MILLER PACIFIC ENGINEERING GROUP

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Building Solutions
152 Auburn Street
San Rafael, California

TOWN OF FAIRFAX

JUL 02 2014

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Attn: Mr. Mark Bruce and Mr. John Fraine

Re: Geotechnical Investigation
Mitigation for Potential Slope Instability
15 Wood Lane (APN 002-081-07)
Fairfax, California

Introduction and Purpose

This letter summarizes the results of our Phase 1 Geotechnical Investigation for mitigation of potential slope instability at 15 Wood Lane (APN 002-081-07) in Fairfax, California. A site location map is shown on Figure 1. The purpose of our Phase 1 services is to explore subsurface conditions, evaluate existing surface conditions and the potential for slope instability to affect the site, and develop conceptual options and associated design criteria for mitigation of any identified slope-stability related hazards

Project Description

Based on discussions with you and Mr. Michael Watkins, we understand the planned project includes remodeling and constructing a small addition to an existing 2-story single-family residence sited in the level, northern portion of the parcel. The central portion of the parcel consists of a small, relatively level vineyard, supported on the downslope side by a low retaining wall, while the southern portion of the parcel includes a series of level terraces, with intermediate slopes locally inclined in excess of 1:1 (horizontal:vertical), which have been graded into a north-facing hillside naturally sloping at about 3:1. The site plan shown on Figure 2 shows the location of the existing residence relative to the graded backyard terraces.

It is understood that the Town Engineer has determined the existing project geotechnical report, prepared by Earth Science Associates of San Rafael, California, did not adequately consider the potential for slope instability, originating either on- or off-site, to affect the residence and threaten human safety. Accordingly, the Town Engineer has required additional information, including geotechnical design criteria for potential mitigation options, be submitted in order that issuance of a building permit may be considered. It is also understood that the Town strongly encouraged consideration of conditions related to a previous debris-flow landslide which destroyed a nearby residence at 39 Wood Lane, just southwest of the subject property.

Scope of Services

The scope of our Phase 1 services is described in our proposal letter dated May 29, 2014 and includes review of available geologic mapping and geotechnical background data, including previous reports provided by the client, a detailed site reconnaissance for observation of existing conditions, exploration of subsurface conditions with 1 auger boring, laboratory testing for determination of the soils' pertinent engineering properties, development of design criteria

EXHIBIT # C

and conceptual options for mitigation of slope-instability hazards which may affect the residence, and preparation of this report.

It should be noted that the scope of our Phase 1 services is limited to consideration of slope-instability hazards only, and does not include consideration or evaluation of other project aspects, including geotechnical design for other project features or consideration of geologic hazards aside from slope-instability. Issuance of this report completes our Phase 1 scope of services. Future phases of work may include geo-civil design of new improvements, supplemental consultation, and/or geotechnical observation and testing during construction.

Regional Topography and Geology

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by abundant landsliding and erosion, owing in part to its typically high levels of precipitation and seismic activity.

The oldest rocks in the region are the sedimentary, igneous, and metamorphic rocks of the Mesozoic-age (225- to 65-million years old) Franciscan Assemblage. Within Marin County, a variety of sedimentary and volcanic rocks of Tertiary (1.8- to 65-million years old) and Quaternary (less than 1.8-million years old) age locally overlie the basement rocks of the Franciscan Assemblage. Tectonic deformation and erosion during late Tertiary and Quaternary time (the last several million years) formed the prominent coastal ridges and intervening valleys typical of the Coast Ranges province. The youngest geologic units in the region are Quaternary-age (last 1.8 million years) sedimentary deposits, including alluvial deposits which partially fill most of the valleys and colluvial deposits which typically blanket the lower portions of surrounding slopes.

The project site lies at the northern foot of a low ridge which bounds the east side of the Deer Park Creek Drainage in southern Fairfax, and which rises to elevations over +350-feet above mean sea level (MSL) southwest of the site. Regional geologic mapping[1] indicates the lower-lying, level portion of the project site is underlain by alluvial deposits, which the upland, sloping portions of the site are underlain by colluvial soils. Upland areas located offsite to the south of the property (higher on the ridgeline) are mapped as being underlain by Franciscan Mélange bedrock. A debris-flow scar is also shown in the general vicinity of the southern property boundary. A regional geologic map is shown on Figure 3.

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[1] Smith, T.C., Rice, S.J., and Strand, R.G. (1976), "Geology of the Upper Ross Valley and the Western part of the San Rafael Area, Marin County, California" *in* Geology for Planning in Central and Southeastern Marin County, California, California Department of Conservation, Division of Mines and Geology Open-File Report 76-2, Plate 1B, Map Scale 1:12,000.

Review of Reference Documents

We reviewed the existing geotechnical report[2] for the project, prepared by Earth Science Consultants of San Rafael, California. The report includes brief descriptions of existing site conditions, regional geologic and relative slope-stability mapping; a summary of subsurface exploration consisting of 6 apparent auger borings in the area of the residence and 4 "dynamic probe" borings in the rear yard/terraced slope area (none of which apparently extended to bedrock); and brief recommendations regarding site drainage and foundations. The report does not include any discussion or evaluation of commonly-considered geologic hazards which could affect the development (such as slope instability), and no laboratory testing was apparently performed for the project.

We have also reviewed a plan review letter issued by the Town Engineer, Mr. Ray Wrysinski, P.E., and dated March 20, 2014, indicating that ". . . a letter is required, bearing the signature and seal of the geotechnical engineer, which provides design criteria for stabilizing the hillside so the building site is reasonably protected from landslide damage or provides criteria for protection such as a debris barrier to keep the building site reasonably protected from landslide damage". A subsequent letter issued by Mr. Wrysinski and dated April 24, 2014, indicates that the response to his initial plan review is insufficient.

Finally, we reviewed photographs and documentation from our files and a Marin Independent Journal newspaper article[3] regarding the previous landslide at the nearby property at 39 Wood Lane, just southwest of the site, to gain a cursory familiarity with site conditions at the time of that landslide, which occurred following very heavy rains around New Year's Day of 2005.

Site Reconnaissance and Existing Conditions

We performed a site reconnaissance with Mr. Michael Watkins on May 28, 2014, to observe and document existing conditions. Currently, the parcel is developed with an existing 2-story, single-family residence in the northern, level portion of the property. An approximately 4-foot high concrete retaining wall is located about 10-feet south of the residence, with an approximately 4-foot high wooden retaining wall located about 30-feet farther south. The area between the walls is level and developed as a small vineyard. The concrete wall appears to be performing relatively well, with no significant distress noted, while the wooden wall shows a slight lean and some minor distress. South of the wooden wall, steep, south-facing slopes form a shallow swale which drains toward the residence below. Within the property boundaries, the south-facing swale has been graded to form a series of 2- to 3-foot wide level benches, with intermediate cuts ranging in height from about 3- to 10-feet and generally inclined between about 0.5:1 and 1:1. Beyond the eastern and western property boundaries, natural slopes on adjacent properties are typically inclined between about 2:1 and 3:1. The terraces are generally vegetated with mature fruit trees, with some large, approximately 50-foot tall Douglas fir trees on the uppermost level bench near the southern property line.

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[2] Earth Science Consultants (2014), "Geotechnical Investigation, Older House Remodel, Improvements and Addition, 15 Wood Lane, Fairfax, California", Job No. 14-7038.

[3] Wolfcale, J. (2006), "Surveying the Damage – Mudslide Rips Fairfax Home in Deer Park", Marin Independent Journal, San Rafael, dated January 3, 2006, accessed at http://www.marinij.com/ci_3367637 on June 17, 2014.

Upslope of the terraced yard area, natural slopes are somewhat flatter, inclined at about 3:1, and expose similar colluvial deposits as observed within the project site beneath a thin veneer of organic detritus. Areas upslope of the property are heavily vegetated with thick stands of oak, bay, and madrone trees.

Soils exposed in the terrace slopes/backcuts and on natural slopes south of the site are typically composed of slightly porous, medium-stiff top stiff sandy clay colluvium with lesser angular gravel- to boulder-size fragments of light brown arkosic sandstone. Highly weathered, crushed, weak sandstone underlies the soil mantle and is locally exposed near the base of some cuts nearer the southern (upslope) property boundary.

During our reconnaissance, we observed a shallow depression just upslope of the southern property boundary and located coincident with the centerline of the natural swale, which may be a historic debris-flow scar. The wood-and-wire fence marking the southern property line appears to have been previously undermined and re-supported with metal pipes at this location, suggesting minor ongoing erosion of the scar or a more recent, likely small, debris-flow. We also noted some minor ground cracking and sloughing/raveling of soils on the steeper terrace faces which appears due to a combination of steep slope inclinations and minor "creep" owing to slightly plastic clayey surface soils. However, no other evidence suggestive of significant recent or imminent instability, including fresh scarps, debris piles, leaning trees, or ground cracking, was observed either within the site or on slopes above the site, extending nearly to the crest of the ridge about 100 linear feet above the upslope (southern) property line. Surface reconnaissance beyond (upslope of) this point was not performed due to very heavy brush and vegetation which largely obscures subtle topographic features.

Also during our reconnaissance, we briefly observed conditions at the 39 Wood Lane property. From review of the Marin Independent Journal article referenced above, we understand a debris-flow landslide impacted and heavily damaged a 2-story duplex which was sited near the Wood Lane frontage. Hillside geometry behind the former duplex was noted during our reconnaissance to differ somewhat from conditions at 15 Wood Lane, in that a prominent, narrow, deeply-incised drainage emanates and discharges directly behind the former duplex at 39 Wood Lane. At 15 Wood Lane, it appears that the apparent swale upslope of the residence has been more or less created by historic grading activity within the property boundaries. Evidence of a pre-existing, well-developed natural drainage which may act as a debris-flow source, such as exists at 39 Wood Lane, is generally lacking at 15 Wood Lane. In addition, side scarps at 39 Wood Lane expose several feet of well-developed, dark-colored, organic-rich soils which would be prone to instability when subjected to heavy rainfall on steep slopes. Conversely, surface conditions at 15 Wood Lane suggest relatively thin, predominantly clayey colluvial and residual soil deposits exist over relatively shallow bedrock.

Subsurface Exploration and Laboratory Testing

Subsurface exploration for the project included one auger boring drilled with portable, hydraulic-powered equipment and one hand-augered boring performed on June 5, 2014 at the approximate location shown on Figure 2. Soil and rock materials encountered were examined and logged by our Engineering Geologist, and samples collected from select intervals for laboratory testing. The terms and methodology used in classifying earth materials are briefly described on the Soil and

Rock Classification Charts, Figures A-1 and A-2, respectively. Exploratory boring logs are shown on Figures A-3 and A-4.

Laboratory testing for the project included determination of moisture content, dry density, plasticity index/Atterberg limits, and percentage of particles passing the No. 200 sieve (fines content) in general accordance with applicable ASTM standards. Results of the moisture content, dry density, and fines content tests are shown on the exploratory boring logs, Figures A-3 and A-4, while plasticity index results are presented on Figure A-5. The subsurface exploration and laboratory testing program is discussed in greater detail in Appendix A.

Subsurface exploration performed previously by Earth Science Associates included performance of 6 apparent auger borings at the approximate locations shown on Figure 2. Auger borings ranged in depth from about 2.5- to 9.0-feet below the ground surface and generally encountered medium dense to dense sandy soils, although about 8-feet of loose sandy materials were encountered in Boring 4. None of the borings encountered bedrock, nor do the exploratory logs indicate the soils' provenance, although they are generally assumed to be colluvial in origin based on local topography and apparent historic grading. Previous exploratory boring logs are included for reference in Appendix B.

Subsurface Conditions

The results of our subsurface exploration generally confirm the regionally-mapped geology. Boring 1 was drilled on the uppermost graded bench, near the southern property boundary, as shown on Figure 2. Boring 1 encountered approximately 1.5-feet of colluvial deposits composed of medium-dense silty sand, underlain by about 3-feet of residual soils composed of medium-dense clayey sand with lesser angular sandstone fragments. At a depth of 4.5-feet, Boring 1 encountered weak, moderately hard, moderately weathered sandstone bedrock. Harder drilling was noted at a depth of about 11.5-feet, and at 13.0-feet, we noted occasional 2- to 4-inch thick interbeds of dark gray, highly sheared siltstone. Boring 1 was terminated at a maximum explored depth of 16.5-feet below the ground surface.

Boring 2 was excavated using a manually-operated bucket auger (hand auger) near the base of the slope as shown on Figure 2, and encountered about 4.5-feet of medium dense clayey sands colluvial soils underlain by dark gray, highly- to completely-weathered siltstone with lesser sandstone bedrock. Boring 2 was terminated at a maximum explored depth of 5.0-feet.

Geologic Hazards Evaluation

As previously noted, the scope of this report is limited to evaluation of potential slope-instability and related hazards, such as seismic shaking, erosion, and expansive soils. Other hazards, such as flooding, liquefaction, and others, have therefore not been considered. Slope instability and related hazards are discussed in detail below.

Seismic Shaking

Strong seismic ground shaking may induce slope instability, particularly where low factors of safety currently exist (typically where weak geologic materials coincide with very steep slopes or human activity has resulted in loss of support, such as by excavation at the toe of slopes). The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical attenuation relations provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, and probable peak ground accelerations are summarized in Table B.

TABLE B
ESTIMATED PEAK GROUND ACCELERATION
FOR PRINCIPAL ACTIVE FAULTS
15 Wood Lane
APN 002-081-07
Fairfax, California

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (kilometers)	Median Peak Ground Acceleration (g)(1)
San Andreas	8.0	11	0.34
San Gregorio	7.4	12	0.28
Hayward	7.3	19	0.19
Rodgers Creek	7.3	25	0.15
West Napa	6.6	36	0.08

1. Caltrans ARS Online (Acceleration response spectra calculator tool), Version 2.3.06, http://dap3.dot.ca.gov/ARS_Online/, accessed June 18, 2014
2. Values determined using Vs^{30} = 270 m/s for Site Class "C" (Very Dense Soil and Soft Rock) in accordance with 2013 California Building Code.

The calculated bedrock accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc...) can influence the actual ground surface accelerations. Compliance with provisions of the California Building Code (CBC) should result in structures that do not collapse in an earthquake; however, damage may still occur and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their proximity, the San Andreas, San Gregorio, Hayward, and Rodgers Creek Faults present the highest potential for severe ground shaking.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Mitigation measures should include designing any future improvements and structures in accordance with the most recent (2013) version of the California Building Code. Recommended CBC seismic coefficients and spectral* *accelerations are provided in the Conclusions and Recommendations section of this report.*

Erosion
Sandy soils on moderate slopes and clayey soils on steep slopes are susceptible to erosion, particularly when exposed to concentrated surface water flow. Significant erosion, particularly at the base of steep slopes, can result in increased susceptibility to slope instability. During our site reconnaissance, we did observe evidence of minor surface erosion, primarily on natural slopes south (upslope) of the site and on the steeper terrace cut slopes. We did not observe evidence of significant subsurface erosion or "piping". Therefore, the risk of damage to improvements or heightened susceptibility to slope instability as a result of erosion is moderate to high.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Minor erosion of the onsite terrace slopes is not anticipated to significantly impact the residence, although periodic cleanup of soil debris at the base of slopes should be expected. Erosion of natural slopes south of the property is more likely to result in significant instability that may affect the residence in the form of debris flow landslides. Therefore, surface and subsurface drainage should be provided around the upland areas of the property so as to minimize the potential for significant erosion. Recommendations for geotechnical site drainage are provided in the Conclusions and Recommendations section of this report.*

Expansive Soils
Moderate and highly plastic silts and clays, when located near the ground surface on moderate to steep slopes, can exhibit expansive characteristics (shrink-swell) in response to wetting/drying cycles which in turn leads to slow, progressive downslope movement known as "slope creep". We observed local desiccation cracking of near-surface soils during our reconnaissance, which is indicative of soil expansion. Additionally, laboratory test results indicate the near-surface soils are moderately plastic, which is also indicative of moderate expansion potential.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Surface and subsurface drainage improvements should be provided to reduce the frequency/magnitude of soil wetting/drying cycles and associated expansive behavior and slope creep. Any retaining structures should be designed to account for moderate soil expansion potential. Recommendations for site drainage and retaining wall construction are presented in the Conclusions and Recommendations section of this report.*

Landsliding and Slope Instability
The project site is located in moderately- to steeply-sloping terrain with a history of landsliding and slope instability. Slope instability is common where steep slopes are underlain by weak geologic materials, and is often exacerbated by soil saturation due to rainfall. Human grading activities, such as excavations at the base of slope or placement of fill above steep slopes, can also contribute to instability.

Regional geologic mapping indicates the presence of two debris-flow scars in the immediate vicinity of the project site as shown on Figure 3. Relative slope-stability mapping[4] performed simultaneous with regional geologic mapping in the mid-1970's indicates that the project site, along with the rest of the prominent ridgeline which rises south of the site and extends further south along the east side of Wood Lane, lies within relative slope-stability "Zone 3", defined as "areas where the steepness of slopes approaches the stability limits of the underlying geological materials". It is noted that the prominent drainage upslope of 39 Wood Lane, which was subject to the aforementioned 2005 debris-flow, is mapped as being within "Zone 4", defined as "The least stable category . . . these slopes should be considered naturally unstable, subject to potential failure even in the absence of activities and influences". Mapping performed following the severe storms of January, 1982, which caused widespread landsliding throughout Marin County and the greater North Bay Area, does not indicate any debris flows in the immediate vicinity of the project site triggered by that storm[5].

We are currently unaware of any widely-accepted method for predicting the potential for landslide mobilization, or for mobilization of various modes of instability. However, in general, soils having high void ratios and low liquid limits in conjunction with relatively low density and plasticity are more susceptible to liquefaction and debris-flow development, while soils having higher plasticity and density, coupled with lower void ratios and high liquid limits will be less prone to debris flows and more susceptible to slope "creep" and slower-moving modes of transport. Based on our laboratory testing, the upper 1.5- to 2.0-feet of colluvial soils, consisting of porous silty sand with a lower liquid limit, are judged prone to debris-flow development. Residual soils, which were encountered between depths of about 2.0- and 4.5-feet, have somewhat higher liquid limits and higher plasticity indices, would be less susceptible to debris-flow development, although they would be more prone to slope "creep" on steep slopes, owing to their moderate to high plasticity and associated expansion potential.

Due to gently sloping grades, we judge that there is generally little risk of instability in areas north of Boring 2 as shown on Figure 2. The steeper, terraced areas between the location of Boring 2 and the south property line are judged to have moderate to high potential for slope instability, although since the upper mantle of porous, debris flow-prone colluvium has been largely removed by previous grading, instability in this area is anticipated to consist mainly of "creep" and more localized sloughing and raveling. These modes of instability may result in the periodic need for debris removal from the base of slopes, but are not likely to significantly impact the residence. We judge the likelihood of debris flows originating within the property boundaries is generally low.

The most significant threat to the residence is posed by debris flows originating high on the slopes beyond the southern property line, where thicker deposits of porous, debris flow-prone colluvial soils exist. Because of the high anticipated flow velocities associated with a large quantity of material and considerable elevation change from the upper slope to the residence, we judge there

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[4] Smith, T.C., Rice, S.J., and Strand, R.G. (1976), "Interpretation of the Relative Stability of Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California", California Department of Conservation, Division of Mines and Geology Open-File Report 76-2, Plate 2B, Map Scale 1:12,000.

[5] Davenport, C. (1984), "Locations of Major Slope Failures in Eastern marin County, California during the 1981-1982 Winter" *in* An Analysis of Slope Failures in Eastern Marin County, California, Resulting from January 3 & 4, 1982 Storm, California Department of Conservation, Division of Mines and Geology Open-File Report 84-22SF, Map Scale 1:62,500.

is a moderate to high potential for slope instability originating offsite to result in damage to the residence and/or threaten the safety of its occupants.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Localized sloughing and raveling of terrace slopes within the property is unlikely to affect the residence, given the relatively small quantities of soil involved and the significant setback provided for the residence from the terraced yard area. Some occasional debris cleanup at the base of slopes may be anticipated, but no mitigation is required for this mode of instability.*

Any or a combination of several mitigation measures for more significant instability, consisting of debris flows or avalanches originating off-site, could be considered at the site, including grading repairs (reinforced earth buttress), retaining walls, debris barriers or debris catchment walls, and surface or subsurface drainage improvements, at varying levels of complexity and cost. Conceptual mitigation measures and associated geotechnical design criteria for more significant debris-flow landsliding are discussed in detail in the Conclusions and Recommendations section of this report.

Conclusions and Recommendations

Based on our review of reference materials, site reconnaissance, and experience with similar projects, it is our professional opinion that the residence at 15 Wood Lane is at moderate risk to be affected by future debris-flow or debris avalanche landslides originating on natural slopes south of the property. Mitigation is recommended in the form of either retaining structures or debris-catchment structures, in combination with both surface and subsurface drainage improvements. Terraced retaining walls or grading-type repairs may also be considered as feasible options, although our experience suggests that they would be prohibitively expensive and unnecessary at this site. Geotechnical site drainage recommendations, conceptual mitigation options, and associated design criteria are discussed in detail in the following sections.

Geotechnical Site Drainage Recommendations

Since soil saturation commonly contributes to slope instability, providing adequate surface and subsurface drainage provisions is paramount to improving overall stability. Since the highest risk is posed by instability originating offsite to the south, it is recommended that the adjacent property owner, believed to be to Town of Fairfax, be consulted regarding the potential for construction of new drainage improvements by mutual agreement, creation of an easement, or other means.

We recommend that, at a minimum, subdrainage be provided on the slope above the southern property line, generally consisting of perforated pipes in a gravel-filled trench as detailed on Figures 4 and 5. Drains should be laid out in a "herringbone" shape, with the main collector pipe running down the fall line of the slope, roughly in the center of the swale. At the base of the subdrain system, near the south property line, the perforated pipe should transition to a solid discharge pipe and the gravel trench should be provided with a compacted clay or controlled-density fill (CDF) check dam. The solid discharge line should extend downslope and convey runoff into an established storm drain system.

We also recommend that a concrete v-ditch be constructed along the southern property line to capture any surface runoff that may otherwise cause erosion of contribute to saturation of the subsurface within the property limits. The ditch should be constructed in accordance with Marin County standards and either be extended downslope to discharge into the storm drain system or be provided with a catch basin at the low end and a solid discharge pipeline which conveys water to the nearest storm drain. Concrete v-ditches may also be considered for improved performance along the level graded terraces within the property to reduce the risk of localized sloughing and raveling due to erosion or "creep" of the surface soils on the terrace slopes

Drainage improvements alone will somewhat reduce the risk of slope instability by reducing the likelihood of soil saturation; however, instability may still occur even with optimal soil moisture conditions due to removal of toe support by previous grading and generally over-steepened slopes within the property. Therefore, it is recommended the drainage improvements be provided in conjunction with one of the following mitigation options.

Conceptual Mitigation Option 1 – Integrated Debris Catchment Wall

Option 1 would entail integrating a debris catchment structure into the residence itself, where the rear exterior wall of the residence is designed as a catchment wall capable of withstanding debris impact as schematically shown on Figure 6. In this event, the rear wall of the residence would likely need to be constructed of reinforced concrete, and window/door placement and materials will require careful consideration by the project Structural Engineer and Architect. If windows and doors will remain as part of the rear catchment wall, cleanup of debris within the residence should be anticipated following a debris flow event, and no bedrooms should be allowed along the rear wall on the ground floor.

If this option is chosen, the rear exterior wall of the residence should be designed by the project Structural Engineer to withstand an equivalent fluid pressure of 200 pcf for a minimum height of 10-feet. Wall foundations should consist of drilled, cast-in-place concrete piers designed in accordance with the criteria shown in Table A.

TABLE A
DEEP FOUNDATION DESIGN CRITERIA – INTEGRATED CATCHMENT WALL
15 Wood Lane
APN 002-081-07
Fairfax, California

Minimum Diameter:	18 inches
Skin Friction[1]:	
Medium-Dense Sandy Alluvial Soils:	750 psf
Weathered Bedrock:	2,000 psf
Lateral Passive Resistance[2,3,4]:	
Medium Stiff/Dense Alluvium:	300 pcf
Weathered Bedrock:	450 pcf

Notes:
(1) Uplift resistance is equal to 80% of the total skin friction.
(2) Equivalent Fluid Pressure, not to exceed 10 times value in psf.
(3) Apply values over effective width of 2 pier diameters.
(4) Ignore upper 12 inches unless confined by asphalt or concrete.

Although this is likely the least expensive option, costing only the additional engineering and construction materials required for additional fortification of the rear exterior wall, we understand that due to architectural and other considerations, this is likely not a desirable solution. This option would result in a reduced threat to the safety of the building's occupants during a landslide, but would not affect the likelihood of slope instability or the potential for damage to areas not protected by the rear exterior wall.

Conceptual Mitigation Option 2 – Freestanding Debris Barrier

An alternative to an integrated catchment structure would be to construct a freestanding debris barrier, which would be placed on the slope between the landslide source area and the residence, and also be designed to withstand the force of impact associated with a future landslide and protect the residence from impact. The debris barrier is an engineered system consisting of steel vertical supports with breakaway bases and a chain-link mesh "curtain" supported by the vertical posts and horizontal wire ropes supports at the top and bottom. The wire ropes incorporate a brake-ring system allowing for rapid deceleration of the slide mass, and are anchored using drilled and grouted rock anchors. The debris barrier is designed to deform elastically during slide impact and sustain repairable damage. Following a slide, the debris barrier may be re-used by removing captured slide debris and repairing broken upright supports and any other damaged components. The debris barrier would be designed in-house by the manufacturer, and the exact design will depend on the chosen placement. Based on our assessment of slope stability risks above, we judge the most effective debris barrier location would be high on the slope, near the southern property line as shown on schematically on Figure 7, where the barrier would provide protection from the most acute debris-flow threat from the thicker colluvium located upslope of the site. This barrier would also be founded on a pier-and-grade beam foundation system intended to strengthen and support the most instability-prone soils, resulting in appreciable improvement in overall slope stability, with the barrier's vertical supports attached directly to the top of the grade beam. This barrier would likely be on the order of about 75-feet long and about 6-feet high.

Foundations for the debris barrier should be designed in accordance with the criteria shown in Table B. Note that drilled piers should be designed to retain the upper few feet of surface soils and resist an equivalent fluid pressure of 130 pcf to account for saturated conditions during a landslide.

TABLE B
GEOTECHNICAL DESIGN CRITERIA – FREESTANDING DEBRIS BARRIER
15 Wood Lane
APN 002-081-07
Fairfax, California

Grade Beam	
Minimum Width:	24 inches
Minimum Depth:	24 inches
Drilled Piers	
Minimum Diameter:	18 inches
Skin Friction[1]:	
Colluvial/Residual Soils:	Ignore
Weathered Bedrock:	2,000 psf
Lateral Active Earth Pressure[2]:	
Colluvial/Residual Soils:	130 pcf
Lateral Passive Resistance[2,3,4]:	
Medium Stiff/Dense Alluvium:	Ignore
Weathered Bedrock:	200 pcf

Geotechnical Material Properties

	Phi[5]	C (psf)[6]	Gamma (pcf)[7]
Colluvium/Residual Soil	25	200	130
Bedrock	38	2,000	140

Rock Anchors	
Min. Diameter Grouted Holes:	6 inches
Skin Friction:	
Colluvium/Residual Soil	Ignore
Bedrock:	1,500 psf

Notes:
(1) Uplift resistance is equal to 80% of the total skin friction.
(2) Equivalent Fluid Pressure, not to exceed 10 times value in psf.
(3) Apply values over effective width of 2 pier diameters.
(4) Assumes sloping conditions below wall.
(5) Angle of Internal Friction, effective stress, unitless
(6) Apparent (effective) Cohesion, for seismic conditions 500 psf of additional cohesion may be included
(7) Unit Weight of Soil/Bedrock
(8) Soil nails should be designed for load-testing up to 150% of the design load. Load testing to be performed in general accordance with the procedures recommended by the Post-Tensioning Institute (2004).

In addition to providing protection for the residence from future impact by debris-flows, this barrier would improve stability by retaining the portion of slope above the south property line. Construction costs for drilled-pier and grade beam foundation systems are typically on the order of $1,500 per linear foot, while debris barriers typically cost on the order of $125 per square foot. Assuming a 15% construction cost surcharge due to limited access and difficult slope conditions and inclusion of "soft costs" on the order of 15% (for engineering, design review, permitting, etc.), this option is anticipated to cost between about $100,000 and $200,000 for a 75-foot long, 6-foot high barrier at the top of the slope.

Conceptual Mitigation Option 3 – Combined Retaining Wall/Debris Barrier
Option 3 would include constructing and backfilling a new retaining wall near the base of the slope as shown on Figure 8. A new, shorter debris barrier would then be constructed atop the wall. The new wall would be on the order of 10- to 12--feet high at the highest point, tapering in an arcuate shape to the property lines on either end, and would be backfilled with compacted soil. The new wall would provide additional lateral support to the base of the slope and reduce the risk of instability originating within the property, while the debris barrier and level backfilled area would provide some runout area and additional protection for the residence from slides originating offsite. The retaining wall should be designed to resist an equivalent fluid pressure of 130 pcf. Design of retaining wall foundations and debris barrier rock anchors should utilize the criteria shown above in Table B, and the debris barrier itself would be designed for the site conditions by the manufacturer.

Based on an estimated construction cost of $125 per square foot for a 90-foot long, 8-foot average height wall and a 90-foot long, 6-foot high debris barrier, Option 3 is anticipated to cost on the order of about $180,000, including soft costs for engineering and permitting on the order of 15%.

Other Conceptual Options
There are other approaches which may considered to lessen the risk of damage or injury due to slope instability. Additional options include a reinforced earth buttress, which would essentially restore pre-existing grades to match those on adjacent properties and would result in significant improvement in overall stability. This would involve excavating a keyway behind the house and compacting fill to restore an approximate slope angle of 2:1 to 3:1. The level rear yard area would be significantly reduced, and the existing vines and fruit trees likely would need to be removed. This option is also anticipated to be expensive due to marginal access for heavy earth-moving equipment and the cost associated with importing large quantities of soil fill.

Another option that may be considered is a series of terraced retaining walls. A series of walls on the order of 5- to 8-feet high and separated from one another by about 10-feet could be constructed and backfilled to create a series of level runout areas as well as provide additional lateral support to the base of the slope. This would result in an improvement in overall stability as well as a reduction in the risk of damage or injury. However, this option is anticipated to be more expensive than those discussed in detail above, as it would require essentially the same foundation and wall construction as Option 3 (albeit without the debris barrier) many times over. If more detailed information regarding mitigation options not discussed in detail is desired, we should be contacted to provide additional consultation.

Supplemental Services

Depending on the Town's requirements and Owner's desired level of risk following their review of this report, supplemental services could include additional consultation, geo-civil design for new slope-stabilization, debris-impact protection, and/or drainage improvements at the site, and geotechnical observation and testing during construction. We will be happy to submit a scope and budget proposal for any desired supplemental service upon request.

It should be noted that the estimated costs listed are based on bids received over the last several years for similar projects. Costs are subject to significant variation based on a variety of site-specific and "global" economic factors, and we strongly suggest consulting with a qualified, licensed Contractor specializing in hillside foundation and retaining wall construction if more refined estimates are required. If needed, we can provide contact information for several Contractors who, in our experience, are suitably qualified and capable to perform the work.

We trust that this letter addresses your concerns and includes the information you require at this time. Should there be any questions or concerns regarding our investigation, please do not hesitate to contact us.

Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY

CERTIFIED ENGINEERING GEOLOGIST
MICHAEL JEWETT
No. CEG 2610
EXP. 1-31-15
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
SCOTT A. STEPHENS
2398
Exp. 6/30/15
GEOTECHNICAL
STATE OF CALIFORNIA

Mike Jewett
Engineering Geologist No. 2610
(Expires 1/31/15)

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/15)

Attachments: Figures 1 through 8; Appendices A and B

SITE: LATITUDE, 37.9786°
LONGITUDE, -122.5927°

SITE LOCATION

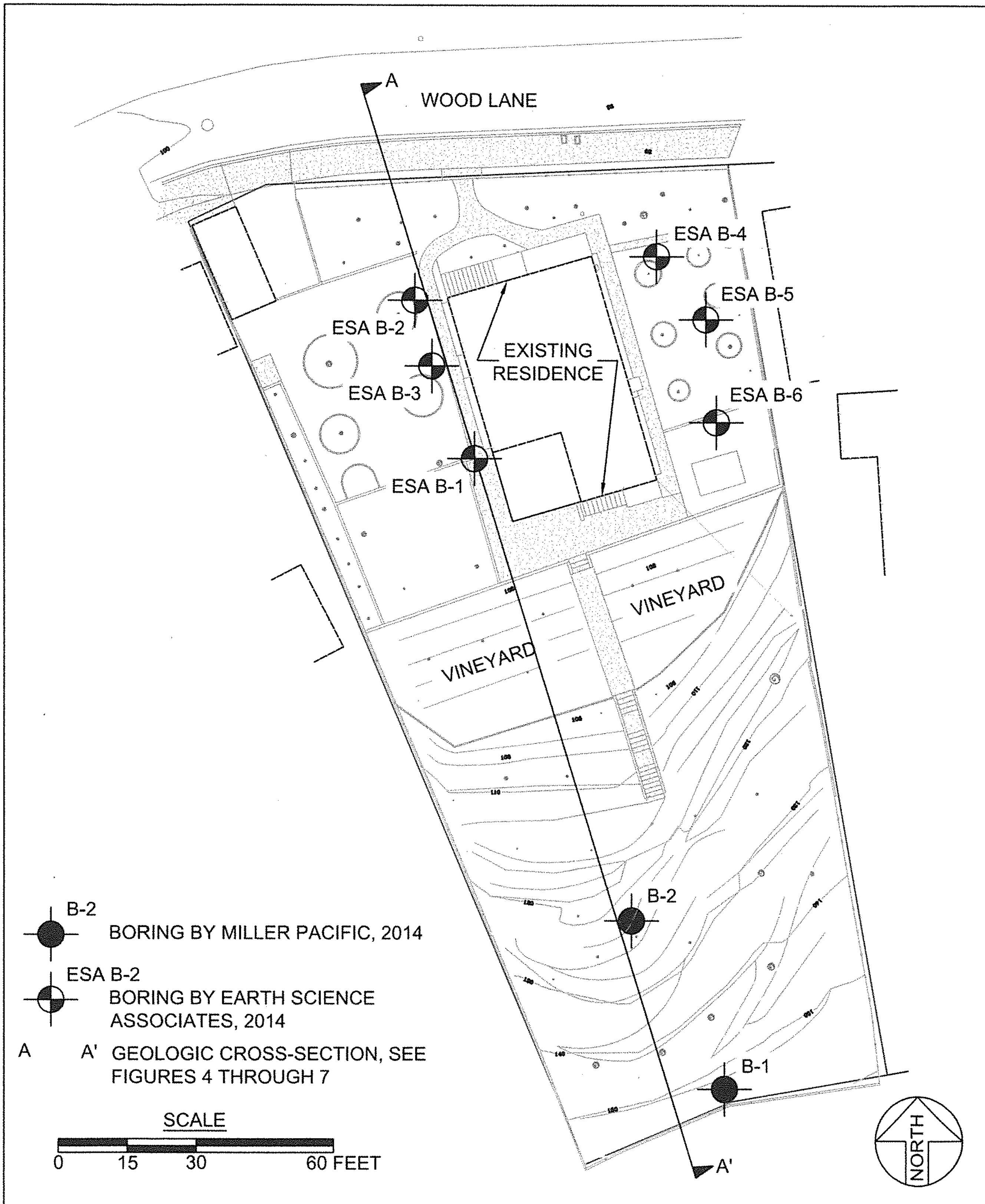
N.T.S.

NORTH

REFERENCE: Google Earth, 2014

Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE LOCATION MAP 15 Wood Lane APN 002-081-07 Fairfax, California Project No. 2038.001 Date: 6-18-14	Drawn MFJ Checked	1 FIGURE

FILE: 2038.001SLM.dwg



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SITE PLAN

15 Wood Lane
APN 002-081-07
Fairfax, California

Project No.
Date:

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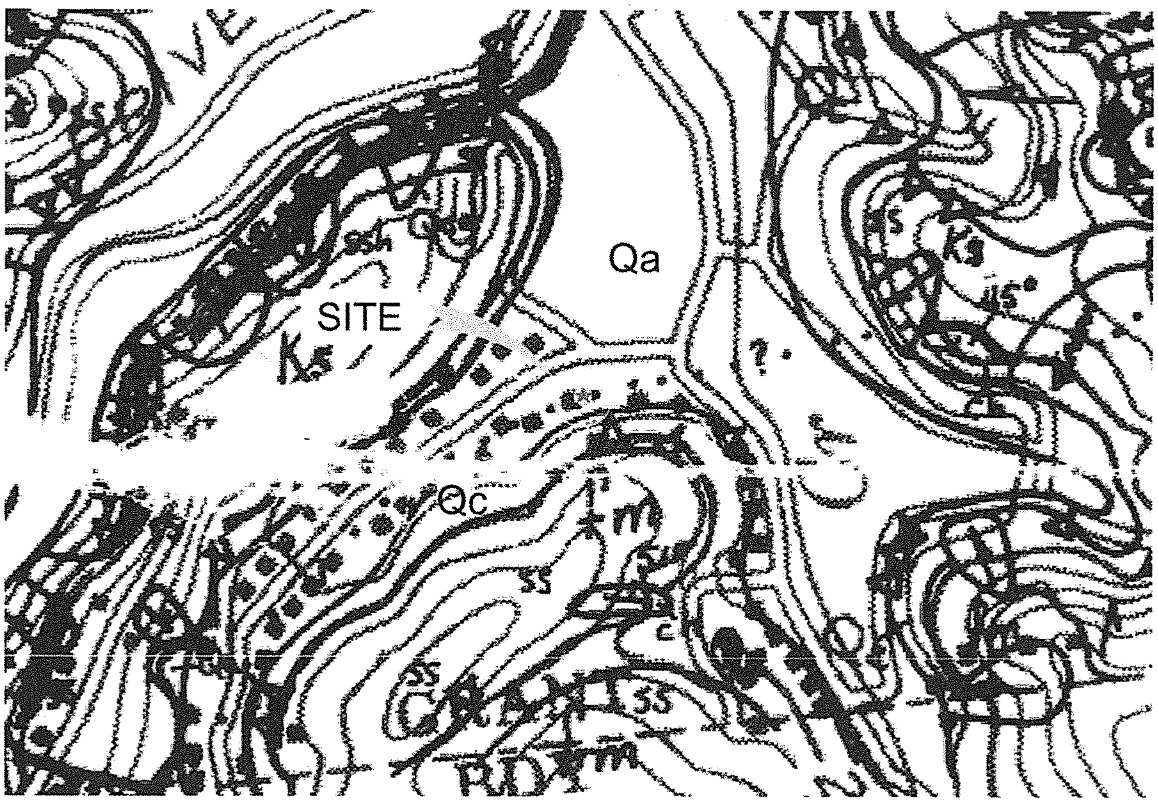
2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

LEGEND:

Qa Alluvium - Unconsolidated clays, silts, sands and gravels deposited in stream channel, terrace, and floodplain environments

Qc Colluvium - Unsorted sands, silts, clays and weathered rock fragments accumulated on or at the base of slopes by natural gravitational or slope wash processes

Fm Franciscan Melange - A tectonic mixture consisting of small to large masses of resistant rock types, predominantly sandstone, greenstone, chert, and serpentinite embedded in a matrix of pervasively sheared or pulverized rock material.

- ss = sandstone
- sp = serpentinite
- gs = greenstone
- sh = shale

△ Landslide deposits and debris avalanche scars that are too small to be delineated at this scale

Headwall scarps of block slump and debris flow landslides, and scarps left at sources of soil and rock debris avalanches.

Slopes exhibiting evidence of continuous or intermittent downslope surface creep.

Debris Flow Landslides. Predominantly deposits of un-consolidated and unsorted soil and rock debris that have moved downslope en masse or in increments by flow or creep processes.

REFERENCE: Rice, S.J., Strand, R.G., and Smith, T.C., "Geology of the Upper Ross Valley and the Western Part of the San Rafael Area" *in* Geology for Planning in Central and Southeastern Marin County, California, California Department of Conservation, Division of Mines and Geology Open-File Report 76-2 S.F., Plate 1C, Map Scale 1:12,000

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2038.001GM.dwg

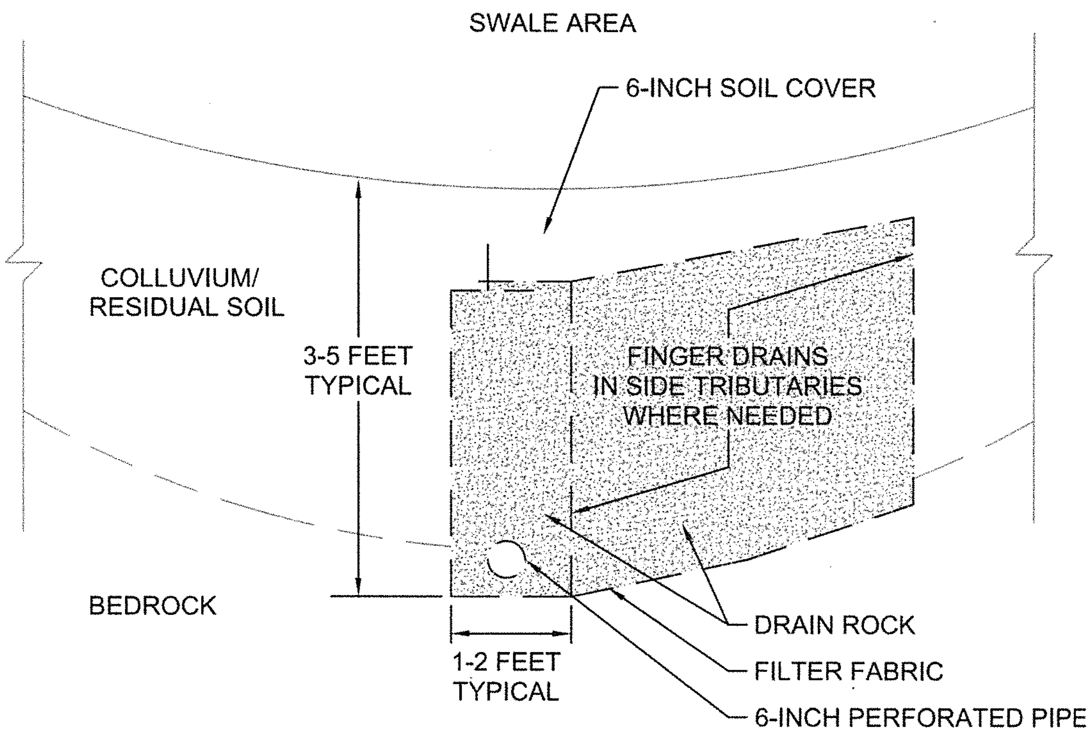
REGIONAL GEOLOGIC MAP

15 Wood Lane
APN 002-081-07
Fairfax, California

Project No. 2038.001 Date: 6/17/14

Drawn MFJ
Checked

3
FIGURE



NOTES:

(1) PIPE TO BE 6-INCH DIAMETER, PERFORATIONS DOWN, S=0.02 MIN., WITH TIGHT PIPE TO GRAVITY DISCHARGE

(2) FILTER FABRIC TO BE MIRAFI 140N

(3) PERMEABLE MATERIAL TO BE CALTRANS CLASS 1A OR 1B

(4) USE SWEEPS FOR ALL SUBDRAIN BENDS/ELBOWS

TYPICAL DETAIL TRENCH SUBDRAIN

(NO SCALE)

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FILE: 2038.001

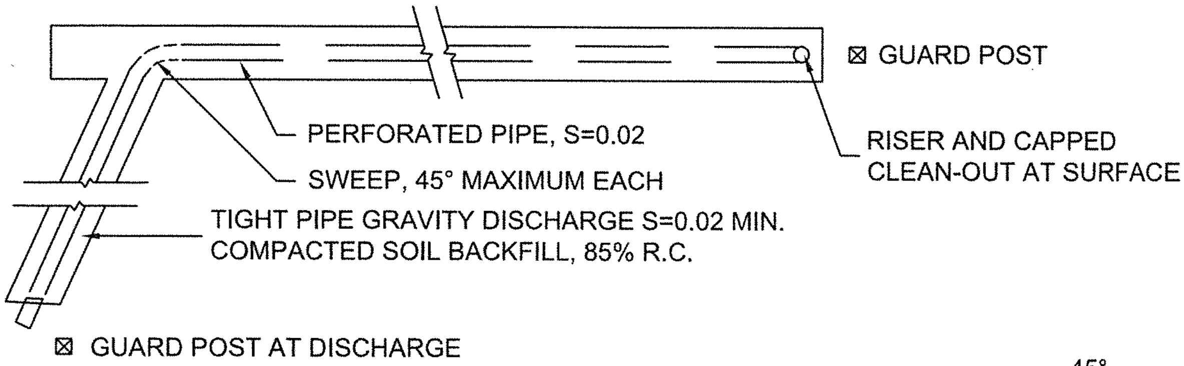
TYPICAL DETAIL TRENCH SUBDRAIN

15 Wood Lane
APN 002-081-07
Fairfax, California

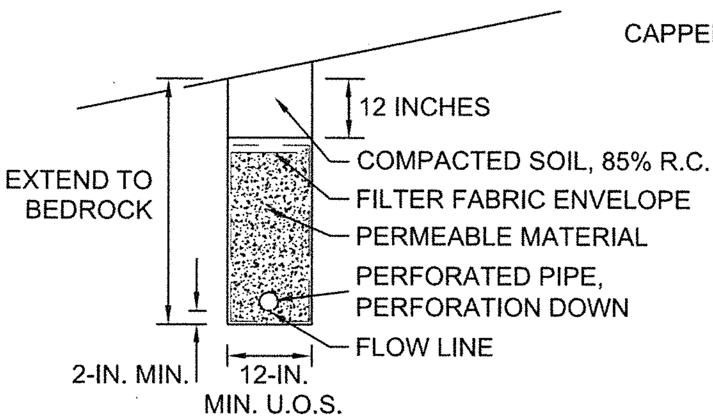
Project No. 2038.001 Date: 6-17-14

Designed SAS
Drawn MFJ
Checked

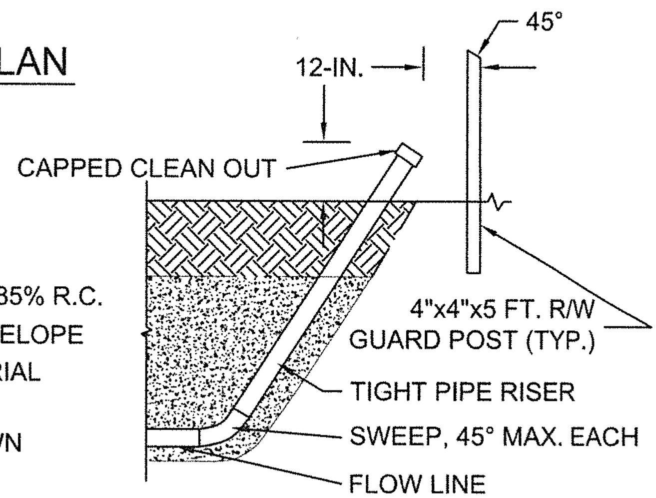
4
FIGURE



PLAN



SECTION



PARTIAL PROFILE

NOTES:

(1) PIPE TO BE 6-IN. DIAMETER, SCHEDULE 40 PVC
(2) FILTER FABRIC TO BE MIRAFI 140N OR EQUIVALENT
(3) PERMEABLE MATERIAL TO BE CALTRANS CLASS 1A OR 1B
(4) R.C. = RELATIVE COMPACTION

TYPICAL DETAILS FOR SUBDRAINS, RISERS, AND DISCHARGE

NO SCALE

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FILE: 2038.001DETAILS.dwg

TYPICAL DETAILS FOR SUBDRAINS, RISERS, AND DISCHARGE

15 Wood Lane
APN 002-081-07
Fairfax, California

Project No. 2038.001 Date: 6-17-14

Designed SAS
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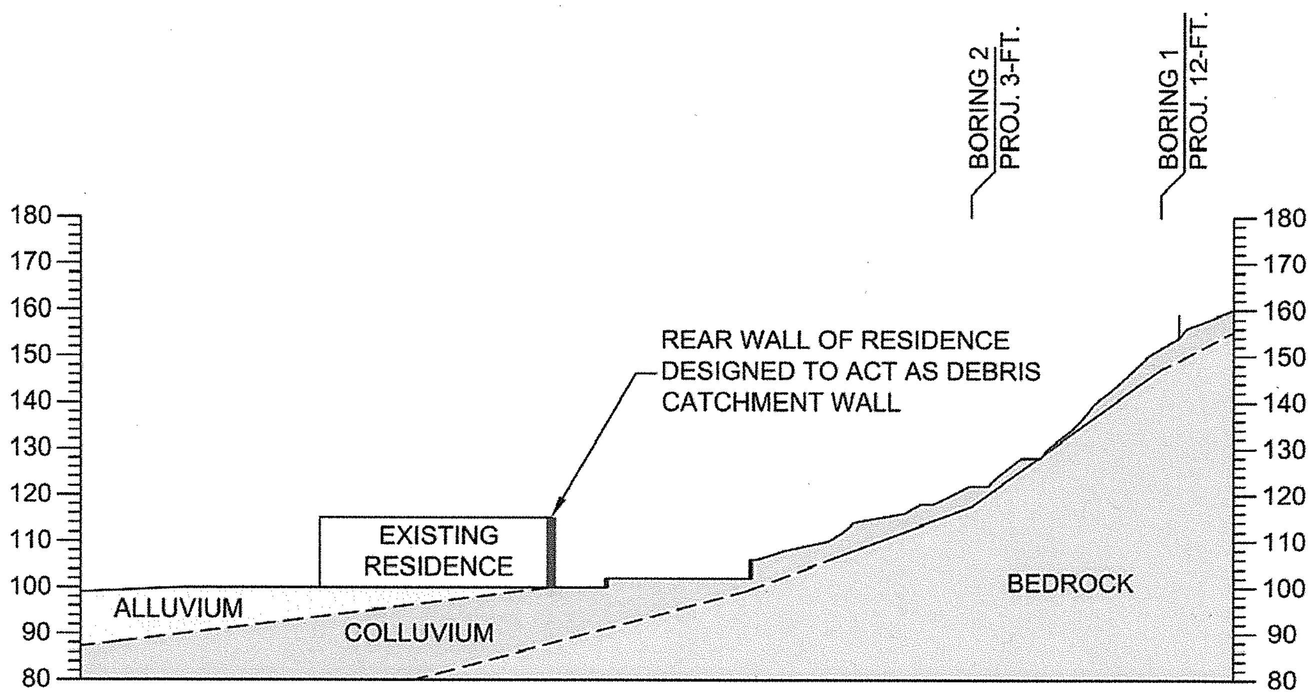
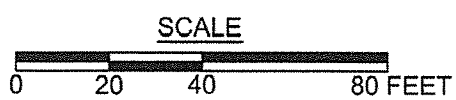
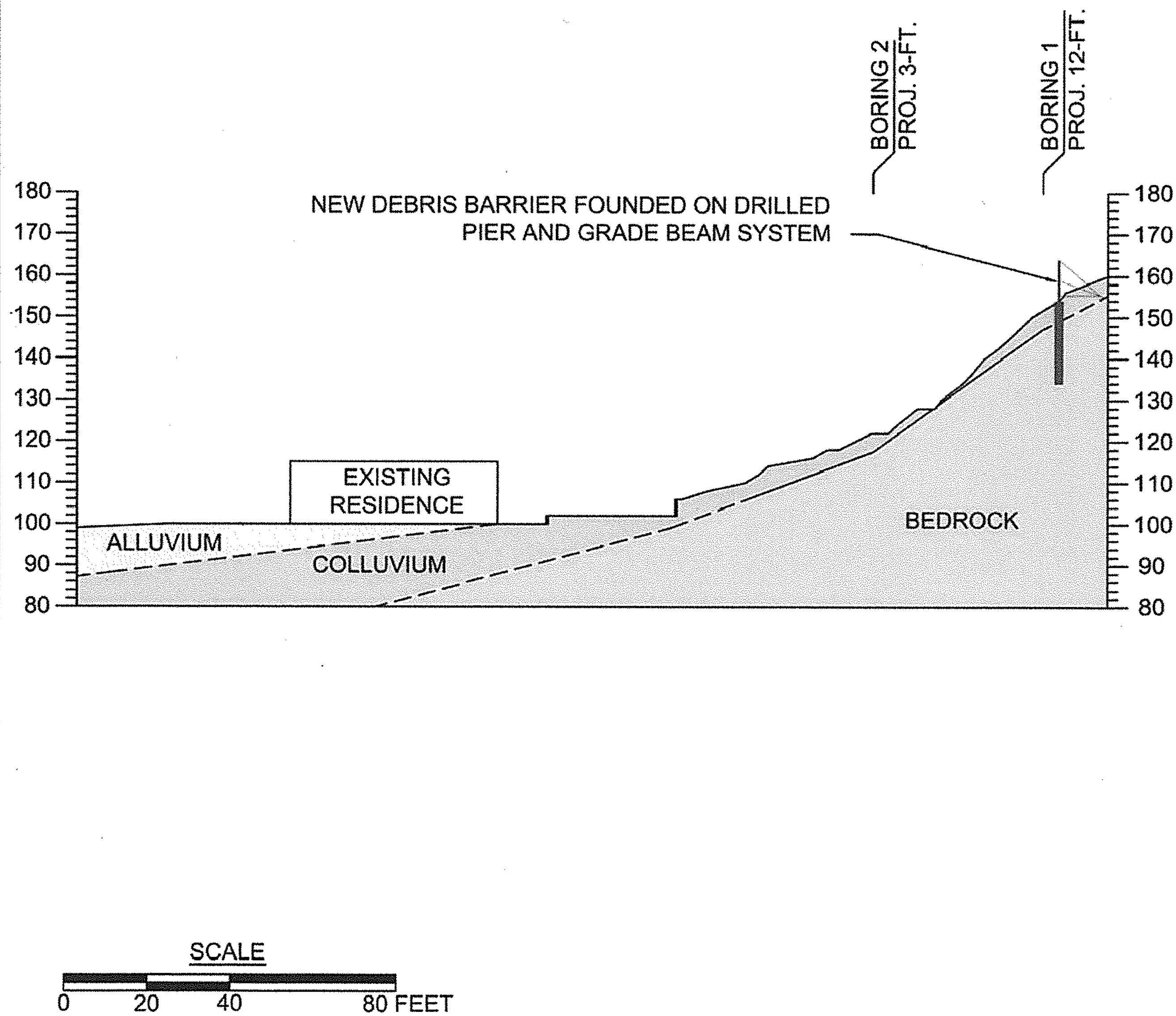
5
FIGURE





Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	OPTION 1 - INTEGRATED DEBRIS CATCHMENT WALL 15 Wood Lane APN 002-081-07 Fairfax, California Project No. Date:	Drawn MFJ Checked 6 FIGURE



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OPTION 2 - FREESTANDING DEBRIS BARRIER

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Project No. Date:

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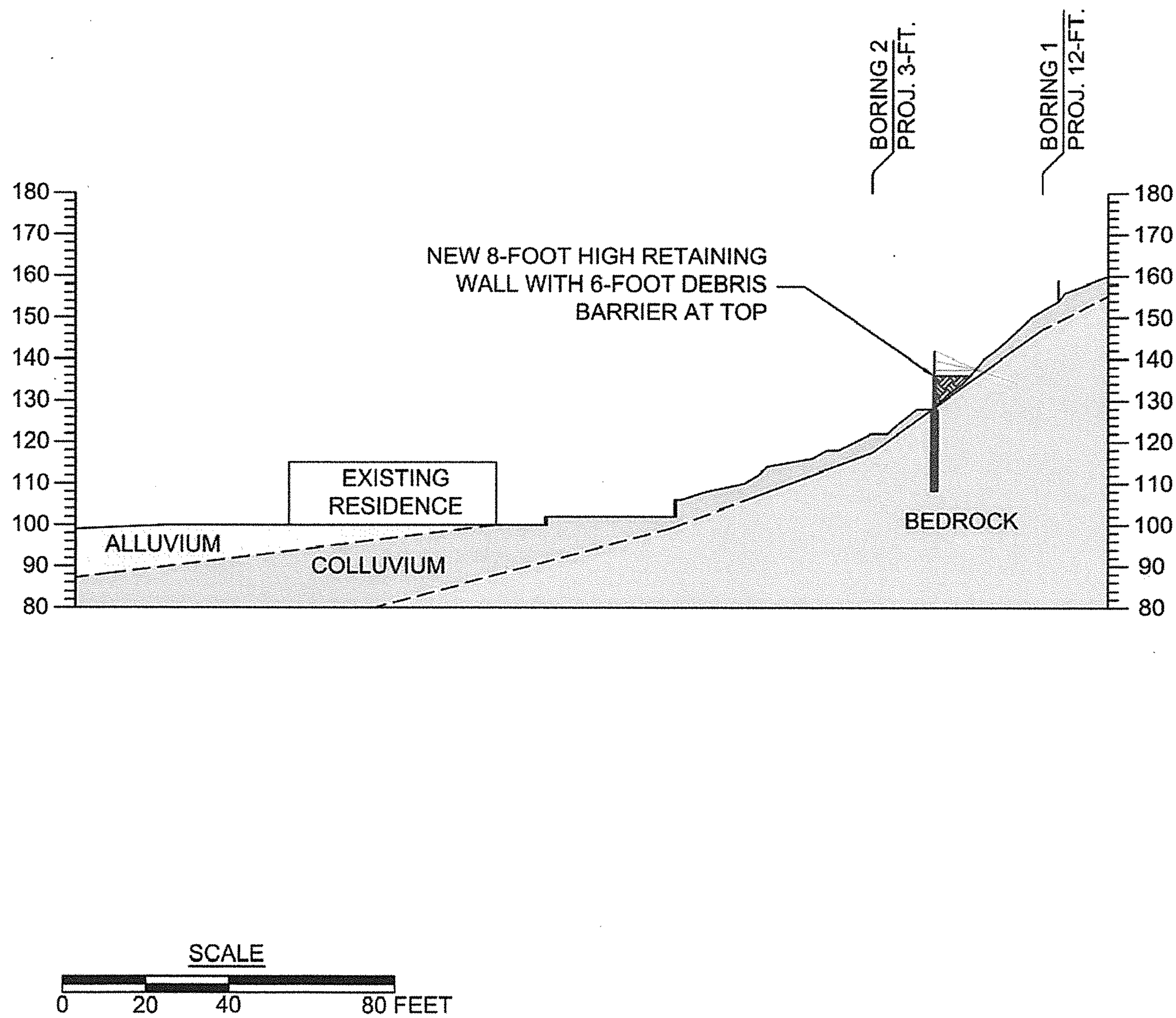
7
FIGURE



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OPTION 3 - COMBINED RETAINING WALL AND DEBRIS BARRIER

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Fairfax, California

Project No. Date:

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8
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. Soil and Rock Classification Systems

We explored subsurface conditions at the site with two exploratory borings drilled on June 5, 2014 at the approximate locations shown on Figure 2. Borings were excavated to depths between 5.0- and 16.5-feet using a portable, hydraulic-powered drill rig quipped with 4-inch solid continuous flight augers (Boring 1) and a 3.5-inch manually-operated auger (Boring 2).

Soil and rock materials encountered were logged and identified by our engineering geologist in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figure A-1, Soil Classification Chart and Key to Log Symbols and Figure A-2, Rock Classification Chart. The exploratory boring logs are presented on Figures A-3 and A-4

B. Laboratory Testing

We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate engineering properties. Samples were collected by use of a 3.0-inch diameter split-barrel Modified California sampler equipped with 2.5 x 6-inch brass tube liners and a 2.5-inch diameter split-barrel Standard Penetration Test (SPT) sampler without liners. Both samplers were driven by use of a 140-pound hammer falling 30 inches, and the number of blows required to drive the final 12-inches was recorded on the boring logs.

Samples were examined in the field, sealed to prevent moisture loss, and carefully transported to our laboratory. The following laboratory tests were conducted in in general accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Amount of Material in Soils Finer than No. 200 (75-µm) Sieve, ASTM D 1140; and
- Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D 4318.

Moisture content, density, and fines content test results are shown on the Boring Logs, Figures A-3 and A-4. Plasticity index results are presented on Figure A-5.

The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the excavation at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate, and changes in surface and subsurface drainage.

MAJOR DIVISIONS		SYMBOL	DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
	SAND with fines	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, muck, and other highly organic soils
ROCK			Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

- MODIFIED CALIFORNIA
- HAND SAMPLER
- STANDARD PENETRATION TEST
- ROCK CORE
- THIN-WALLED / FIXED PISTON
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

TV	FIELD TORVANE (UNDRAINED SHEAR)
UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

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FILE: 2038.001BL.dwg

SOIL CLASSIFICATION CHART

15 Wood Lane
APN 002-081-07
Fairfax, California

Project No. 2038.001 Date: 6-17-14

Drawn MFJ
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A-1
FIGURE

FRACTURING AND BEDDING

Fracture Classification	Spacing	Bedding Classification
Crushed	less than 3/4 inch	Laminated
Intensely fractured	3/4 to 2-1/2 inches	Very thinly bedded
Closely fractured	2-1/2 to 8 inches	Thinly bedded
Moderately fractured	8 to 24 inches	Medium bedded
Widely fractured	2 to 6 feet	Thickly bedded
Very widely fractured	greater than 6 feet	Very thickly bedded

HARDNESS

Low	Carved or gouged with a knife
Moderate	Easily scratched with a knife, friable
Hard	Difficult to scratch, knife scratch leaves dust trace
Very hard	Rock scratches metal

STRENGTH

Friable	Crumbles by rubbing with fingers
Weak	Crumbles under light hammer blows
Moderate	Indentations <1/8 inch with moderate blow with pick end of rock hammer
Strong	Withstands few heavy hammer blows, yields large fragments
Very strong	Withstands many heavy hammer blows, yields dust, small fragments

WEATHERING

Complete	Minerals decomposed to soil, but fabric and structure preserved
High	Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates
Moderate	Fracture surfaces coated with weathering minerals, moderate or localized discoloration
Slight	A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation
Fresh	Rock unaffected by weathering, no change with depth, rings under hammer impact

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

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ROCK CLASSIFICATION CHART

15 Wood Lane
APN 002-081-07
Fairfax, California

Project No. 2038.001 Date: 6-17-14

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A-2
FIGURE

BORING 1

EQUIPMENT: Portable Hydraulic Drill Rig with 4-inch Solid Continuous Flight Augers
DATE: 6/5/2014
ELEVATION: +/-153-Feet* (Assumed Datum)
*REFERENCE: Stephen J. Flatland, PLS, 2013

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			SILTY SAND (SM) Light gray, dry, loose, fine-grained, slightly porous, ~45% low plasticity silt, ~10% fine to coarse, angular to subrounded sandstone fragments [COLLUVIUM]
LL=48 PI=17	47.5% P200	AM1 1.96	9	20.0	77					
LL=54 PI=32	20.0% P200	AM1 1.02	10	14.8	91	1				CLAYEY SAND WITH GRAVEL (SC) Light yellow-brown, moist, loose, fine to coarse-grained, ~20% medium to high plasticity clay, ~25% fine to coarse angular sandstone fragments [RESIDUAL SOIL]
							5			
			20	9.3	115	2				SANDSTONE Light yellow-brown fine-grained arkose, moderately hard, weak, crushed, moderately weathered with oxidation stains along fracture planes [BEDROCK]
			101	6.6	125	3	10			Harder drilling noted, grades hard, moderately strong, closely to intensely fractured at 8.0-feet.
										Harder drilling noted at 11.5-feet
						4				Easier drilling noted, grades with occasional 2- to 4-inch interbeds of dark gray, sheared, crushed siltstone at 13.0-feet.
							15			
			38	12.5		5				
										Boring terminated at 16.5-feet. No groundwater encountered during exploration.
						6	20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

15 Wood Lane
APN 002-081-07
Fairfax, California

Project No. 2038.001 Date: 6-17-14

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A-3
FIGURE

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)
						0	0		
						1			
							5		
						2			
						3	10		
						4			
							15		
						5			
						6	20		

BORING 2

EQUIPMENT: 3.5-inch Manual Bucket Auger

DATE: 6/5/2014

ELEVATION: +/-122-Feet* (Assumed Datum)

*REFERENCE: Stephen J. Flatland, PLS, 2013

CLAYEY SAND (SC) (0 to ~4.5 feet)
Medium red-brown, moist, medium dense, fine to coarse-grained, ~40% low to medium plasticity clay, ~10% fine to coarse angular sandstone, chert, and shale fragments [COLLUVIUM]

MELANGE (~4.5 to 5.0 feet)
Medium to dark gray with local red mottling, primarily highly sheared and highly weathered siltstone with lesser inclusions of highly weathered arkosic sandstone [BEDROCK]

Boring terminated at 5.0-feet.
No groundwater encountered during exploration.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

15 Wood Lane
APN 002-081-07
Fairfax, California

Project No. 2038.001 Date: 6-17-14

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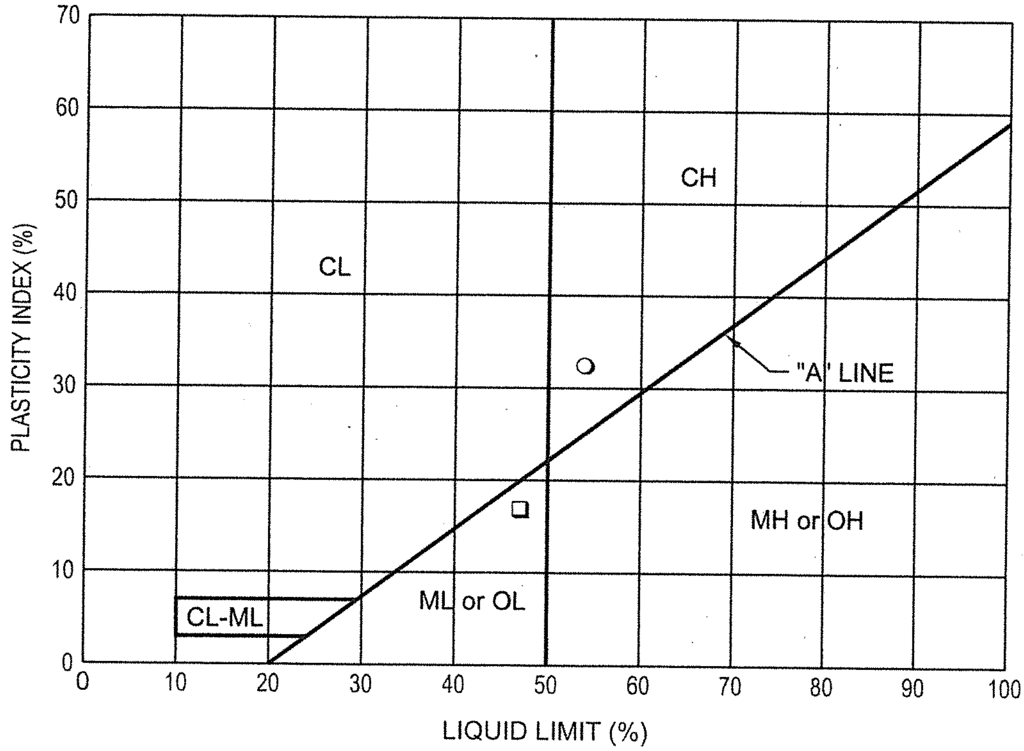
A-4
FIGURE



SYMBOL	SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)
❑	BORING 1 1.0-feet	SILTY SAND (SM) Light gray	48	31	17
○	BORING 2 4.5-feet	CLAYEY SAND WITH GRAVEL (SC)	54	22	32

REFERENCE: Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D 4318

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FILE: 2038.001PI.dwg

PLASTICITY INDEX TEST RESULTS

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Project No. 2038.001 Date: 6-19-14

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A-5
FIGURE

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APPENDIX B

SUBSURFACE EXPLORATION BY OTHERS

LOG OF BORING: 1

Equipment: Portable
Elevation: Exist Grade   Date: 1/9/14

Shear Strength (lbs/sq ft): 5000, 4000, 3000, 2000, 1000, 0

Moisture Content (%) | Dry Density (pcf) | Depth (ft) | Sample

Blows/Ft. *

CONCRETE FOUNDATION

Depth (ft): 0, 5, 10, 15, 20

Blows/Ft.: 0, 20, 40, 60, 80, 100

- CONCRETE FOUNDATION, hard
- COMPACTED DENSE SANDY GRAVEL GW
- DARK GRAY SILTY SAND (SM), med. dense, damp

The log of subsurface conditions shown herein applies only at the specific boring or test pit or probe location on the date indicated. It may not be representative of subsurface conditions at other locations and/or other times.

NO FREE WATER WAS OBSERVED IN TEST PIT OR BORING AT TIME OF INVESTIGATION. HOWEVER, THE GROUND WATER TABLE MAY RISE OR SEEPAGE MAY BE PRESENT DURING THE WINTER

Blow / Ft. **

** 1-3/4" diameter cone penetration test, converted to Equivalent Standard Penetration Blow Counts

* Standard Penetration Test

EARTH SCIENCE CONSULTANTS
Soil • Foundation and Geotechnical Engineers

Job No. 14-7038   Appr ______   Date: 1/10/2014

LOG OF BORING: 1

15 Wood Lane
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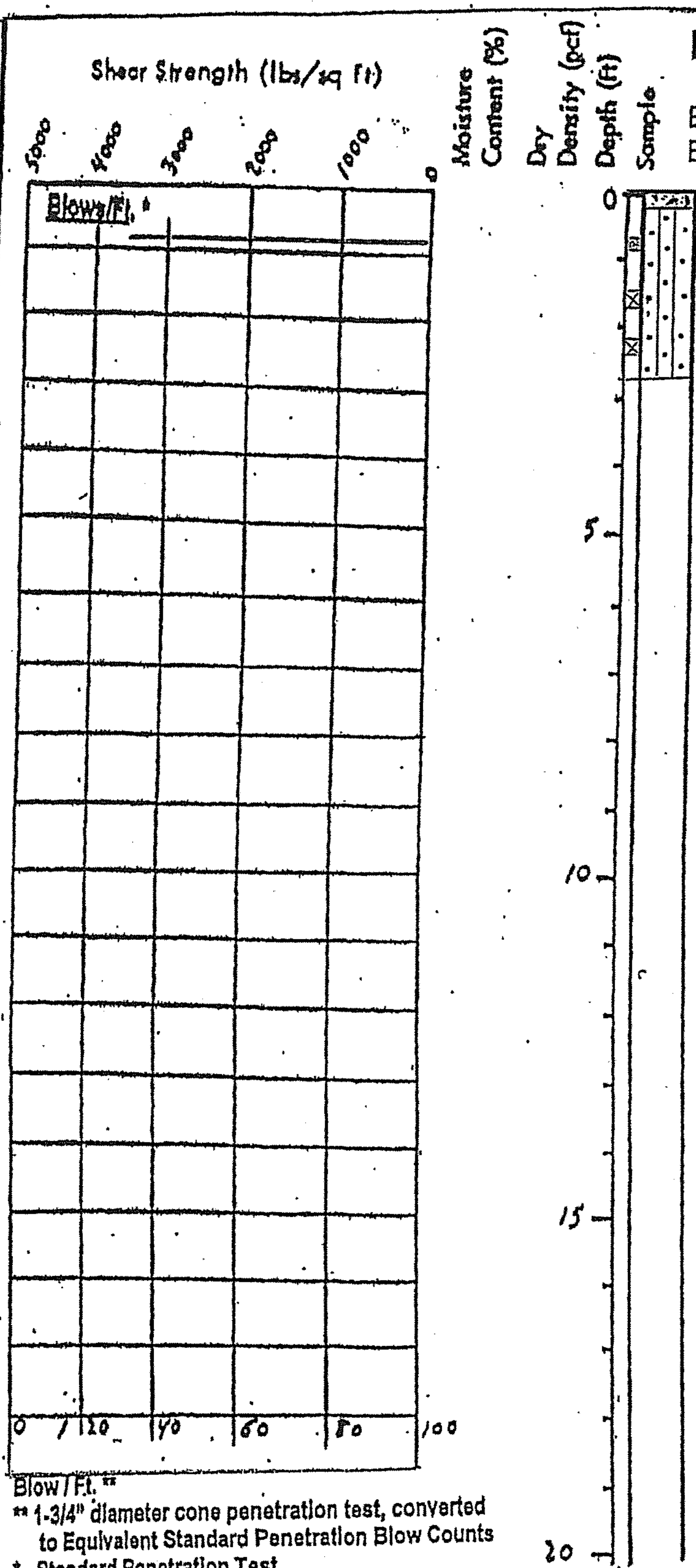
PLATE

3



Blow / Ft. **
** 1-3/4" diameter cone penetration test, converted to Equivalent Standard Penetration Blow Counts
* Standard Penetration Test

LOG OF BORING: 2

Equipment: Portable
Elevation: Exist Grade Date: 1/9/14

GRAY SAND PEA GRAVEL, loose, dry

BROWN SILTY FNIE SAND (SM), med. dense to dense

The log of subsurface conditions shown herein applies only at the specific boring or test pit or probe location on the date indicated. It may not be representative of subsurface conditions at other locations and/or other times.

NO FREE WATER WAS OBSERVED IN TEST PIT OR BORING AT TIME OF INVESTIGATION. HOWEVER, THE GROUND WATER TABLE MAY RISE OR SEEPAGE MAY BE PRESENT DURING THE WINTER

EARTH SCIENCE CONSULTANTS Soil • Foundation and Geotechnical Engineers Job No. 14-7038 Appr ____ Date: 1/10/2014	LOG OF BORING: 2 15 Wood Lane Fairfax, CA	PLATE 4

LOG OF BORING: 3

Equipment: Portable
Elevation: Exist Grade   Date: 1/9/14

Shear Strength (lbs/sq ft): 5000, 4000, 3000, 2000, 1000, 0

Blows/Ft. *

Moisture Content (%) | Dry Density (pcf) | Depth (ft) | Sample

Depth (ft): 0, 5, 10, 15, 20

0 – SANDY PEA GRAVEL (GM), loose, dry

BROWN FINE COMPACTED SAND (SP), dense to med. dense

The log of subsurface conditions shown herein applies only at the specific boring or test pit or probe location on the date indicated. It may not be representative of subsurface conditions at other locations and/or other times.

NO FREE WATER WAS OBSERVED IN TEST PIT OR BORING AT TIME OF INVESTIGATION. HOWEVER, THE GROUND WATER TABLE MAY RISE OR SEEPAGE MAY BE PRESENT DURING THE WINTER

0 20 40 60 80 100

Blow / Ft. **

** 1-3/4" diameter cone penetration test, converted to Equivalent Standard Penetration Blow Counts

* Standard Penetration Test

EARTH SCIENCE CONSULTANTS
Soil • Foundation and Geotechnical Engineers

Job No. 14-7038   Appr ______   Date: 1/10/2014

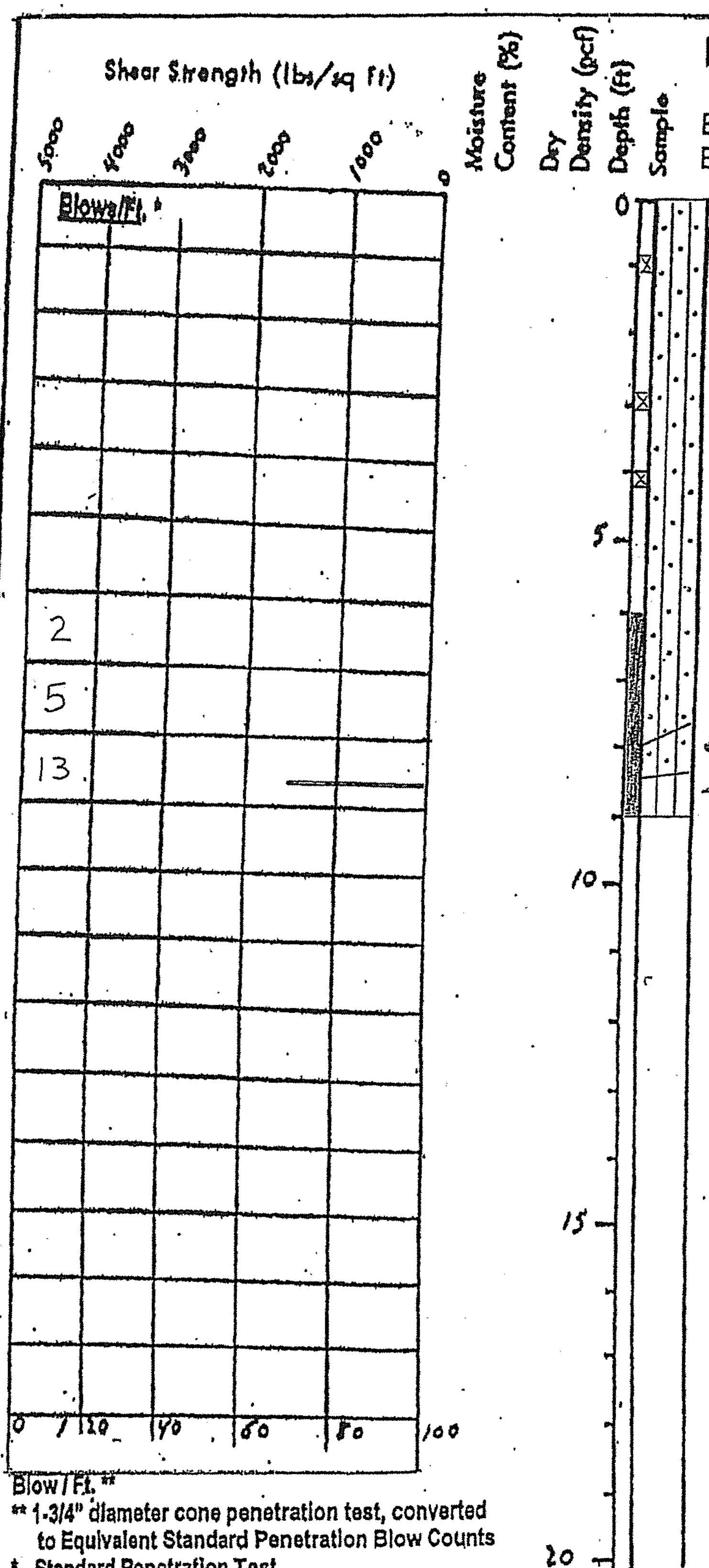
LOG OF BORING: 3

15 Wood Lane
Fairfax, CA

PLATE **5**

LOG OF BORING: 4

Equipment: Portable
Elevation: Exist Grade Date: 1/9/14



Blow / Ft. **

** 1-3/4" diameter cone penetration test, converted to Equivalent Standard Penetration Blow Counts

* Standard Penetration Test

DARK GRAY BROWN SILTY FINE SAND (SM), loose, damp, not compacted

LOOSE

LOOSE

LOOSE

GRAY BROWN SILTY FINE SAND (SM), medium dense, damp

GRAY BROWN FINE SANDY SILT (ML), stiff, damp t moist

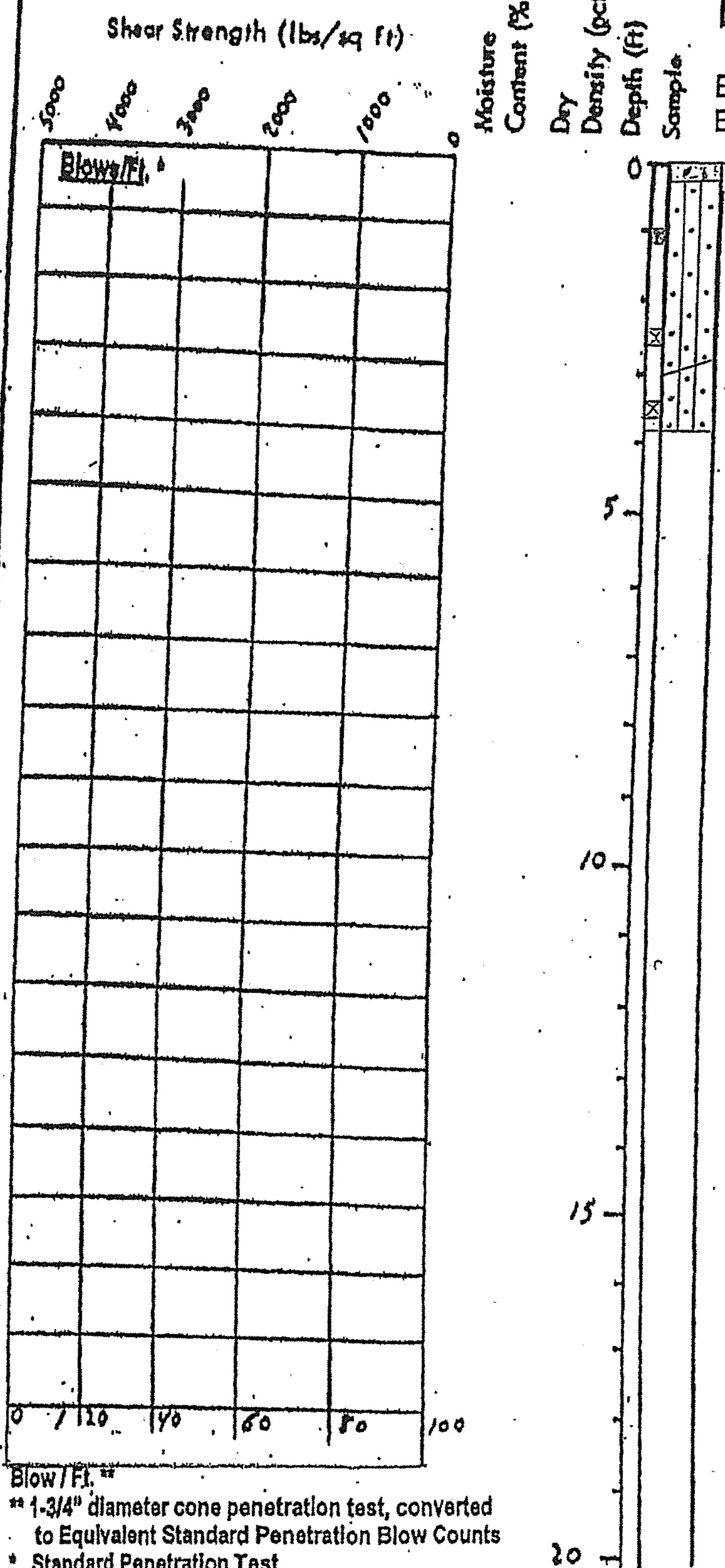
The log of subsurface conditions shown herein applies only at the specific boring or test pit or probe location on the date indicated. It may not be representative of subsurface conditions at other locations and/or other times.

NO FREE WATER WAS OBSERVED IN TEST PIT OR BORING AT TIME OF INVESTIGATION. HOWEVER, THE GROUND WATER TABLE MAY RISE OR SEEPAGE MAY BE PRESENT DURING THE WINTER

EARTH SCIENCE CONSULTANTS Soil • Foundation and Geotechnical Engineers Job No. 14-7038 Appr. Date: 1/10/2014	LOG OF BORING: 4 15 Wood Lane Fairfax, CA	PLATE 6

LOG OF BORING: 5

Equipment: Portable
Elevation: Exist Grade Date: 1/9/14



GRAY PEA GRAVEL, loose, dry

DARK GRAY BROWN SILTY FINE SAND (SM), med. dense, damp
Dense @ 1'

LIGHT TAN SILTY FINE SAND (SM), dense, dry with sub-round & angular rock fragments

The log of subsurface conditions shown herein applies only at the specific boring or test pit or probe location on the date indicated. It may not be representative of subsurface conditions at other locations and/or other times.

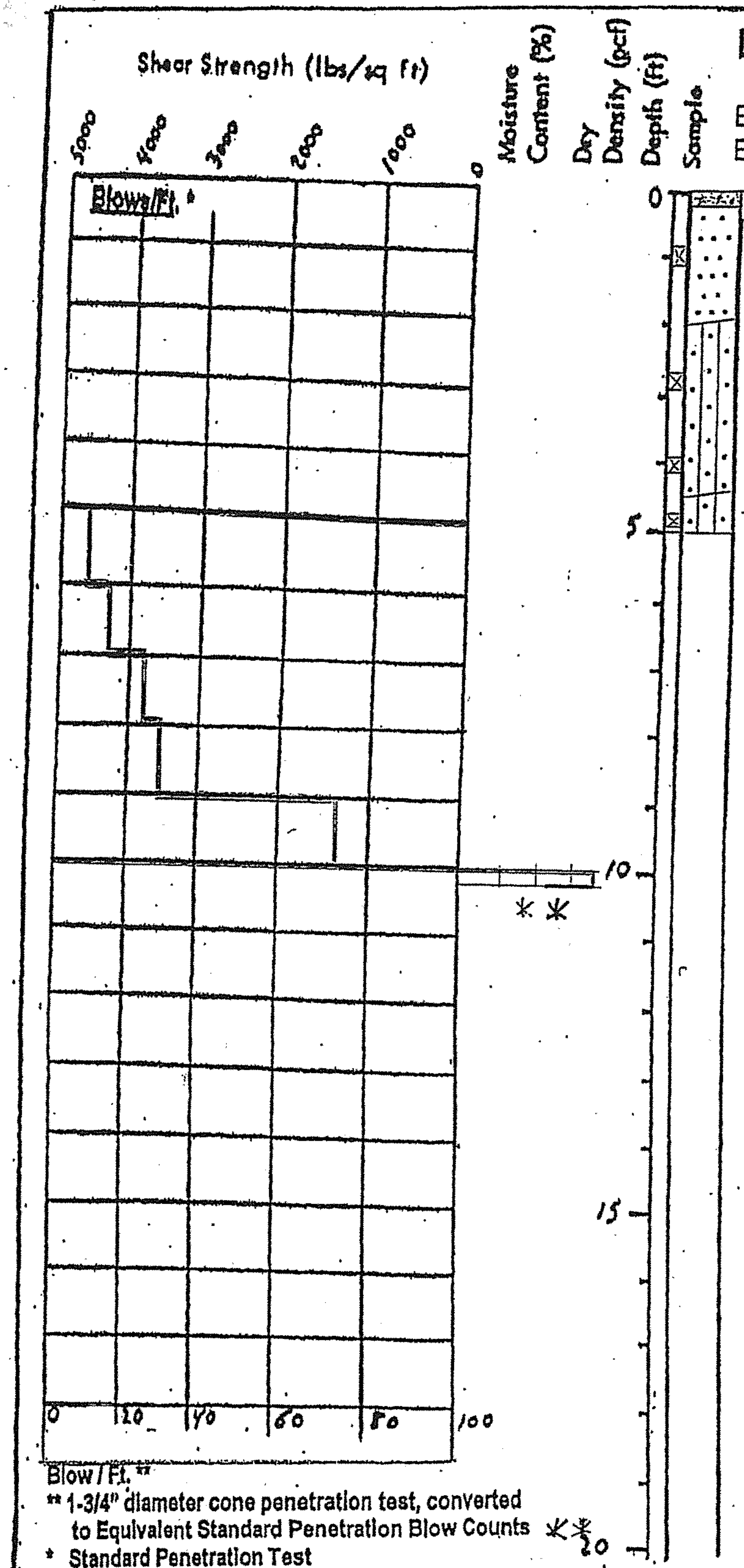
NO FREE WATER WAS OBSERVED IN TEST PIT OR BORING AT TIME OF INVESTIGATION. HOWEVER, THE GROUND WATER TABLE MAY RISE OR SEEPAGE MAY BE PRESENT DURING THE WINTER

** 1-3/4" diameter cone penetration test, converted to Equivalent Standard Penetration Blow Counts
* Standard Penetration Test

EARTH SCIENCE CONSULTANTS Soil • Foundation and Geotechnical Engineers Job No. 14-7038 Appr ___ Date: 1/10/2014	LOG OF BORING: 5 15 Wood Lane Fairfax, CA	PLATE 7

LOG OF BORING: 6

Equipment: Portable
Elevation: Exist Grade Date: 1/9/14



2" PEA GRAVEL

DARK GRAY BROWN SILTY FINE SAND (SP), med. dense, damp

DARK BROWN SILTY SAND (SM), med. dense, moist

DARK BROWN SILTY SAND (SM), med. dense, damp

The log of subsurface conditions shown herein applies only at the specific boring or test pit or probe location on the date indicated. It may not be representative of subsurface conditions at other locations and/or other times.

NO FREE WATER WAS OBSERVED IN TEST PIT OR BORING AT TIME OF INVESTIGATION. HOWEVER, THE GROUND WATER TABLE MAY RISE OR SEEPAGE MAY BE PRESENT DURING THE WINTER .

EARTH SCIENCE CONSULTANTS Soil • Foundation and Geotechnical Engineers Job No. 14-7038 Appr. [signature] Date: 1/10/2014	LOG OF BORING: 6 15 Wood Lane Fairfax, CA	PLATE 8

MPEG
MILLER PACIFIC
ENGINEERING GROUP

December 15, 2014
File: 2038.001cltr.doc

Town of Fairfax
142 Bolinas Road
Fairfax, California 94930

Attn: Ms. Linda Neal and Mr. Ray Wrysinski

Re: Geotechnical Response to Planning Department Comments
15 Wood Lane (APN 002-081-07) Improvements
Fairfax, California

Gentlemen:

As requested following our recent project meeting, this letter clarifies our recommendations regarding drainage improvements at 15 Wood Lane in Fairfax, California. We understand, upon further review of plan review letters previously issued by the Town of Fairfax, that no explicit request of requirement for drainage improvements has been made. However, we understand that clarification is desired regarding the need (or lack thereof) for drainage improvements at the site as discussed in our Geotechnical Investigation report dated June 24, 2014 and addendum letter dated October 14, 2014. In short, it is our opinion that, provided the integrated catchment wall is designed and constructed as recommended in our October 14, 2014 letter, construction of additional drainage improvements at the site as a means of protecting life safety is not necessary.

In our June 24, 2014 Geotechnical Investigation report (Page 8, paragraph 3), we concluded that the residence at 15 Wood Lane is at moderate risk to be affected by future debris flow or debris avalanche landslides originating on natural slopes south of the property (beyond the property limits). We do wish to note that the slope in question has apparently performed well during several notable rainy seasons and heavy storms, including in 1982-83, 1986, 1989, 1996-97, and 2005. We concluded that the risk of landslides capable of affecting the residence and originating within the property limits is generally low. Our recommendations for mitigation of risks associated with potential landslides originating offsite are generally twofold; 1) provide a debris barrier to protect the residence in the event of landslide initiation or, 2) provide drainage improvements for the purpose of reducing the risk of soil saturation and associated landsliding.

It remains our opinion that construction of subsurface drainage improvements on the adjacent property to the south (upslope), generally at the location of the primary anticipated source area for future debris flows, is the most effective method by which to reduce the risk of future landsliding. However, this primary debris flow source area is beyond the scope of control of the project Owners. Therefore, the Owners have elected to pursue a mitigation option consisting of a debris catchment wall integral to the residence itself. The purpose of the catchment wall is to protect the residence from collapse and protect the life safety of its occupants in the event of landslide occurrence. In no way is the catchment wall intended to reduce the risk of soil saturation or (by association) landslide initiation, nor does its effectiveness rely on additional improvements (such as drainage) acting in concert. Therefore, it is our opinion that, provided the integrated catchment wall is designed and constructed as recommended, construction of additional drainage improvements either on- or offsite as a means of protecting life safety is not necessary.

EXHIBIT # D

We trust that this letter provides the information you require at this time. Should there be any questions or concerns, please do not hesitate to contact us.

Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY

CERTIFIED ENGINEERING GEOLOGIST
MICHAEL JEWETT
No. CEG 2610
EXP. 1-31-15
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
SCOTT A. STEPHENS
2398
Exp. 6/30/15
GEOTECHNICAL
STATE OF CALIFORNIA

Mike Jewett
Engineering Geologist No. 2610
(Expires 1/31/15)

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/15)

MILLER PACIFIC
ENGINEERING GROUP

October 14, 2014
File: 2038.001bltr.doc

Building Solutions
152 Auburn Street
San Rafael, California

Attn: Mr. Mark Bruce and Mr. John Fraine

Re: Addendum to Geotechnical Investigation
Integrated Debris-Catchment Wall Design Criteria
15 Wood Lane (APN 002-081-07)
Fairfax, California

Introduction

As requested, this letter presents design-level geotechnical criteria for use by the project Structural Engineer in design of the planned integrated debris-catchment wall at 15 Wood Lane in Fairfax, California. This letter also presents our professional opinion regarding the suitability of the proposed debris-catchment wall at 15 Wood Lane in Fairfax, California with respect to its effect on the property's protection from damage arising from possible debris-flow landslides originating on neighboring parcels. We judge that the proposed scope of work at 15 Wood Lane does not impact the stability of the hillside, either on or off the property.

We previously issued a Geotechnical Investigation for the project dated June 24, 2014, for the purpose of providing conceptual options for mitigation of potential debris-flow landslides originating on the neighboring property to the south. We understand at this time that the project Owner has elected to pursue "Conceptual Mitigation Option 1" as presented in our Investigation report, a scenario in which the rear wall of the planned residence is structurally designed to withstand the impact of future debris-flows as a means of improving the level of protection of life safety at the residence.

Geotechnical Design Criteria for Proposed Integrated Debris-Catchment Wall

As discussed in our previous Investigation report, the residence's new rear wall should be designed by a suitably qualified, licensed Structural Engineer to withstand debris impact having an equivalent fluid pressure of 200 pcf up to a height of at least 5-feet for the purpose of protecting the residence from structural collapse in the event of debris-flow impact. This may require "returns" or buttresses to some extent along the north and south exterior walls where they adjoin the planned catchment wall. Wall foundations could consist of drilled, cast-in-place concrete piers; alternatively, shallow foundations may be utilized provided sufficient lateral support can be achieved to resist sliding and overturning in the event of debris impact. Catchment wall foundations should be designed in accordance with the criteria shown in Table A.

TABLE A
FOUNDATION DESIGN CRITERIA – INTEGRATED CATCHMENT WALL
15 Wood Lane
APN 002-081-07
Fairfax, California

Drilled, Cast-in-Place Concrete Piers

Minimum Diameter:	18 inches
Skin Friction[1]:	
Medium-Dense Sandy Alluvial Soils:	750 psf
Lateral Passive Resistance[2,3,4]:	
Medium Stiff/Dense Alluvium:	300 pcf

Shallow Spread Footings

Minimum Width:	18 Inches
Minimum Depth:	18 inches
Maximum Allowable Bearing Capacity[5]:	2,000 psf
Lateral Passive Resistance[2,4]	
Medium Stiff/Dense Alluvium:	300 pcf
Coefficient of Base Friction:	0.30

Notes:
(1) Uplift resistance is equal to 80% of the total skin friction.
(2) Equivalent Fluid Pressure, not to exceed 10 times value in psf.
(3) Apply values over effective width of 2 pier diameters.
(4) Ignore upper 12 inches unless confined by asphalt or concrete.
(5) Dead plus live loads. May increase values by 1/3 for total loads, including wind and seismic.

We judge that apertures, such as windows and doors, may be utilized within the wall at the discretion of the Owner and Structural Engineer, so long as their inclusion does not compromise the wall's ability to withstand debris impact without structural collapse. Note that windows or doors extending to within 5-feet of the ground surface could break during a debris-flow event, resulting in mud or debris within the residence.

Our previous Investigation recommended the wall be designed to withstand debris impact for a minimum height of 10-feet. Upon further consideration and review of our project files regarding the 2005 debris-flow at 39 Wood Lane, which was inundated with no more than about 3-feet of debris following a large debris-flow event originating in a much larger source area, we conclude that a design height of 5-feet is sufficient at this site to reduce the potential for injury to occupants or structural collapse.

Suitability of Proposed Improvements

The project plan review letter issued by the Town Engineer, Mr. Ray Wrysinski, P.E., and dated March 20, 2014, indicates that ". . . a letter is required, bearing the signature and seal of the geotechnical engineer, which provides design criteria for stabilizing the hillside so the building site is reasonably protected from landslide damage or provides criteria for protection such as a debris barrier to keep the building site reasonably protected from landslide damage".

As indicated in our previous Geotechnical Investigation report, it is our professional opinion that construction of the proposed integrated debris-catchment wall will result in a significant increase in protection of life safety for occupants of the residence and will also provide reasonable protection from structural collapse following impact from a possible, but uncertain, debris-flow event. It should be noted that the primary purpose of this integrated wall is to prevent structural collapse, and that some cosmetic damage and debris cleanup may be expected following a debris flow, depending on the extent and placement of apertures such as windows and doors. Additionally, no improvement in the protection of life safety for persons outside the structure will be realized in the event of a debris-flow.

Construction of the proposed integrated debris-catchment wall will not affect stability of the slope south of the residence, and will neither increase nor reduce the risk of debris-flow occurrence. In addition, it is our opinion that construction of new surface or subsurface drainage improvement within the property boundaries, as apparently requested by the Town as part of any mitigation submittal, will have no significant effect on slope stability, will not reduce the risk of debris-flow initiation, and will not provide any meaningful improvement either in protection of life safety or in protection from structural damage in the event of a debris-flow. Therefore, we judge that, if the interest is protecting life safety and protecting the residence from structural collapse to minimize economic dislocation from geological hazards, then the integrated debris-catchment wall is sufficient to accomplish these purposes. Construction of any new drainage improvements within the property limits is not warranted or necessary at this time.

October 14, 2014

We trust that this addendum provides the information you require at this time. Should there be any questions or concerns, please do not hesitate to contact us.

Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY

CERTIFIED ENGINEERING GEOLOGIST
MICHAEL JEWETT
No. CEG 2610
EXP. 1-31-15
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
SCOTT A. STEPHENS
2398
Exp. 6/30/15
GEOTECHNICAL
STATE OF CALIFORNIA

Mike Jewett
Engineering Geologist No. 2610
(Expires 1/31/15)

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/15)

MILLER PACIFIC
ENGINEERING GROUP

June 26, 2014
File: 2038.001pro.doc

TOWN OF FAIRFAX

JUL 02 2014

RECEIVED

Building Solutions
152 Auburn Street
San Rafael, California 94901

Attn: Mr. Mark Bruce and Mr. John Fraine

Re: Proposal for Geotechnical Services During Construction
Erosion and Sediment Control Monitoring
15 Wood Lane
Fairfax, California

Introduction

Per your request, we are pleased to propose to provide our geotechnical services during construction of planned improvements to the existing single-family residence at 15 Wood Lane in Fairfax, California. The purpose of our services is to perform regular site visits through the winter months to verify that erosion and sediment control measures implemented during construction are in compliance with regulatory requirements set forth by the Town of Fairfax building official. We previously issued a Geotechnical Investigation Report addressing slope-stability concerns at the site on June 24, 2014 as part of our Phase 1 scope of services.

Based on review of a letter issued by the Town Engineer, Mr. Ray Wrysinski, and dated March 20, 2014, we understand that the Town is requiring erosion- and sediment-control measures during construction to be implemented in accordance with applicable requirements of the Marin County Stormwater Pollution Prevention Program, State Water Resources Control Board, and the Fairfax Town Code. In addition, the Town requires regular inspections by the project Civil or Geotechnical Engineer to verify compliance with said requirements

Scope of Services

Phase 2 – Erosion- and Sediment-Control Monitoring During Construction

We will perform intermittent site visits between October 15 and April 15, for the duration of project construction, to observe existing site drainage and erosion conditions, observe erosion- and sediment-control measures implemented to date, and to determine whether the onsite erosion- and sediment-control measures are in compliance with the Town's requirements. Upon satisfactory completion of the project, our observations and opinion regarding project compliance will be summarized in a brief letter report.

Schedule and Fee

Our services will be provided in accordance with our existing Agreement dated June 2, 2014, and the attached Schedule of Charges and Cost Estimate Worksheet. For the purpose of this proposal, we have assumed construction will take place over the course of 1 winter season, and therefore have anticipated a total of 7 site visits (1 per month from October 15 to April 15). It

MILLER PACIFIC
ENGINEERING GROUP

should be noted that the exact number and duration of our visits may be affected by several factors, including adjustment of the construction schedule, the need for additional visits if required by the Town or to verify any out-of-compliance measures are corrected, delays due to inclement weather, and other factors. We will notify you promptly of any condition likely to affect our overall budget estimate. Based on our understanding of the project and the assumptions outlined herein, we propose the following fee arrangements:

Phase 2 – Erosion and Sediment Control Monitoring.........Time and Expense, Estimate $2,300

We are pleased to have the opportunity to provide our services on this project and are prepared to begin work upon your authorization. When you wish us to proceed, please sign and return one copy of this letter.

Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY

Mike Jewett
Engineering Geologist No. 2610
(Expires 1/31/15)

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/15)

Accepted and Agreed

7/1/14
Date

TOWN OF FAIRFAX

142 BOLINAS ROAD, FAIRFAX, CALIFORNIA 94930
PHONE (415) 453-1584 / FAX (415) 453-1618

MEMORANDUM

To: Linda Neal – Principal Planner

Date: January 6, 2015
Page 1 of 1

From: Ray Wrysinski
Town Engineer

Subject: New Single Family Residence
15 Wood Lane
Fairfax, CA

A.P. 002-081-07

I have reviewed the 12/15/14 letter, by the Miller Pacific Engineering Group, provided with your 12/17/14 transmittal. That Geotechnical Engineering letter was a response to the Town Planning Department letter of about 11/20/14 on this project. The Planning Department letter referred to the 11/19/14 Town Engineer's Memorandum on this project. The issue to be resolved was the requirement in the 6/24/14 Miller Pacific report, on this project, on pages 9 and 10, for subsurface and surface drainage.

The 12/15/14 letter states that "provided the integrated catchment wall is designed and constructed as recommended in our October 14, 2014 letter, construction of additional drainage improvements at the site as a means of protecting life safety is not necessary".

The "life safety" issue is carefully noted in the 12/15/14 letter and additional discussion of the reasoning behind the Geotechnical Engineer's recommendations is given in their 10/14/14 letter. I have stated my concerns about soil stability at this site in previous Town Engineer memorandums. I believe the 6/24/14 Miller Pacific report does a good job of identifying potential soil stability problems at this site so anyone who carefully reads these documents and Town letters, on this project, will be informed of the potential (not certainty) for serious stability problems. With all that in mind, I recommend allowing the project processing to proceed with the requirement that the construction must satisfy the Miller Pacific Engineering Group recommendations. As some additional information, the 12/15/14 letter states the slope in question performed well during the 1982-83 storm (probably a reference to the 1/4/82 storm) and heavy storms after that date. This suggests the bank, on this site, was graded before that date. We have not been able to determine when the bank was graded. The date given suggests that Miller Pacific knows when the bank was graded. If they do know, I would appreciate it if they would inform the Town of the date when the bank was graded.

As noted above, I recommend that the processing of this project proceed.

Ray Wrysinski

Ray Wrysinski, P. E.
Town Engineer

EXHIBIT # E

TOWN OF FAIRFAX
142 BOLINAS ROAD, FAIRFAX, CALIFORNIA 94930
PHONE (415) 453-1584 / FAX (415) 453-1618

MEMORANDUM

To: Linda Neal – Principal Planner

Date: November 19, 2014

From: Ray Wrysinski
Town Engineer

Subject: New Single Family Residence
15 Wood Lane
Fairfax, CA

A.P. 002-081-07

I have reviewed the documents provided with your 10/22/14 transmittal. The items reviewed included the 24 sheet plans set by Building Solutions, dated 10/20/14 and the 10/14/2014 Addendum to Geotechnical Investigation by Miller Pacific Engineering Group.

The 7/22/14 Town Engineer Memorandum indicated that previously required information had been provided and it stated that the soil stabilization and drainage information was still needed. The applicant asked for clarification on the stability and drainage issues and I have a copy of an e-mail that provided additional information on requirements but I do not have a date on that e-mail. I believe it was produced soon after my 8/28/14 e-mail to you on that subject.

The above noted plans have significantly different sheets than previous submittals so some care in using them should be exercised. I noted that these plans no longer list the project engineers and project surveyor. That information will be needed at the permit review stage of the project. The plans state that the protective barrier for the house will be designed in accordance with the non-existent 10/14/2004 report. I know what report they are referring to but I think it is of enough importance that the report should be correctly identified.

The Miller Pacific Addendum, noted above, provides specific criteria for protecting the site residence from the probable future debris flow on this site. I did note that this addendum changes the recommended debris barrier catchment wall height to five feet high as compared to the 10 foot height recommended in the 6/24/14 report by Miller Pacific. So you will realize I see the significant difference in the two wall heights, the design force against a 10 foot high wall is four times the design force against a five foot high wall within the criteria provided. It is proportional to the square of the height of the wall. Certainly my comfort level would be much higher with the 10 foot high wall in place to protect this building when thinking of the unpredictable nature of the event that may happen. Setting that aside, the 10/14/2014 addendum provides a clear discussion of the justification for the five foot high wall to protect the building. On that basis, I recommend accepting the wall design criteria that is given.

As noted above, drainage is also an issue that must be dealt with. The 10/14/2014 Miller Pacific Addendum states on page 3, last paragraph, "In addition, it is our opinion that construction of new surface or subsurface drainage improvement within the property boundaries, as apparently requested by

EXHIBIT # F

the Town as part of any mitigation submittal, will have no significant effect on slope stability, will not reduce the risk of debris-flow initiation, and will not provide any meaningful improvement either in protection of life safety or in protection from structural damage in the event of a debris-flow. Therefore, we judge that, if the interest is protecting life safety and protecting the residence from structural collapse to minimize economic dislocation from geological hazards, then the integrated debris-catchment wall is sufficient to accomplish these purposes. Construction of any new drainage improvements within the property limits is not warranted or necessary at this time". The above quote discusses a non-existent Town design request for drainage within the property and it does not resolve the drainage issue. We need to, again, go back to the 6/24/2014 Miller Pacific report. Drainage is discussed in other parts of the report but the paragraph on page 9, under Geotechnical Site Drainage Recommendations is a good reference. It states "Since soil saturation commonly contributes to slope instability, providing adequate surface and subsurface drainage provisions is paramount to improving overall stability. Since the highest risk is posed by instability originating offsite to the south, its is recommended that the adjacent property owner, believed to be to Town of Fairfax, be consulted regarding the potential for construction of new drainage improvements by mutual agreement, creation of an easement, or other means". The report goes on to describe additional, "at a minimum" drainage improvements including drainage within the property limits. The need for drainage improvements above the southern property line (outside the building site property limits) and within the property limits is clearly defined in the 6/24/2014 report. This need for drainage improvements, required in the 6/24/2014 Miller Pacific Report, cannot be ignored and must be resolved. The Town has asked for the resolution of the drainage improvement requirement by having that drainage work shown on the plans that will be provided for Planning Commission review. I will restate that it has been determined that the Town is not the owner of the land above the southern property line (noted in the 6/24/2014 report).

The statements in the 10/14/2014 addendum appear to indicate that drainage improvements said to have been requested by the Town to be placed within the property are not needed. The Town did not request such improvements. What must be resolved is the very specific requirement in the 6/24/2014 report for drainage improvement southerly of this building site property line and drainage improvements within this site's property lines. The Town has suggested that the Miller Pacific Engineering Group may, after further consideration beyond the 6/24/2014 report, be able to provide justification for not constructing the drainage improvement they recommended in that report. If the decision is reached by this consultant, that the recommended drainage is not required and the Town receives a properly signed and sealed letter from the consultant that clearly and reasonably describes why those drainage improvements are not needed, then the Town may agree that the drainage improvements are not a required part of the project.

I hope the above provides the information you need. If you have questions about these comments, please let me know what they are.

Ray Wrysinski

Ray Wrysinski, P. E.
Town Engineer

TOWN OF FAIRFAX

142 BOLINAS ROAD, FAIRFAX, CALIFORNIA 94930
PHONE (415) 453-1584 / FAX (415) 453-1618

MEMORANDUM

To: Linda Neal – Senior Planner

Date: July 22, 2014

From: Ray Wrysinski
Town Engineer

Subject: New Single Family Residence
15 Wood Lane
Fairfax, CA

A.P. 002-081-07

I have reviewed the documents included with your 7/1/14 transmittal. The items reviewed included a 6/26/14 letter from Building Solutions, a 6/26/14 letter from Miller Pacific Engineering Group, four pages of information signed Jay Nelson – Geotechnical Engineer and also signed (6/25/14) by Mark Bruce of Building Solutions, a Nov., 2013 Boundary and Topographic Survey by Stephen Flatland with his signature and Seal and a 6/24/14 Geotechnical Investigation by Miller Pacific Engineering Group.

The submitted documents were reviewed to determine if they satisfied the requirements in the 4/24/14 Town Engineer review memorandum.

The submitted Boundary and Topographic Survey, now, shows the required title report information related to checking for easements and it also has the surveyor's signature and seal resolving those issues.

There was a requirement to provide information on the potential for landslide damage at this site. The information that was in this submittal, from Jay Nelson, was also received with your 4/2/14 transmittal and that information was not what was needed regarding the landslide question. The 6/24/14 Miller Pacific investigation does provide the information needed for our review of the landslide question.

A point I need to make, now, is that we have two geotechnical engineers involved in this project. Earth Science Consultants (Jay Nelson) is primarily involved in the house related soils engineering and Miller Pacific Engineering Group is primarily involved in the hillside stability and site drainage issues. There may be some overlap of work between these two engineering firms during the permit processing and project construction.

On page 8 of the Miller Pacific, 6/24/14, investigation report it states, in the third paragraph, "---areas between the location of Boring 2 and the south property line are judged to have moderate to high potential for slope instability, ---". In paragraph four it states "The most significant threat to the residence is posed by debris flows originating high on the slopes beyond the southern property line ---". The report goes on in significant conceptual detail on how to deal with these slope stability problems.

The Miller Pacific report provides a number of recommendation and options for dealing with site instability and debris flow problems. The report indentifies some basic, needed, drainage improvements.

They give three conceptual options for protective improvements and also discuss an earth buttress option and an option for placing a series of walls. They discuss rough cost estimates, for some of the work, ranging from $100,000 to $200,000.

The Town owned Barker Avenue street right of way that is above and adjacent to the southerly property line of this site is noted in the Miller Pacific report as a potential source of debris flow material. The grading excavation, previously, done on this site, downhill of and adjoining Barker Avenue, has, in my opinion (also refer to the 6/24/14 report, middle of pg. 10), increased the potential instability of the soil mass in Barker Avenue (also see bottom pg. 7 of the 6/24/14 report). On pages 9 and 10 of the Miller Pacific report, minimum, drainage installation recommendations for the Barker Avenue area are given. I recommend that the Town complete an agreement with an encroachment permit as a project requirement for the installation and maintenance of those drainage improvements. Maintenance of those improvements must include any, needed, future repairs. We have discussed that there will be a required Hold Harmless Agreement, between the Town and the property owner, related to soil that may move downhill from Barker Avenue. In reviewing this encroachment permit with the Town Attorney, you should discuss the possible requirement for additional soil stabilization work beyond the above drainage improvements to provide greater assurance that soil in Barker Avenue will not move downhill onto this project site. At this time we need to have a written statement from the Miller Pacific Engineering Group that will clarify if there is a necessity for the, above, drainage work to extend into the property above Barker Avenue.

The concrete ditch, along the project site southerly property line, called for in the 6/24/14 report will be on weak soil at the top of a steep cut bank. This weak soil will likely creep downhill resulting in the concrete ditch cracking and failing. Design criteria must be provided to prevent creep movement failure of this ditch. Topography map information must be provided for the area where the above drainage improvements will be placed, uphill of the southerly property line, so the location of the work and the affect of the work on existing trees can be shown. 100 year storm flow drainage calculations, at the construction permit review stage of the project, are required to show the stormwater flow quantity and conduit sizing to get the flow down the hill to the house yard area. A drainage easement must be provided for this flow transmission from Barker Avenue and down through this site.

The above drainage improvements may affect future development of Barker Avenue for access to the property above Barker Avenue. The design proposed must show that it minimizes access use problems for future development of Barker avenue.

The work to provide site landslide protection improvements is very significant and must be shown on the plans for this stage of review so that it can be considered by the Planning Commission. The proposed design must be shown in good conceptual so that locations of work and improvments, including heights of structures, can be seen on the plans. As may be appropriate, soil removal quantities from trenching, from drilled piers and from select backfill placement (as needed for subdrains and wall drains) must be shown so that grading quantities can be considered by the Planning Commission. If earth buttress work is proposed, the grading quantities for that must be given. Construction level design information must be provided for proposed work when the building permit is applied for. The, needed, conceptual information, for this stage of review, must include showing any tree removals that may be required for the proposed work.

Information required in previous review memorandums has been provided. We must now obtain the additional information sufficient for Planning Commission review of the needed soil stabilization and drainage work.

I recommend that the processing of this project be delayed until the above, noted, information is provided.

Ray Wrysinski

Ray Wrysinski, P. E.
Town Engineer

REGISTERED PROFESSIONAL ENGINEER
RAY C. WRYSINSKI
No. 23518
Exp. 12/31/15
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MEMORANDUM

To: Linda Neal – Senior Planner

Date: March 20, 2014

From: Ray Wrysinski
Town Engineer

Subject: New Single Family Residence
15 Wood Lane
Fairfax, CA

A.P. 002-081-07

I have reviewed the documents that were enclosed with your 2/25/14 transmittal. The items reviewed were a 23 sheet plan set from Building Solutions, dated 2/19/14 and included in that set was a boundary and topographic survey, dated Nov., 2013, by Stephen Flatland, Professional Land Surveyor and there was a Geotechnical Investigation report by Earth Science Consultants, dated January 29, 2014.

A site review was done 3/19/14.

Town Code Section 17.072.080 provides a list of submittal requirements for development. The requirements include providing a topographic and boundary survey signed by a licensed surveyor. The survey must include boundary lines, dimensions and easements If there are no easements, a notation on the survey must be included, stating that there are no easements. The easement requirement for this survey map can be resolved by a note on the plan stating "All easements are shown" or if there if there are no easements a note such as "Based on the review of the title report (give the date and title company source of the report) and based on the surveyor's knowledge of this site, there are no easements". Two copies of a current title report and two copies of the current fee title deed must be submitted for use by the Town in reviewing this application. We normally require a recorded record of survey for confirmation of the site boundary on projects like this. Due to the fact that the proposal keeps the building aligned with the existing building envelope, we may not need to require that record of survey. The record of survey requirement will be determined based on review of the submitted survey with the title report and deed information to be submitted. The topographic and boundary survey, submitted, must provide the required easement information and must bear the signature and seal of the surveyor responsible for it.

The submitted survey shows existing trees so that requirement is satisfied. The required structures, fences, retaining walls and driveway are shown. The grading and drainage plan shows a very schematic drainage plan but since this existing developed site is not being regraded that plan is sufficient for this stage of processing. The survey shows the existing grades near the house to be very flat which suggests that the existing condition drained poorly. The drainage piping shown on the grading plan must be sized to carry the substantial flow that will come to the building area from the hillside southerly and above the building location. To resolve this problem in an uncomplicated way, the project architect and project civil engineer must provide a written sign-off on the drainage near the house, as being satisfactory, prior to the building permit being finaled. Since there are no new hard surface areas being proposed, that would increase peak stormwater flows, there is no requirement to provide mitigation for increased stormwater flows.

There is a requirement that sanitary sewer, water and storm drainage lines must be shown and labeled with their sizes. Since this is an existing, developed, site, there should not be a need for excavation for these underground utility lines so they do not need to be shown on this plan.

No trees are shown to be removed so there is no need for a Tree Committee report and permit.

A report by a registered civil engineer specializing in soils and foundation engineering is required. The submitted report provides much of the needed information. Site drainage and watershed boundaries and site drainage patterns are in the requirements but since this is an existing developed site, those requirements are waived.

The geotechnical report stated that the scope of work included only subsurface conditions for the actual proposed structure and generally did not include accessory areas. More information is required. In my site review, I found conditions that provide significant concern for landslide damage to this building site. The hillside above the building site has graded areas that exceed 100% slope for short distances and for somewhat longer distances (more than 20 feet) some slopes exceed 80%. Normally where soil is determined to be stable, graded slopes are not állowed to exceed 50%. A letter is required, bearing the signature and seal of the geotechnical engineer, which provides design criteria for stabilizing the hillside so the building site is reasonably protected from landslide damage or provides criteria for protection such as a debris barrier to keep the building site reasonably protected from landslide damage. This would include protection from material, above the site property line, that might come down to the house. Consideration of landslide danger to a building site does not end at the property line. If the geotechnical engineer feels strongly that there is no soil stability problem or landslide danger that could damage the building, then a very specific letter (bearing the engineer's signature and seal), must be submitted, that states the stability of the soil uphill of the house has been studied and has been found to be reasonably stable and there is no significant landslide danger to the proposed house. If such a letter is submitted and found to be satisfactory, by the Town, then the requirement for soil stabilization or debris barrier construction will not apply. For reference, the geotechnical engineer may want to look at the nearby 39 Wood Lane site where there was a house prior to the 2005 landslide at that location.

Stormwater pollution prevention requirements must be satisfied. Normally a Stormwater Pollution Prevention Plan must be submitted. Since the building area is very flat and no significant grading is proposed, this plan requirement can be satisfied by adding notes to the landscaping plan in the submitted plan set. Add a note stating that "The work shall comply with the requirements of the Marin County Stormwater Pollution Prevention Program, Minimum Erosion/Sediment Control Measures For Small Construction Projects (2 pgs. of details, see – website of the Marin County Stormwater Pollution Prevention Program)", Add a note stating "The work shall comply with the current State Water Resources Control Board requirements and that the work must satisfy Fairfax Town Code Section 8.32 and 17.072.090 requirements". An additional requirement is that the project Civil Engineer or the project Geotechnical Engineer must visit the project site on a regular basis, during the winter months, to confirm that the erosion and sediment control improvements are in place and are adequate.

I recommend that the processing of this project be delayed until the above, noted, information is provided.

Ray Wrysinski

Ray Wrysinski, P. E.
Town Engineer