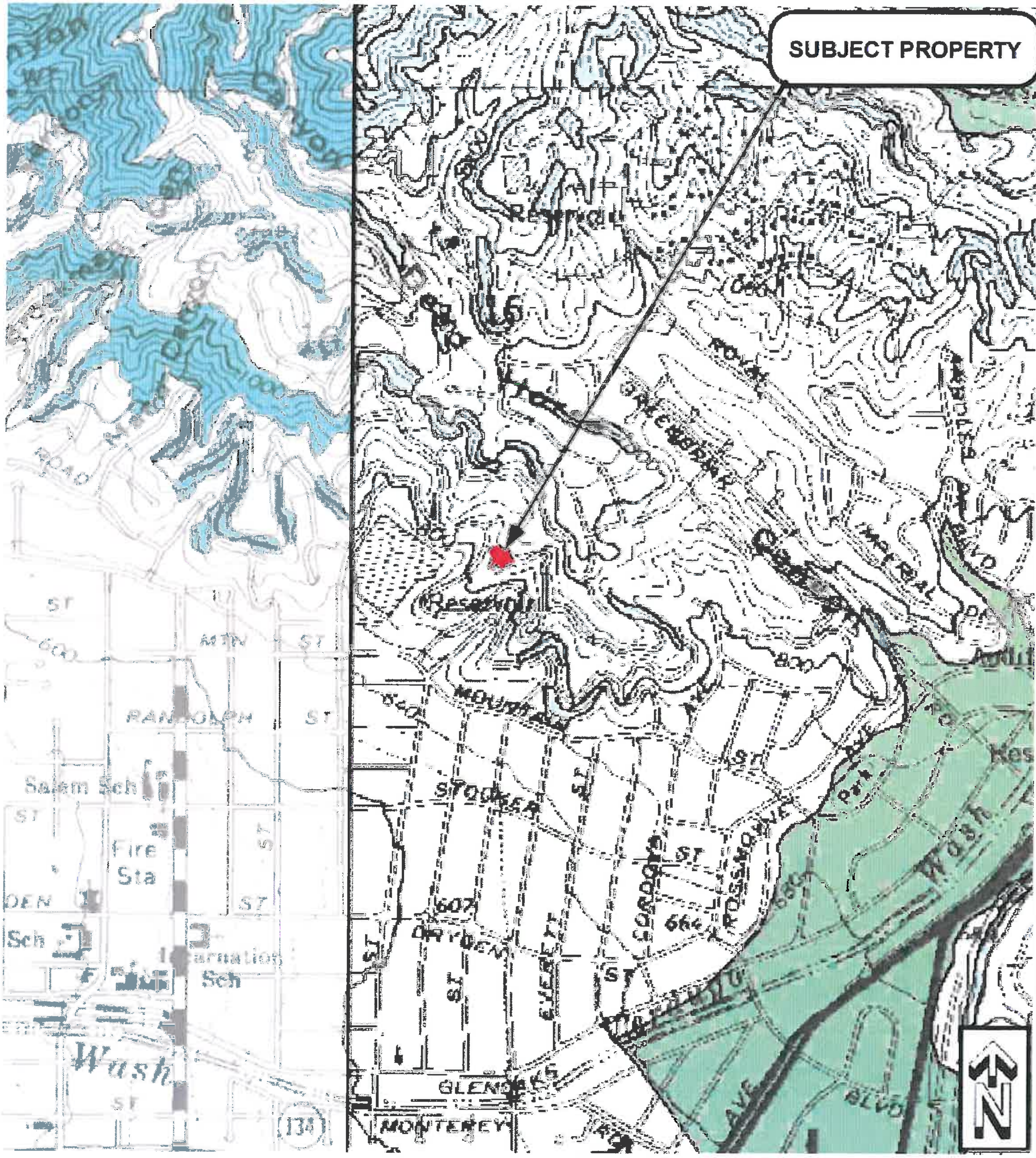
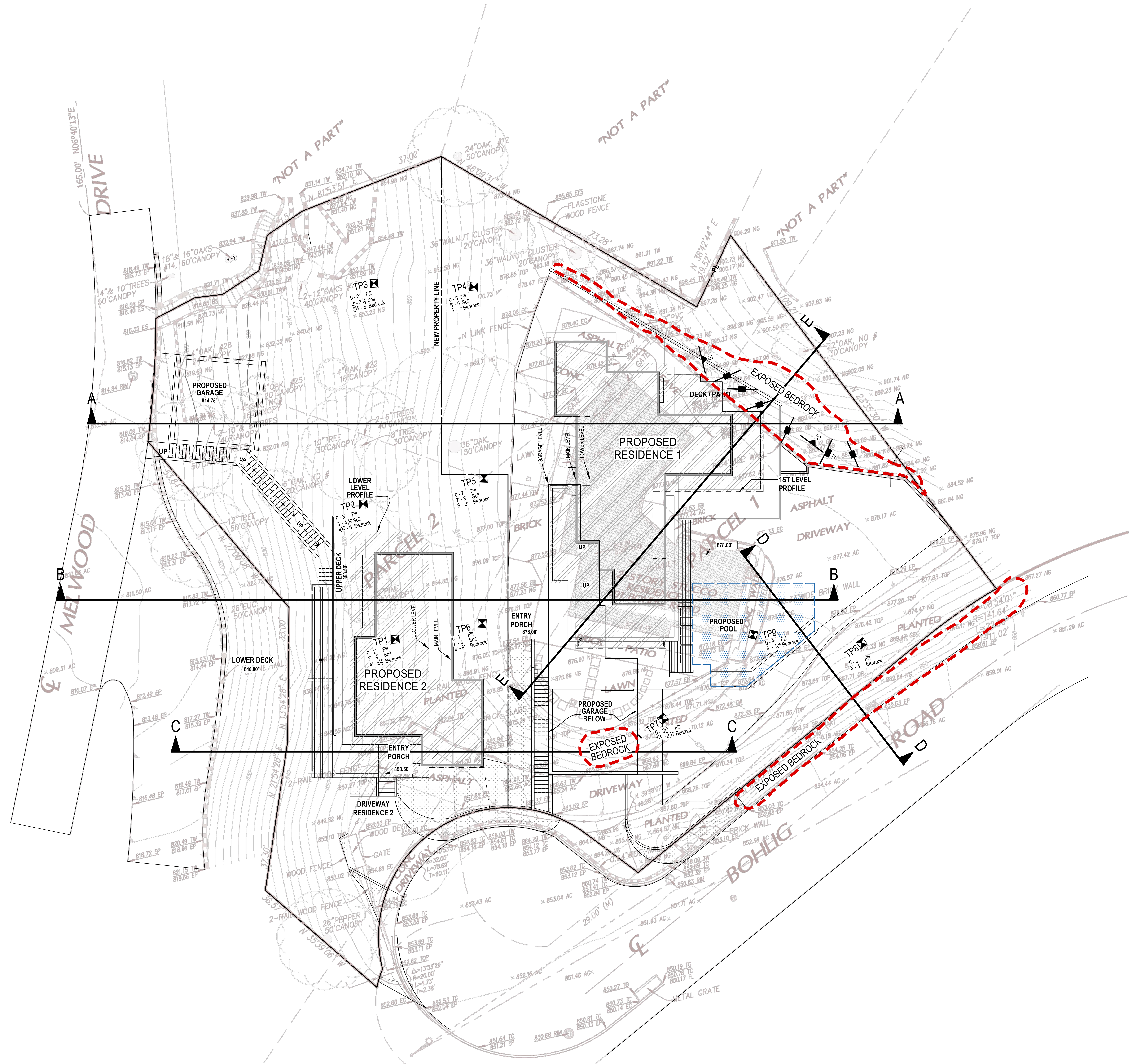
SEISMIC HAZARD ZONES MAP

BG: 22747 CLIENT: NEAGU

GEOLOGIST: GC SCALE: 1"=1,000'

Reference: State of California Seismic Hazard Zones, Burbank and Pasadena Quadrangles, California Geological Survey (1999)





GRAPHIC SCALE

10 7.5 5 2.5 0 5 10 20

(IN FEET)

LEGEND

TP9 — LOCATION AND NUMBER OF HAND-DUG TEST PIT

0 to 7' Fill — DEPTH OF FILL (FEET)
7' to 8' Soil — DEPTH OF SOIL (FEET)
8' to 9' Bedrock — DEPTH TO BEDROCK (FEET)

50 — STRIKE AND DIP OF FOLIATION

70 — STRIKE AND DIP OF JOINT

STRIKE OF VERTICAL JOINT

GEOLOGIC CONTACT

E E — LINE OF CROSS SECTION

REFERENCE: TOPOGRAPHIC SITE SURVEY DATED MARCH 15, 2017, PREPARED BY CHRIS D. NELSON & ASSOCIATES, INC. AND PROPOSED SITE PLAN CONCEPT DATED 6/07/2017, PREPARED BY MALEKIAN AND ASSOCIATES.

DECEMBER 08, 2017

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GLENDALE, CA 91206
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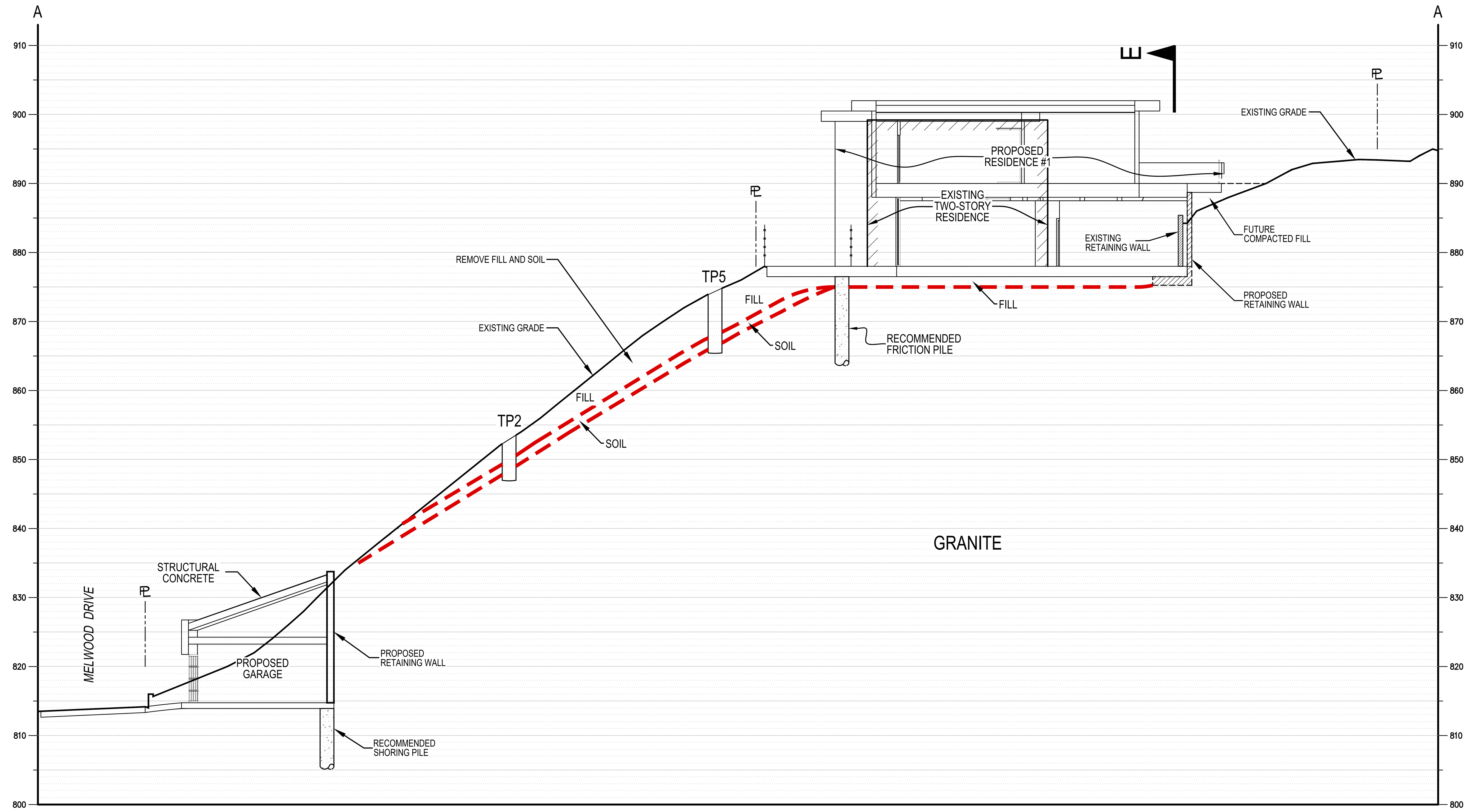
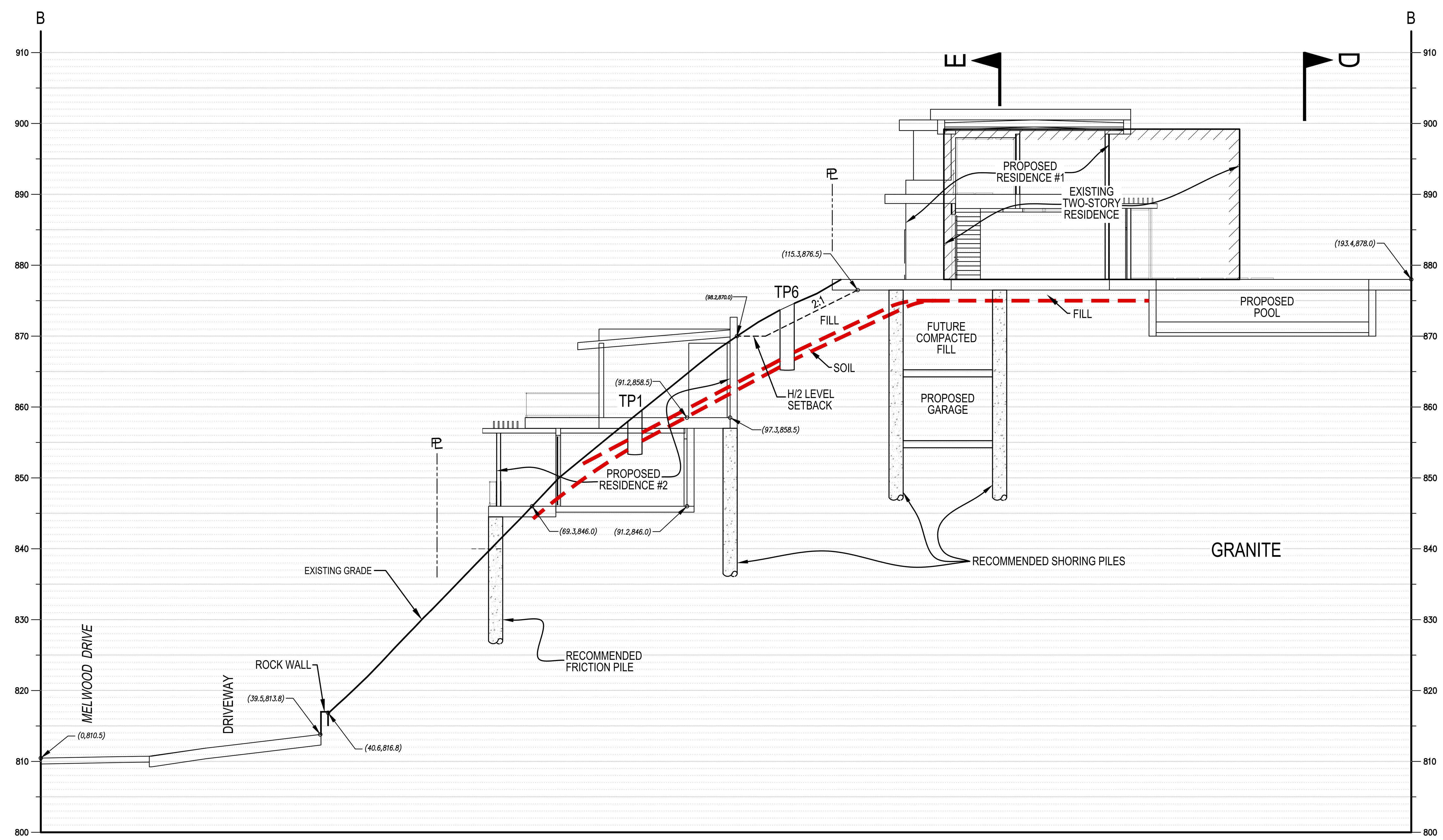
GEOLOGIC MAP

BG: 22747	NEAGU
CONSULTANT: GC/MP	SCALE: 1" = 10'
DRAWN BY: JTR	



SECTION A - A



SECTION B - B

DECEMBER 08, 2017

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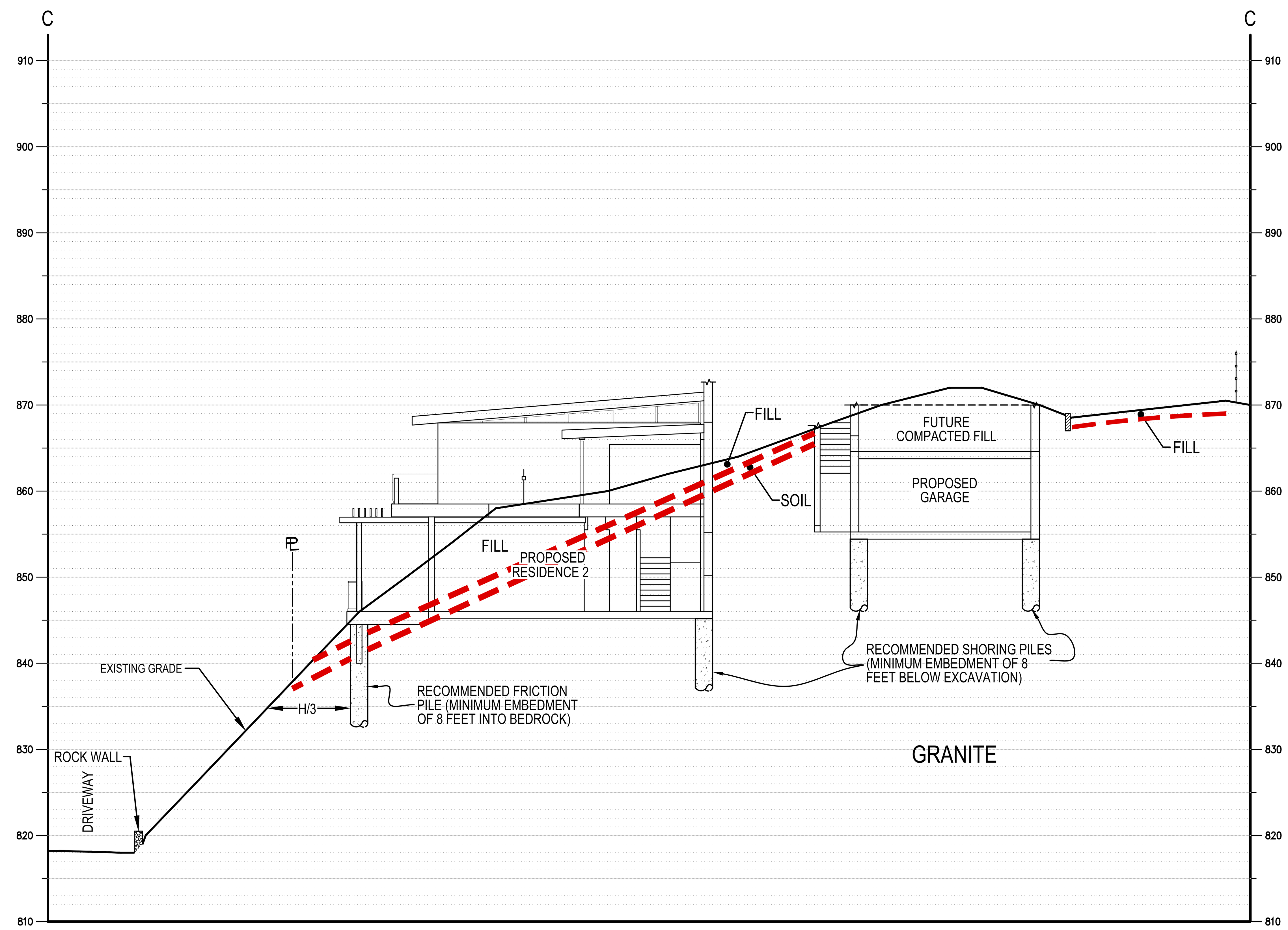
SECTIONS A & B

BG: 22747 NEAGU

CONSULTANT: GC/MP

DRAWN BY: JTR

SCALE: 1" = 10'



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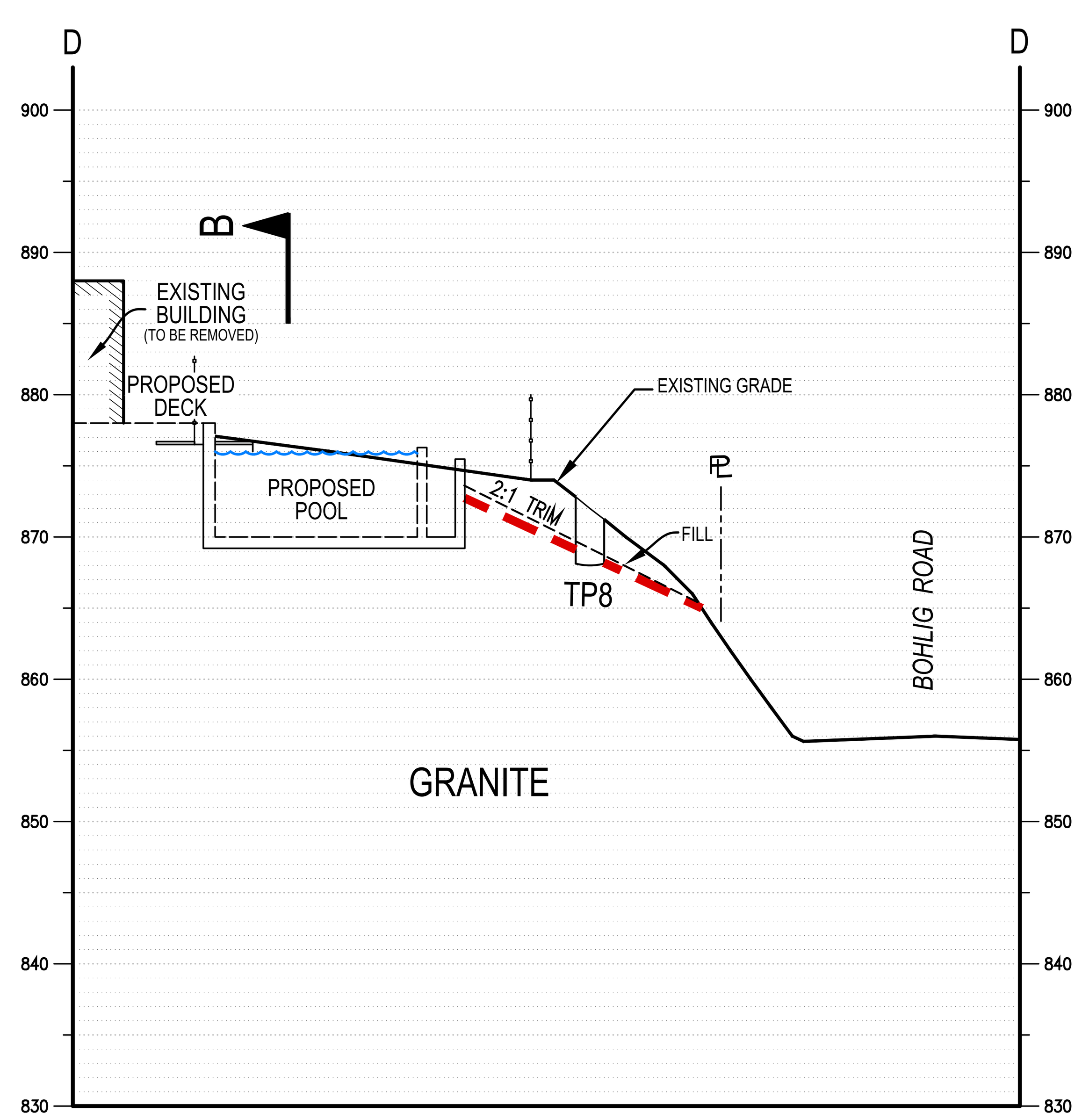

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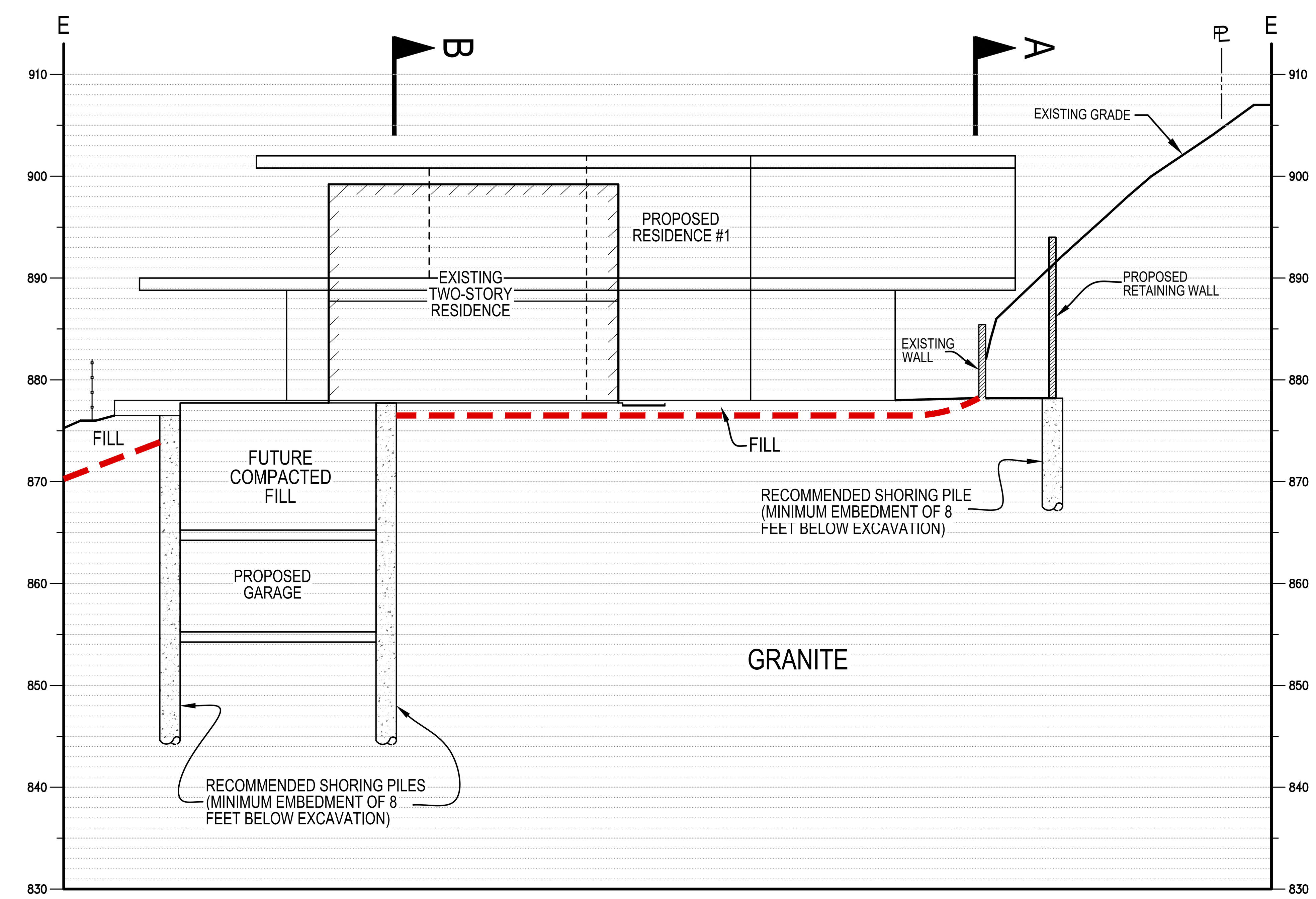

SECTION E - E

NOVEMBER 08, 2017

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SECTIONS C, D & E

BG: 22747 NEAGU

CONSULTANT: GC/MP

DRAWN BY: JTR

SCALE: 1" = 10'

CAD FILE: S:\LandProjects\22747 NEAGU\DWG\Geologic Map_12082017_recover.dwg PLOT DATE