HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

May 18, 2021
Project Number 2764-02-21

Mr. Coby Friedman
96 Forrest Avenue
Fairfax, California 94930

RE: Geotechnical Report Update
Proposed Residence
79 Wood Lane
Fairfax, California

Dear Mr. Friedman:

This presents the results of our geotechnical report update for the proposed residence at 79 Wood Lane in Fairfax, California. We previously performed a geotechnical investigation for the project and summarized results in our report dated July 29, 2016. Our current work is being provided in accordance with the terms and conditions outlined in our professional services agreement dated April 26, 2021.

GEOTECHNICAL REPORT UPDATE

We conclude that the geotechnical design criteria presented in our July 29, 2016 report are applicable to the proposed project with the following modifications:

Foundation Support

As outlined in our July 29, 2016 report, the site is blanketed by varying thicknesses of relatively weak and compressible fills and native soils which are subject to differential settlement due to foundation loading. Although the deposits encountered in our borings at and near the site generally contained sufficient fine grained materials (silt and clay), or are dense enough to preclude liquefaction. Localized discontinuous layers of cleaner sands subject to localized liquefaction may be present within the deeper alluvium. We therefore judge that a few inches of liquefaction settlement should be anticipated. Empirical correlations developed by Ishihara (1985) indicate that the thickness of the soil cap overlying potentially liquefiable deposits at this site should be adequate to prevent surface manifestations (such as sand boils or ground loss) due to liquefaction of the underlying layers. Mitigating the risk of differential movement under static loading and/or seismic settlement would necessitate extending foundation support into underlying competent soils with either drilled, cast-in-place, reinforced concrete piers or helical piers. The owner has chosen to accept the risk of future maintenance and/or repairs associated with

ATTACHMENT C

differential movement of a shallow mat foundation system. Supplemental geotechnical criteria for a mat foundation system are presented below:

Site Preparation and Grading

Following demolition of existing structures, existing foundations should be removed and areas to be developed should be cleared of designated trees, tree roots, brush and deleterious material. The area to be developed should be stripped of the upper soils containing root growth and organic matter. The cleared materials and strippings should be removed from the site. Pipes, septic tanks, leach fields, and other buried objects should be removed, and the resultant voids cleaned and backfilled with approved fill which is placed and compacted as outlined below.

In and within 3 horizontal feet of planned mats, overexcavation should extend at least 12 inches below both existing grade and at least 12 inches below planned mat subgrade elevation. Additional overexcavation may be required depending on conditions observed by our representative in the field during construction. The depth and extent of required overexcavation should be approved in the field by Herzog Geotechnical prior to placement of fill or improvements.

Soils exposed by required excavations should be scarified to a depth of at least 8 inches, moisture conditioned to near optimum moisture content, and recompacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 test procedure. Optimum moisture content is the water content of the soil (percentage by dry weight) corresponding to the maximum dry density.

Where soft or yielding conditions are encountered, it may be necessary to deepen overexcavations and to blanket the bottom of the overexcavated surface with an approved geotextile stabilization fabric (Mirafi 500X, or equivalent). The depth and extent of required overexcavations and the requirement for stabilization fabric should be evaluated in the field by Herzog Geotechnical during construction. Trucks or construction equipment can cause "pumping" and damage of weak and wet subgrade soils, and can cause a substantial increase in the amounts of overexcavation required. The contractor should not operate trucks or equipment on deflecting areas. Excavation within soft areas should be performed from unexcavated perimeter areas using an excavator. In areas where yielding is encountered, additional overexcavation, stabilization fabric, and select granular imported material may be required.

We anticipate that with the exception of organic topsoils, on-site soils will be suitable for reuse as engineered fill. Lumps greater than 4 inches in largest dimension and perishable materials should be removed, and the fill materials should be approved by the geotechnical engineer prior to use. Imported fill should consist of clean well-graded soil with little or no potential for expansion. The fill material should have a plasticity index of 15 percent or less, and a maximum liquid limit of 40 percent. Herzog Geotechnical should observe and approve imported fill material before it being brought to the site.

Approved fill material should be placed in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned to within 3 percent of optimum moisture content, and recompacted to at least 90 percent relative compaction. Mat subgrade should be smooth and unyielding.

Mat Design Criteria

Mat foundations should be at least 12 inches thick, and should be designed using an allowable bearing capacity of 1000 pounds per square foot (psf), and a modulus of subgrade reaction of 10 pounds per cubic inch (pci). Mats should be designed to span at least a 6 foot square zone of non-support and to cantilever at least 3 feet square zones of non-support at building edges under full dead load. Resistance to lateral forces can be obtained using a passive equivalent earth pressure of 150 pounds per cubic foot (pcf) and a soil friction factor of 0.30 times net vertical dead load. If unacceptable settlement occurs in the future, the mat may re-leveled by mud-jacking. Therefore, it would be prudent to design mats to be capable of resisting uplift stresses in the event that future mud-jacking is required.

Mat subgrade should be sloped to drain into a 12 inch deep trench excavated beneath the middle of the mat. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid-perforated PVC or ABS (Schedule 40, SDR 35 or equivalent) pipe should be placed on a 1-inch layer of drain rock at the bottom of the trench with perforations down. The trench should be backfilled with drain rock up to slab subgrade elevation. The filter fabric should be wrapped over the top of the drain rock. The pipe should be sloped to drain by gravity to a non-perforated pipe which discharges at an approved outlet. The trench for the non-perforated pipe should be backfilled with properly compacted soil.

Mats should be underlain by a capillary moisture break consisting of at least 4 inches of free-draining crushed rock or gravel at least 1/4 inch, and no larger than 3/4 inch, in size. Moisture vapor detrimental to floor coverings or stored items will condense on the underside of the mat. A moisture vapor barrier should therefore be installed over the capillary break. The barrier should be specified by the mat designer. It should be noted that conventional concrete mat construction is not waterproof. The local standard under-slab construction of crushed rock and vapor barrier <u>will</u> <u>not</u> prevent moisture transmission through the mat. Where moisture sensitive floor coverings are to be installed, a waterproofing expert and/or the flooring manufacturer should be consulted for their recommended moisture and vapor protection measures, including moisture barriers, concrete admixtures and/or sealants.

Seismic Design Criteria

Probabilistic seismic hazard analyses were performed to define site-specific design earthquake motions as specified in the *California Building Code* (2019) and Chapter 21 of ASCE 7-16. The analyses were performed using the USGS *Unified Hazard Tool*, and consisted of evaluating Maximum Considered Earthquake (MCE) spectral ground motions with a 2% probability of being exceeded in 50 years. The analyses were performed based on an average shear wave velocity within the upper 30 meters of 259 meters per second, corresponding to a Site Class D as defined in Table 20.3-1 of ASCE 7-16. The probabilistic values at each spectral period were then adjusted to Risk Targeted Ground Motions (MCE_R) using the USGS *Risk-Targeted Ground Motion Calculator*, and scaled to their maximum rotated component (RotD100) using the methodology outlined by Shahi and Baker (2013). The calculated risk targeted values were then scaled by 2/3 to develop the design spectral response acceleration (S_a). Per code requirements, the design spectral response acceleration at each period exceeds 80 percent of S_a determined in accordance with ASCE-7-16. The probabilistic design spectra were compared to deterministic spectra performed utilizing the four published attenuation relationships outlined previously. The 84th percentile of the maximum deterministic ground motion was utilized to evaluate MCE ground motions, and these values were scaled to their RotD100 component (Shahi and Baker, 2013) to determined Sa_M values. Per section 21.2.3 of ASCE-7-16, the site specific spectral response at each period was taken as the lesser of the MCE acceleration values calculated from the probabilistic and deterministic methods. Resultant site-specific coefficients for use in equivalent lateral force (ELF) and modal response spectrum analysis (MRSA) procedures are presented below:

Site Class	D
Site Coefficient F_a	1.0
Site Coefficient F_v	1.7
0.2 sec Spectral Acceleration S_S	1.50
1.0 sec Spectral Acceleration S_1	0.60
0.2 sec Max Spectral Response S_{MS}	1.78
1.0 sec Max Spectral Response S_{M1}	2.05
0.2 sec Design Spectral Response S_{DS}	1.19
1.0 sec Design Spectral Response S_{D1}	1.37
Design Category	D

A site-specific response spectrum can be developed upon request.

Per *Exception #2* in Section 11.4.8 of *ASCE 7-16*, site-specific criteria need not be used for structures on Class D sites with $S_1 \geq 0.2$, provided the value of the seismic response coefficient C_S is determined by Equation 12.8-2 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $1.5T_S \leq T \leq T_L$ or Equation 12.8-4 for $T > T_L$.

SUPPLEMENTAL SERVICES

Our conclusions and recommendations are contingent upon Herzog Geotechnical being retained to review the project plans and specifications to evaluate if they are consistent with our recommendations, and being retained to provide observation and appropriate field and laboratory testing during demolition, site clearing, slab and mat subgrade overexcavation, scarification and recompaction, fill and backfill placement and compaction, pier drilling, concrete placement in cased and/or dewatered pier holes, helical pier installation and load testing, tieback drilling, installation and load testing, and subdrainage installation. We should also be notified to observe the completed project. Steel, concrete, surface drainage, and waterproofing should be inspected by the appropriate party, and are not part of our work.

We should be notified at least 48 hours before the beginning of each phase of work requiring our observation, and upon resumption after interruptions. These services are performed on an as-requested basis. We cannot provide comment on conditions, situations or stages of construction that we are not notified to observe.

LIMITATIONS

This report update has been prepared for the exclusive use of Mr. Coby Friedman and his consultants for the proposed project described in this report. Services performed by Herzog Geotechnical have been conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession practicing in the same locality under similar conditions at the time the services were provided. No other representation, expressed or implied, and no warranty or guarantee is included or intended in this letter or in any opinion, documented or otherwise. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

Our work did not include an environmental assessment or an investigation of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, groundwater or air, on or below, or around the site, nor did it include an evaluation or investigation of the presence or absence of wetlands. Our work also did not address the evaluation or mitigation of mold hazard at the site.

We trust this provides the information required at this time. If you should have further questions, please call.

Sincerely,
HERZOG GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
CRAIG W. HERZOG
No. 002383
GEOTECHNICAL
STATE OF CALIFORNIA

Craig Herzog, G.E.
Principal Engineer

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

AUG 31 2020

February 26, 2018
Project Number 2764-01-16

Ms. Caitlin Evans
6732 Second Avenue NW
Seattle, WA 98117

RE: Geotechnical Report Update and Review of Civil Plans
79 Wood Lane
Fairfax, California

Dear Ms. Evans:

This presents the results of our report update and geotechnical review of the civil plans for the proposed the proposed residence at 79 Wood Lane in Fairfax, California. We previously provided geotechnical recommendations for the project in our report dated July 29, 2016.

REPORT UPDATE

Based on our review, we conclude that the geotechnical design criteria presented in our July 29, 2016 report is applicable to the proposed project with the following modification:

Seismic Design Criteria

The following updated seismic design criteria were developed in accordance with the *California Building Code* (2016) and *ASCE 7-10* (July 2013 errata):

Site Class	D
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5
0.2 sec Spectral Acceleration S_S	1.50
1.0 sec Spectral Acceleration S_1	0.65
0.2 sec Max Spectral Response S_{MS}	1.50
1.0 sec Max Spectral Response S_{M1}	0.98
0.2 sec Design Spectral Response S_{DS}	1.00
1.0 sec Design Spectral Response S_{D1}	0.65
Seismic Design Category	D

GEOTECHNICAL PLAN REVIEW

We reviewed Sheets 1 through 3 of the civil plans by ILS Associates, Inc. dated February 16, 2018 (*Design Review*). Based on our review, we conclude that these plans are in general conformance with the intent of our geotechnical recommendations.

SUPPLEMENTAL SERVICES

We should review the building permit-level structural and civil plans and specifications to evaluate if they are consistent with our recommendations. The contractor should closely review the recommendations presented in the soils report prior to commencing work. During construction, we should provide to provide observation and appropriate field and laboratory testing during demolition, site clearing, slab subgrade overexcavation, scarification and recompaction, fill and backfill placement and compaction, pier drilling, concrete placement in cased and/or dewatered pier holes, helical pier installation and load testing, tieback drilling, installation and load testing, and subdrainage installation. We should also be notified to observe the completed project. Steel, concrete, surface drainage, and waterproofing should be inspected by the appropriate party, and are not part of our work. We should be notified at least 48 hours before the beginning of each phase of work requiring our observation, and upon resumption after interruptions. We cannot provide comment on conditions, situations or stages of construction that we are not notified to observe.

LIMITATIONS

Services performed by Herzog Geotechnical have been conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession practicing in the same locality under similar conditions at the time the services were provided. No other representation, expressed or implied, and no warranty or guarantee is included or intended in this letter or in any opinion, documented or otherwise.

We trust this provides the information required at this time. If you should have further questions, please call.

Sincerely,
HERZOG GEOTECHNICAL

Craig Herzog, G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
CRAIG W. HERZOG
No. 002383
GEOTECHNICAL
STATE OF CALIFORNIA

Two copies submitted

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

AUG 31 2020

July 29, 2016
Project Number 2764-01-16

Ms. Caitlin Evans
6732 Second Avenue NW
Seattle, WA 98117

RE: Report
Geotechnical Investigation
79 Wood Lane
Fairfax, California

Dear Ms. Evans:

This presents the results of our geotechnical investigation the proposed residence at 79 Wood Lane in Fairfax, California. The scope of our investigation was to review selected geologic references, observe exposed site conditions, drill four borings in the project areas, perform laboratory testing, conduct engineering analyses, and develop geotechnical recommendations for the design and construction of the project. Our scope of work was outlined in our proposal dated July 12, 2016.

PROJECT DESCRIPTION

The project will consist of demolishing an existing house at the site and constructing a new two-story, single family residence. The project is shown on the plans by Peter Cohan, Architect dated October 2015.

WORK PERFORMED

Prior to performing our investigation, we reviewed selected geologic references. We explored the subsurface conditions in the project area to the extent of four test borings ranging between approximately 6 and 18 feet deep. Due to limited access, the test borings were drilled with portable drilling equipment. The approximate locations of the test borings are shown on the attached *Site Plan*, Plate 1.

Our Principal Engineer observed the drilling, logged the subsurface conditions encountered, and collected soil samples for visual examination and laboratory testing. Samples were retrieved using Sprague and Henwood and Standard Penetration Test samplers driven with a 70-pound hammer. Penetration resistance blow counts were obtained by dropping the hammer through a 30-inch free fall. The number of blows was recorded for each 6 inches of sampler penetration. These blow counts were then correlated to equivalent standard penetration resistance blow counts. The blows per foot recorded on the boring logs represent the accumulated number of correlated standard penetration blows that were required to drive the sampler the last 12 inches or fraction thereof.

Logs of the test borings are presented on Plates 2 through 5. The soils encountered are described in accordance with the criteria presented on Plate 6. Bedrock is described in accordance with the *Engineering Geology Rock Terms* presented on Plate 7. The logs depict our interpretation of subsurface conditions on the date and at the depths indicated. The stratification lines on the logs represent the approximate boundaries between soil types; the actual transitions may be gradational.

Selected samples were laboratory tested to determine their moisture content and dry density. Laboratory test results are posted on the boring logs in the manner described on the *Key to Test Data*, Plate 6.

FINDINGS

Surface Conditions

The site is located on the southeastern side of Wood Lane in Fairfax, California. The northwestern portion of the site is relatively flat, and supports a single-story, wood-framed residence and a level orchard. The southeastern portion of the site is a hillside which extends up towards the southeast at inclinations of between approximately 1-1/2:1 and 2:1. An approximately 10-foot high, 1-1/2:1 bank has been cut into the toe of the hillside. The base of the cut is supported by a concrete retaining wall which ranges to several feet high. During our investigation we noted topography on the hillside above the cut bank indicative of old slump-type landsliding (see Plate 1), and a swale area containing possible debris flow deposits extending down to the neighboring property to the southwest.

Subsurface Conditions

Previous geologic mapping by Rice (1976) indicates that the northwestern portion of the site is blanketed by Quaternary aged alluvial soils comprised of sand, silt, clay and gravel which were deposited by stream processes. The mapping indicates the hillside within the southeastern portion of the site to be underlain by bedrock of the Franciscan Melange. The Melange unit is Jurassic to Cretaceous in age, and typically consists of a heterogeneous mixture of sandstone, sheared shale, metavolcanic rock, serpentinite and chert.

Our test borings encountered fill, colluvium (slopewash), alluvium and bedrock. The fill encountered generally consists of soft sandy and gravelly silt and of loose to medium dense silty gravel. The colluvium encountered generally consists loose silty and clayey sand and gravel which washed down from upslope areas. The alluvium encountered consists of medium stiff sandy clay and of medium dense to dense clayey gravel. The fill, colluvium and upper portions of the alluvial deposits are of low expansion potential and are relatively weak and compressible. Our borings indicate that the alluvial deposits encountered become denser and less compressible with increasing depth. Bedrock encountered in the borings generally consists of moderately hard sandstone with interbedded shale.

The approximate test boring locations are shown on the *Site Plan* (Plate 1). The test borings encountered the following profiles:

TABLE 1

	Depth (feet)				Depth to Supporting
Boring	**Fill**	**Colluvium**	**Alluvium**	**Bedrock**	**Material (ft.)**
B-1	0-6.5	6.5-9.0	9.0-18.0+	---	15.0
B-2	0-5.5	5.5-9.5	9.5-18.0+	---	14.0
B-3	0-3.5	3.5-9.0	---	9.0-10.0+	9.0
B-4	---	0-5.5	---	5.5-6.0+	5.5

Descriptions of the subsurface conditions encountered are presented on the boring logs.

Groundwater

Wet to saturated conditions were encountered in Borings 1 and 2 below depths of approximately 14 feet. Free groundwater did not develop in the remaining borings prior to backfilling. Groundwater levels at the site are expected to fluctuate over time due to variations in rainfall and other factors.

GEOLOGIC AND SEISMIC HAZARDS

Slope Stability

Regional mapping by Rice (1976) does not indicate the presence of previous landsliding at the site, and a map by Davenport (1984) of slope failures resulting from the severe 1982 storms does not indicate that sliding was reported near the site at that time. The slopes at the site lie within Slope Stability Zone 3 as defined in *"Geology for Planning: Central and Southeast Marin County"* (Rice, 1976). Zone 3 includes areas where steepness of slopes approach the stability limits of the underlying geologic materials. The zones range from 1 to 4, with Zone 4 indicating least stable.

During our investigation we noted topographic features on the hillside above the project indicative of old instability (see Plate 1). We judge that renewed sliding of these areas may occur as a result of heavy rainfall and/or earthquake shaking, and that the slide debris could impact proposed improvements. It will be necessary to provide debris catchment facilities upslope of improvements as outlined in this report.

Fault Rupture

The property is not within a current Alquist-Priolo Earthquake Fault Zone (EFZ), and we did not observe geomorphic features that would suggest the presence of active faulting at the site. As such, we judge that the risk of ground rupture along a fault trace is low at this site.

Ground Shaking

The San Francisco Bay Region has experienced several historic earthquakes from the San Andreas and other associated active faults. Mapped active faults (those experiencing surface rupture within the past 11,000 years) nearest the site are summarized in the following table.

Fault	Distance		Moment Magnitude[1]	Acceleration (g)[2]	
	Miles	Kilometers		M[3]	M+1 [3]
San Andreas (Northern)	6.4	10.3	7.9	0.45	0.76
Seal Cove/San Gregorio	7.1	11.4	7.5	0.29	0.67
Hayward	11.7	18.9	7.1	0.24	0.43
Healdsburg/Rodgers Creek	15.5	24.9	7.0	0.21	0.37

(1) Estimated maximum magnitudes from CDMG (1996) Open File Report 96-08, and Cao et al. (2002).
(2) Peak ground acceleration averaged from New Generation Attenuation (NGA-West 2) relationships by Abrahamson, Silva & Kamai (2104), Boore, Stewart, Seyhan and Atkinson (2014), Campbell and Bozorgnia (2014), Chiou and Youngs(2014), and Idriss (2014). Estimated shear wave velocity (V_{S30}) = 255 m/s.
(3) M = mean value; M+1 = mean+1 standard deviation value.

Deterministic information generated for the site considering the proximity of active faults and estimated ground accelerations are presented in the table above. The estimated ground accelerations were derived from the above-referenced mean attenuation relationships, and are based on the published estimated maximum earthquake moment magnitudes for each fault, the shortest distance between the site and the respective fault, the type of faulting, and the estimated shear wave velocities of the on-site geologic materials. The deterministic evaluation of the potential for ground shaking assumes that the anticipated maximum magnitude earthquake produces fault rupture at the closest proximity to the site, and does not take recurrence intervals or other probabilistic effects into consideration. This evaluation also does not consider directivity effects or other phenomena which may act to amplify ground motions.

Data presented by the Working Group on California Earthquake Probabilities (USGS, 2015) estimates the chance of one or more large earthquakes (Magnitude 6.7 or greater) in the San Francisco Bay region within the next 30 years to be 72 percent. Consequently, we judge that the site will likely be subject to strong earthquake shaking during the life of the improvements.

Liquefaction/Densification

During ground shaking from earthquakes, liquefaction can occur in saturated, loose, cohesionless sands. The occurrence of this phenomenon is dependent on many factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and position of the ground water table (Idriss and Boulanger, 2008). The soils encountered in our test borings contain a high percentage of fine grained materials (silt and clay). Thus, we judge that the likelihood of liquefaction during ground shaking is low.

Densification can occur in low density, uniformly-graded sandy soils above the groundwater table. We judge that significant densification is unlikely to occur in the areas explored because of the relative dense condition and/or high silt and clay content of the soils encountered in the test borings.

CONCLUSIONS

Foundation Support

Our test borings indicate that the project areas are underlain by varying thicknesses of relatively weak fills and native soils which are subject to differential settlement under foundation and fill loads. We judge that shallow footing foundations supported in these materials would be subject to erratic differential movements. It will therefore be necessary to support improvements on helical piers or drilled, cast-in-place reinforced concrete piers which extend through the weak deposits and into underlying bedrock or approved firm alluvium. Helical piers should only be used in level areas. The use of drilled piers will likely require dewatering and possibly installing and removing casing to prevent caving. We estimate that differential settlements of foundations designed in accordance with the recommendations contained in this report will be on the order of half an inch.

Slab Support

To avoid differential settlement, interior and other settlement sensitive slabs should be structural slabs designed to span between pier foundations.

Exterior slabs-on-grade, walkways, and other elements supported on the ground surface will be subject to differential movement. Settlement of exterior slabs can be reduced, but not eliminated, by overexcavating at least the upper 18 inches of weak and porous soils beneath and within 3 horizontal feet of slab subgrade, scarifying and recompacting soils exposed by overexcavation, and replacing the excavated materials as properly compacted fill. If desired, improved performance may be obtained by deepening the depth of overexcavation and replacement.

Seasonal high moisture contents of some near surface soils may cause soft "pumping" conditions during grading operations which may require additional overexcavation, geotextile reinforcement, and imported granular fill. To reduce the risks of such costly special construction methods, it would be prudent to perform site grading during the late summer and early fall months, to perform excavation of soft areas from the unexcavated perimeter using an excavator, and to restrict heavy trucks or equipment from soft subgrade soils.

Retaining Walls

Retaining walls should be supported on drilled pier foundations which extend into bedrock or approved competent soils. Walls should be provided with adequate backdrainage to prevent hydrostatic buildup.

Debris Catchment

As outlined previously, it will be necessary to provide slide debris catchment facilities upslope of improvements. The catchment should consist of a high-energy, ring net barrier (GeoBrugg®, or equivalent) or of a structural wall or fence. It will be necessary to periodically remove accumulated material from behind the barrier to maintain adequate catchment. Due to the high lateral forces of slide debris, occasional repair of barrier damage is a possibility. However, damage potential to downslope improvements from the slide debris should be substantially reduced.

Geotechnical Drainage

It is important that water be conducted away from improvements in order to reduce moisture changes in the weak on-site soils. Runoff from the hillside above the project should be intercepted with lined swales and conducted to an approved outlet well away from improvements. Perimeter subdrains should be provided to reduce water infiltration beneath the structure, and roofs should be provided with gutters and downspouts. All drains and downspouts must be collected in closed conduits and discharged at an approved erosion resistant outlet well away from improvements.

RECOMMENDATIONS

Seismic Design

Based on the results of our investigation, the following seismic design criteria were developed in accordance with the *California Building Code* (2012) and *ASCE 7-10* (July 2013 errata):

Site Class	D
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5
0.2 sec Spectral Acceleration S_S	1.50
1.0 sec Spectral Acceleration S_1	0.65
0.2 sec Max Spectral Response S_{MS}	1.50
1.0 sec Max Spectral Response S_{M1}	0.98
0.2 sec Design Spectral Response S_{DS}	1.00
1.0 sec Design Spectral Response S_{D1}	0.65
Seismic Design Category	D

Site Preparation and Grading

Areas to be developed should be cleared of structures, trees, vegetation and deleterious material, and then stripped of the upper soils containing root growth and organic matter. The cleared materials and strippings should be removed from the site. Tree roots, foundations, vaults, pipes, septic tanks, leach fields and other buried objects should be removed, and the resultant voids cleaned and backfilled with engineered fill.

In and within 3 horizontal feet of planned non-structural exterior slabs, overexcavation should extend at least 18 inches below both existing grade and at least 18 inches below planned subgrade elevation. Additional overexcavation may be required depending on conditions observed by our representative in the field during construction. The depth and extent of required overexcavations should be approved in the field by Herzog Geotechnical prior to placement of fill or improvements.

Except where saturated, soft or yielding soils are encountered, the soils exposed by required excavations should be scarified to a depth of at least 8 inches, moisture conditioned to near optimum moisture content, and recompacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 test procedure. Optimum moisture content is the water content of the soil (percentage by dry weight) corresponding to the maximum dry density.

Where saturated, soft or yielding conditions preclude scarification and recompaction, it will be necessary to deepen overexcavation at least 8 inches, to blanket the bottom of the overexcavated surface with an approved geotextile stabilization fabric (Mirafi 500X, or equivalent), and to provide an at least 12 inch layer of compacted granular imported fill above the fabric. The depth and extent of required overexcavations and the requirement for stabilization fabric should be evaluated in the field by Herzog Geotechnical during construction. Trucks or construction equipment can cause soil deflection (pumping) and damage of weak and wet subgrade soils, and can substantially increase the amounts of overexcavation required. The contractor should not operate trucks or equipment on deflecting areas. Excavation within soft areas should be performed from unexcavated perimeter areas using an excavator. In areas where soil deflection is encountered, additional overexcavation and select granular imported material may be required. If seepage is encountered, it will be necessary to provide subdrainage as recommended by our representative in the field.

We anticipate that with the exception of organic topsoils, on-site soils will be suitable for reuse as engineered fill. However, considerable moisture conditioning of materials may be required. Lumps greater than 4 inches in largest dimension and perishable materials should be removed, and the fill materials should be approved by the geotechnical engineer prior to use. Imported fill should consist of clean well-graded soil with little or no potential for expansion. The fill material should have a plasticity index of 15 percent or less, and a maximum liquid limit of 40 percent. Herzog Geotechnical should observe and approve imported fill material before it being brought to the site.

Approved fill material should be placed in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned to within 3 percent of optimum moisture content, and recompacted to at least 90 percent relative compaction. Slab subgrade should be smooth and unyielding.

Foundation Support

Helical Piers

Helical piers should only be used in level areas, and should consist of end bearing Chance Anchors (A.B. Chance Company), or equivalent, which are installed using a rotary type torque motor. The helical piers should be installed and corrosion protected in accordance with the manufacturer's specifications. Helical piers extending into approved competent soils or bedrock should be designed using an allowable bearing capacity of 6000 and 9000 pounds per square foot (psf) for dead plus code live loads, respectively. The depth to bedrock and competent soils may be estimated from Table 1 in the *Subsurface Conditions* section of this report. The actual bearing capacity of the piers should be evaluated based on measured torque values obtained during installation. If the piers are not contracted on a guaranteed design-build basis, load testing should be performed on at least one pier to verify capacity.

Helical piers should be interconnected with grade beams to support structural loads. The portions of grade beams extending more than 12 inches below finished grade may impose a passive equivalent fluid pressure of 150 pounds per cubic foot (pcf) to resist short term lateral forces. Helical pier shafts should not be used to resist lateral loads.

Drilled Piers

Drilled piers should be at least 18 inches in diameter and should extend at least 6 feet into bedrock or at least 8 feet into approved competent soils. The depth to bedrock and competent soils may be estimated from Table 1 in the *Subsurface Conditions* section of this report. Design pier depths and diameters should be calculated by the Project Structural Engineer using the criteria presented below. The materials encountered in the pier excavations should be evaluated by our representative in the field during drilling. The sidewalls of pier holes allowed to remain open may be subject desiccation and deterioration, which adversely impacts skin friction capacity. If concrete is not placed in pier holes within 72 hours of drilling, we should be notified to reevaluate the holes to determine if they need to be reamed out or re-drilled.

Piers should be interconnected with grade beams to support structural loads. Piers and grade beams located on slopes steeper than 5:1 should be designed and reinforced to resist creep forces acting from the finished ground surface to the top of the rock, and exerting an active equivalent fluid pressure of 60 pounds per cubic foot (pcf). For piers, this pressure should be assumed to act on 2 pier diameters.

The portion of the piers extending into approved competent soils can impose a passive equivalent fluid pressure of 300 pounds per cubic foot (pcf) acting over 2 pier diameters, and vertical dead plus real live loads of 500 pounds per square foot (psf) in skin friction. The portion of the piers extending into bedrock can impose a passive equivalent fluid pressure of 450 pcf acting over 2 pier diameters, and vertical dead plus real live loads of 1000 psf in skin friction. These values may be increased by 1/3 for seismic and wind loads, but should be decreased by 1/3 for determining uplift resistance. The portion of piers designed to impose passive pressures should have at least 7 feet of horizontal confinement from the face of the nearest slope or wall. End bearing should be neglected due to the uncertainty of mobilizing end bearing and skin friction simultaneously.

Groundwater will likely be encountered, in which case it will be necessary to dewater the holes and/or to place concrete by the tremie method. Caving soils may be encountered, in which case it will be necessary to case the holes. Casing should be carefully maintained ahead of the drill to avoid causing settlement of adjacent areas. Casing should be removed from the holes simultaneous with concrete placement. Hard drilling or coring may be required to achieve required bedrock penetrations.

Retaining Walls

Retaining walls should be supported in bedrock or approved competent soils on drilled pier foundations designed in accordance with the recommendations presented in this report. Wall facing should be extended at least 12 inches below finished downslope grade. Free-standing retaining walls should be designed to resist active lateral earth pressures equivalent to those exerted by a fluid weighing 45 pounds per cubic foot (pcf) where the backslope is level, and 60 pcf for backfill at a 2:1 slope. Retaining walls restrained from movement at the top should be designed to resist an "at-rest" equivalent fluid pressure of 60 pcf for level backfill and 75 pcf for backfill at a 2:1 slope. For intermediate slopes, interpolate between these values. A minimum factor of safety against instability of 1.5 should be used to evaluate static stability of retaining walls.

Seismic wall stability may be evaluated based on a uniform lateral earth pressure of 10xH psf (where H is the height of the wall in feet). This pressure is in addition to the active equivalent fluid pressures presented in the report. For restrained walls, seismic pressures may be assumed to act in combination with active rather than at-rest earth pressures. The factor of safety against instability under seismic loading should be at least 1.1.

In addition to lateral earth pressures, retaining walls must be designed to resist horizontal pressures that may be generated by uphill retaining walls. Where an imaginary 1-1/2:1 (horizontal:vertical) plane projected downward from the base of an upslope retaining wall intersects the downslope wall, that portion of the downslope wall below the intersection should be designed for an additional horizontal uniform pressure equivalent to the maximum calculated lateral earth pressure at the base of the upslope wall.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch diameter, rigid perforated pipe surrounded by a drainage blanket. The top of the drain pipe should be at least 8 inches below lowest adjacent downslope grade. The pipe should be PVC Schedule 40 or ABS with an SDR of 35 or better, and the pipe should be sloped to drain at least 1 percent by gravity to an approved protected outlet. Frequent cleanout risers should be provided for the drains, and sweeps or sanitary wyes should be used to allow for future inspection and maintenance of the drains. The drainage blanket should consist of clean, free-draining crushed rock or gravel wrapped in a filter fabric such as Mirafi 140N. Alternatively, the drainage blanket could consist of Caltrans Class 2 "Permeable Material", in which case the filter fabric may be omitted. A prefabricated drainage structure such as Mirafi Miradrain may also be used provided that the backdrain pipe is embedded in permeable material or fabric-wrapped crushed rock. The drainage blanket should be continuous, at least 1 horizontal foot thick, and should extend to within 1 foot of the surface. The uppermost 1 foot should be backfilled with compacted soil to exclude surface water.

Where migration of moisture through retaining walls would be detrimental or undesirable, retaining walls should be waterproofed as specified by the Architect or Structural Engineer.

Wall backfill should be spread in level lifts not exceeding 8 inches in thickness, brought to near the optimum moisture content, and compacted to at least 90 percent relative compaction. Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building onto or adjacent to the walls, and should be properly braced during the backfilling operations. Backfilling adjacent to walls should be performed only with hand-operated equipment to avoid over-stressing the walls.

Even well compacted backfill will settle about 1 percent of its thickness. Therefore, slabs and other improvements crossing the backfill should be designed to span or to accommodate this settlement.

Interior and Settlement Sensitive Slabs

Interior and other settlement sensitive slabs should be structural slabs designed to span between pier supported foundations.

Interior slab subgrade should be sloped to drain into a 12 inch deep trench excavated beneath the middle of each slab. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid-perforated PVC or ABS (Schedule 40, SDR 35 or equivalent) pipe should be placed on a 1-inch layer of drain rock at the bottom of the trench with perforations down. The trench should be backfilled with drain rock up to slab subgrade elevation. The filter fabric should be wrapped over the top of the drain rock. The pipe should be sloped to drain by gravity to a non-perforated pipe which discharges at an approved outlet. The trench for the non-perforated pipe should be backfilled with properly compacted soil.

Interior slabs should be underlain by a capillary moisture break consisting of at least 4 inches of free-draining, crushed rock or gravel (slab base rock) at least 1/4 inch, and no larger than 3/4 inch, in size. Moisture vapor detrimental to floor coverings or stored items will condense on the undersides of slabs. A moisture vapor barrier should therefore be installed over the capillary break. The barrier should be specified by the slab designer. It should be noted that conventional concrete slab-on-grade construction is not waterproof. The local standard under-slab construction of crushed rock and vapor barrier <u>will</u> <u>not</u> prevent moisture transmission through slab-on-grade. Where moisture sensitive floor coverings are to be installed, a waterproofing expert and/or the flooring manufacturer should be consulted for their recommended moisture and vapor protection measures, including moisture barriers, concrete admixtures and/or sealants.

Exterior Slabs

Exterior non-structural slabs should be underlain by properly compacted fill as outlined in this report. Non-structural slabs should be at least 5 inches thick (or at least 6 inches thick for driveways) and should be reinforced with at least #4 reinforcing bars spaced no more than 12 inches on-center each way to control cracking due to differential movement. Control joints should be provided as determined by the Structural Engineer. Reinforcement should be continuous across joints. Exterior slabs will experience differential movement and should be structurally separated or hinged from pier supported elements to accommodate differential settlements. All slabs should be as designed by the project structural engineer.

Utility Trench Backfill

Trenches should be backfilled with material that is mechanically compacted to at least 90 percent relative compaction. Uncompacted lift thicknesses should not exceed 8 inches. Compaction by jetting should not be permitted. Governmental or public utility requirements exceeding those listed above should govern where applicable.

Debris Barriers

Catchment facilities should be installed upslope of improvements. For planning purposes, catchment should be at least 8 feet high and sized as necessary to provide storage for a debris event of at least 100 cubic yards. The catchment facilities should consist of a dynamic catchment system or of a structural wall or fence. Criteria for these measures are presented below.

Catchment Wall/Fencing

Structural catchment walls and fencing should be designed for an equivalent fluid impact pressure of 125 pounds per cubic foot (pcf). The walls or fencing should be founded on drilled piers designed in accordance with the criteria presented previously.

Dynamic Catchment System

Dynamic catchment fencing should consist of Geogrugg's *UX-050 Protection System* (www.geobrugg.com), or equivalent. The materials and installation for the fence should conform with project-specific shop drawings and specifications by the manufacturer which are prepared and submitted for our review prior to commencing work. Vertical I-beams for the fence should be founded in bedrock and should conform with the requirements outlined previously.

Tiebacks for the system should be inclined downward at an angle of at least 15 degrees from the horizontal. The tiebacks should have minimum unbonded lengths of 10 and 15 feet for bars and strands, respectively. The unbonded length should be extended as necessary to reach the bedrock surface. The allowable skin friction for tiebacks will depend upon drilling method, grout installation pressure, and workmanship. For estimating purposes, the portion of tiebacks grouted into bedrock may be assumed to impose a skin friction value of 2000 psf. The contractor should be responsible for determining the actual length of tiebacks necessary to resist design loads based on the materials encountered and their familiarity with the installation method utilized. Our field engineer should observe conditions during drilling.

Tieback materials, installation, corrosion protection and testing should conform to *Recommendations for Prestressed Rock and Soil Anchors* (Post-Tensioning Institute, latest edition). The tieback bars or strands should be double corrosion protected. The bars or strands should be positioned in the center of the holes, and the bonded length grouted in place from the bottom. If a frictionless sleeve is used over the unbonded length, the bars or strands may be initially grouted over their entire length. When the grout has attained the required compressive strength, one anchor should be subjected to a creep-test the anchors should be proof tested to 1.33 times the design load as outlined by the Post-Tensioning Institute. Proof test loads should be held for 10 minutes, and the deflection at test load between the 1 and 10 minute readings should not exceed 0.04 inches. Replacement tiebacks should be installed for tiebacks that fail the load testing.

Maintenance

The barrier should be periodically inspected for damage, and maintained and repaired as necessary. Clear storage space should be provided and maintained upslope of the barrier. The catchment area behind the barrier accessible for maintenance, and should be cleaned out following each slide episode and annually prior to the winter rains. In addition, it may be necessary to remove fines that migrate through the barrier.

Geotechnical Drainage

Positive drainage should be provided away from foundations, walls and flatwork. Ponding of surface water should not be allowed. Runoff from upslope areas should be intercepted with a lined swale, and drop inlets should be provided at low points as necessary to prevent ponding of surface water. Provisions should be made for fail-safe drainage away from the residence to prevent flooding in the event that the drains become clogged. All roofs should be provided with gutters and downspouts. Downspouts should be connected into closed conduits which discharge at an approved outlet well away from improvements. Conduit should consist of rigid PVC or ABS pipe which is Schedule 40, SDR 35 or equivalent. Downspouts, surface drains and subsurface drains should be checked for blockage, and cleared and maintained on a regular basis. Surface drains and downspouts should be maintained entirely separate from foundation drains.

Foundation drains should be installed adjacent to all perimeter foundations. The drains should consist of trenches which extend 18 inches deep, or 12 inches below lowest adjacent interior or crawl space grade, whichever is deeper, and which are sloped to drain at least 1 percent by gravity. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid perforated PVC or ABS pipe (Schedule 40, SDR 35 or equivalent) should be placed on a 1-inch thick layer of drain rock at the bottom of the trenches with perforations down. Frequent cleanout risers should be provided for the drains, and sweeps or sanitary wyes should be used to allow for future inspection and maintenance of the drains. The pipes should be sloped to drain at least 1 percent by gravity to a non-perforated pipe (Schedule 40, SDR 35 or equivalent) which discharges at an approved outlet. The trench for the perforated pipe should be backfilled to within 6 inches of the ground surface with drain rock. The filter fabric should be wrapped over the top of the drain rock. The upper 6 inches of the trenches should be backfilled with compacted clayey soil to exclude surface water. The trench for the non-perforated outlet pipe should be completely backfilled with compacted soil.

Water will accumulate in crawl spaces. The crawl spaces should be graded to create a smooth sloping surface, and covered with an approved pre-fabricated drainage material such as Mirafi Miradrain 6000. A 4-inch diameter, perforated Schedule 40 or SDR 35 pipe should be provided in a trench excavated extending across the lowest portion of the crawl space. The trench should extend 12 inches deep, and should be sloped to drain at least 1 percent by gravity. The trench should be completely lined with Mirafi 140N filter fabric, or equivalent. The perforated pipe should slope to drain at least 1 percent to a non-perforated Schedule 40 or SDR 35 pipe which discharges at an approved outlet. The surface and trench should then be covered with reinforced gunite.

Supplemental Services

Herzog Geotechnical's conclusions and recommendations are contingent upon our being retained to review the project plans and to provide construction observations services. We should review the project plans and specifications to evaluate if they are consistent with our recommendations. In addition, we should be retained to provide intermittent observation and testing during demolition, site clearing, slab subgrade overexcavation, scarification and recompaction, fill and backfill placement and compaction, pier drilling, concrete placement in cased and/or dewatered pier holes, helical pier installation and load testing, tieback drilling, installation and load testing, and subdrainage installation. We should also be notified to observe the completed project. Steel, concrete, surface drainage, and waterproofing should be inspected by the appropriate party, and are not part of our work.

If during construction subsurface conditions different from those described in this report are observed, or appear to be present beneath excavations, we should be advised at once so that these conditions may be reviewed and our recommendations reconsidered. The recommendations made in this report are contingent upon our being notified to review changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, the recommendations of this report may no longer be valid or appropriate. In such case, we recommend that we review this report to determine the applicability of the conclusions and recommendations considering the time elapsed or changed conditions. The recommendations made in this report are contingent upon such a review.

We should be notified at least 48 hours before the beginning of each phase of work requiring our observation, and upon resumption after interruptions. These services are performed on an as-requested basis and are in addition to this geotechnical reconnaissance. We cannot provide comment on conditions, situations or stages of construction that we are not notified to observe.

LIMITATIONS

This report has been prepared for the exclusive use of Ms. Caitlin Evans and her consultants for the proposed project described in this report.

Our services consist of professional opinions and conclusions developed in accordance with generally-accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided us regarding the proposed construction, the results of our field exploration and laboratory testing programs, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The test boring logs represent subsurface conditions at the locations and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration and may not necessarily be the same or comparable at other times. The location of the test borings was established in the field by reference to existing features, and should be considered approximate only.

There is an inherent risk of instability associated with all hillside area construction. We therefore recommend that the owner obtains appropriate landslide and earthquake insurance.

The scope of our services did not include an environmental assessment or an investigation of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, ground water or air, on or below, or around the site, nor did it include an evaluation or investigation of the presence or absence of wetlands. Our work also did not address the evaluation or mitigation of flood or mold hazard at the site.

We appreciate the opportunity to be of service to you. If you have any questions, please call us at (415) 388-8355.

Sincerely,
HERZOG GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
CRAIG W. HERZOG
No. 002383
GEOTECHNICAL
STATE OF CALIFORNIA

Craig Herzog, G.E.
Principal Engineer

Attachments: References
Plates 1 - 7

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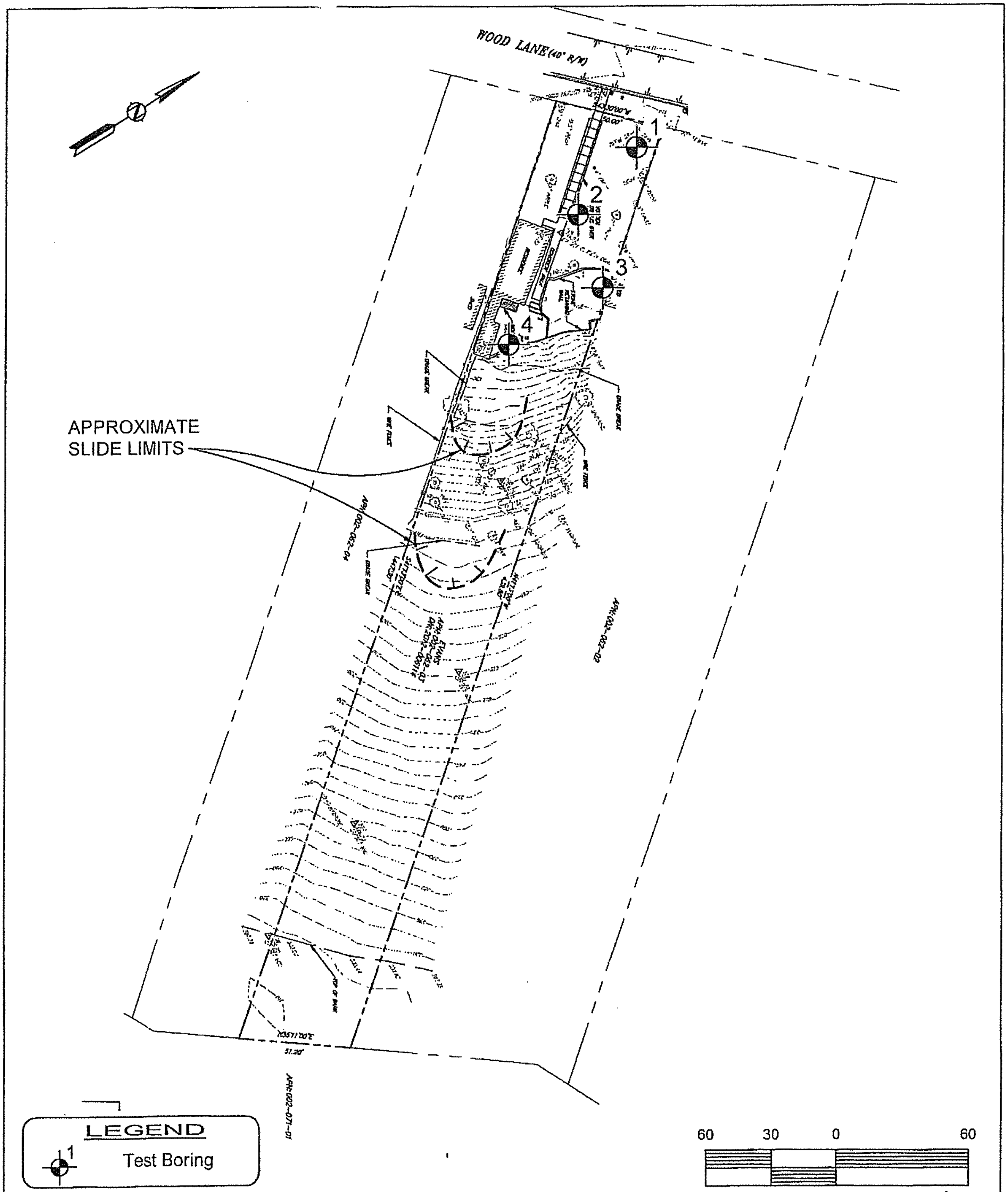

Reference: Topographic Map by ILS Associates, Inc., dated 5/21/14.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS	Job. No: 2764-01-16 Appr: Drwn: LPDD Date: JUL 2016	SITE PLAN 79 Wood Lane Fairfax, California	PLATE 1

EQUIPMENT: 4" Flight Auger | ELEVATION: * *
LOGGED BY: C.H. | START DATE: 7-21-16
FINISH DATE: 7-21-16

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	Description
							0	GRAY-DARK BROWN GRAVELLY SILT (ML), soft, dry (Fill)
							2	GRAY-BROWN SILTY GRAVEL (GM), loose to medium dense, dry (Fill)
		13.5	95		10			
							6.5	YELLOW-BROWN SILTY SAND (SM), loose, dry to moist
		16.5	96		5			
							9	MOTTLED GRAY-BROWN SANDY CLAY (CL), medium stiff, moist
		19.4	108		20			
					11			
					20			
							14.5	MOTTLED YELLOW-ORANGE-BROWN CLAYEY GRAVEL WITH SAND (GC), medium dense to dense, wet to saturated
					25			
					24			
							18	BOTTOM OF BORING 1 @ 18 FEET Hole backfilled prior to groundwater stabilized.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

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Drwn: LPDD
Date: JUL 2016

LOG OF BORING 1

79 Wood Lane

Fairfax, California

PLATE 2

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 7-21-16
FINISH DATE: 7-21-16

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	DEPTH (FEET)	Description
						0	GRAY-BROWN SANDY SILT (ML), soft, dry, with roots (Fill)
		13.2	85		9	1.5	
						5.5	YELLOW-BROWN CLAYEY SAND (SC), loose, moist to wet, with occasional gravel
		12.4	114		8	5.5	
						9.5	DARK GRAY-BROWN SANDY CLAY (CL), medium stiff, moist
		19.4	106		17	9.5	
					24	12	
					28	13.5	
						14	MOTTLED RED-YELLOW-BROWN CLAYEY GRAVEL (GC), medium dense to dense, wet to saturated
					26	15	
					28	16.5	
						18	BOTTOM OF BORING 2 @ 18 FEET

1Hole backfilled prior to groundwater stabilized.

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

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Job No: 2764-01-16
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Drwn: LPDD
Date: JUL 2016

LOG OF BORING 2

79 Wood Lane
Fairfax, California

PLATE 3

EQUIPMENT: 4" Flight Auger

LOGGED BY: C.H.

ELEVATION: **

START DATE: 7-21-16

FINISH DATE: 7-21-16

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	DEPTH (FEET)	Description
						0	BROWN GRAVELLY SILT WITH SAND (ML), loose, dry,porous, with metal debris (Fill)
		10.9	95		17	1.5–2	
						3.5	MOTTLED YELLOW-DARK BROWN SILTY GRAVEL (GM), loose, dry, with angular sandstone fragments
		9.7	106		16	5.5–6	
					20	7.5–9	
					26	9–10	GRAY-BROWN SANDSTONE WITH INTERBEDDED SHALE, moderately hard, weak, highly weathered
						10	BOTTOM OF BORING 3 @ 10 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

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Job No: 2764-01-16

Appr:

Drwn: LPDD

Date: JUL 2016

LOG OF BORING 3

79 Wood Lane

Fairfax, California

PLATE 4

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: 4" Flight Auger LOGGED BY: C.H.	ELEVATION: ** START DATE: 7-21-16 FINISH DATE: 7-21-16
							0	BROWN SILTY SAND WITH GRAVEL (SM), loose, dry	
						X	1		
							2		
						X	3		
						X	4		
							5		
						X	6	GRAY-BROWN SANDSTONE, moderately hard, moderately strong, highly weathered	

BOTTOM OF BORING 4 @ 6 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

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Date: JUL 2016

LOG OF BORING 4

79 Wood Lane

Fairfax, California

PLATE 5

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 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, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

			Shear Strength, psf (Confining Pressure, psf)	
Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Unconsolidated Undrained Direct Shear
PI	Plasticity Index	TV	1320	Torvane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample (2.5-inch ID)	FS	Free Swell	
◪	2-inch-ID Sample	EI	Expansion Index	
◩	Standard Penetration Test	Perm	Permeability	
⊠	Bulk Sample	SE	Sand Equivalent	

KEY TO TEST DATA

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Date: JUL 2016

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

79 Wood Lane

Fairfax, California

PLATE 6

ROCK SYMBOLS

SHALE OR CLAYSTONE	CHERT	SERPENTINITE
SILTSTONE	PYROCLASTIC	METAMORPHIC ROCKS
SANDSTONE	VOLCANIC	DIATOMITE
CONGLOMERATE	PLUTONIC	SHEARED ROCKS

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINNLY BEDDED	2-1/2 to 8 inches
VERY THINNLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2-1/2 to 8 inches
VERY CLOSELY SPACED	3/4 to 2-1/2 inches
EXTREMELY CLOSELY SPACED	Less than 3/4 inch

HARDNESS

SOFT - Pliable; can be dug by hand

FIRM - Can be gouged deeply or carved with a pocket knife

MODERATELY HARD - Can be readily scrached by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visable

VERY HARD - Cannot be scratched with pocket knife; leaves a metallic streak

STRENGTH

PLASTIC - Capable of being molded by hand

FRIABLE - Crumbles by rubbing with fingers

WEAK - An unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - Specimen will withstand a few heavy hammer blows before breaking

STRONG - Specimem will withstand a few heavy ringing hammer blows and usually yields large fragments

VERY STRONG - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

DEGREE OF WEATHERING

HIGHLY WEATHERED - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thourough discoloration, rock disintegration, mineral decomposition

MODERATELY WEATHERED - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

SLIGHTLY WEATHERED - A few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

FRESH - Unaffected by weathering agents, no appreciable change with depth

MPEG MILLER PACIFIC ENGINEERING GROUP

May 25, 2021
File: 201.159cltr.doc

Town of Fairfax
Planning and Building Services Department
142 Bolinas Avenue
Fairfax, California 94930

Attn: Ms. Linda Neal, Principal Planner

Re: Second Planning-Level Geologic, Geotechnical, and Civil Engineering Review
New Single-Family Residence
79 Wood Lane (APN 002-062-03)
Fairfax, California

Introduction

In response to your request and in accordance with our agreement dated March 20, 2018, we have performed a second planning-level review of project plans and supporting documentation for the proposed new single-family residence, attached ADU, and ancillary site improvements at 79 Wood Lane in Fairfax, California. We previously issued comments in our First Review letter dated April 26, 2021. The purpose of our services is to review the submitted documents, comment on the completeness and adequacy of the submittal in consideration of Town requirements, and to provide a recommendation to Town Planning and Building staff regarding project approval.

The scope of our services includes:

- A site reconnaissance to observe existing conditions and review proposed development features;
- Review of provided project documents for conformance to the Town of Fairfax Hill Area Residential Development Ordinance, specifically Town Code Sections 17.072.080(B), (C), (E), and (F), and Section 17.072.110 (C).
- Development of opinions regarding project compliance with applicable Town Code requirements; and
- Development of recommendations to Town staff as to whether the project may be safely constructed in consideration of any geologic, hydrologic, or geotechnical hazards.

It should be noted that the scope of our review is limited solely to geologic, geotechnical, and civil portions of the project, and does not include review of structural, architectural, mechanical, or other items beyond the scope of our qualifications. We recommend that non-geotechnical aspects of the plans be reviewed by suitably qualified professionals.

Project Description

The project generally consists of constructing a new two-story, 1,935 square-foot, 4-bedroom residence with an attached, approximately 700 square-foot Accessory Dwelling Unit (ADU) and detached 1-car garage on an approximately 0.5-acre parcel. An existing 576-foot residence will be demolished to accommodate the new construction. Vehicle access will be provided via a new

ATTACHMENT D

decomposed granite-surfaced driveway extending from Wood Lane, and ancillary improvements will include new underground utilities, exterior flatwork, site retaining walls, and other "typical" residential items.

Project Review

We performed a site reconnaissance on March 15, 2018 to observe existing conditions at the site. Additionally, we previously reviewed the following documents provided by the Town as part of our First Review:

- Herzog Geotechnical (2016), "Report, Geotechnical Investigation, 79 Wood Lane, Fairfax, California", Project Number 2764-01-16, dated July 29, 2016.
- Herzog Geotechnical (2018), "Geotechnical Report Update and Review of Civil Plans, Proposed Residence, 79 Wood Lane, Fairfax, California", Project Number 2764-01-16, dated February 26, 2018.
- ILS Associates (2018), "Record of Survey of the Lands of Stephanie Evans and Patrick Higgins D.N. 2012-006114, Town of Fairfax, Marin County, California", Job No. 8868, dated September 2018.
- First American Title Company (2019) "Preliminary Report, Order No. 2103-6004437, 79 Wood Lane, Fairfax, California 94930" (Title Report), September 6, 2019.
- ILS Associates (2020), "Friedman, 79 Wood Lane, Fairfax, California" (Preliminary Civil Plans), Sheets 1 through 3, Job No. 9473, Design Review set dated July 22, 2020.
- Frederic C. Divine Associates (2021), "New Residence and ADU, 79 Wood Lane, Fairfax, CA 94930, APN 002-062-03", (Preliminary Architectural Plans), Job Number 19049.00, Sheets A1 through A3.2, Planning Comments set dated February 24, 2021.

More recently, we reviewed the following documents for this Second Review:

- Herzog Geotechnical (2018), "Geotechnical Report Update, Proposed Residence, 79 Wood Lane, Fairfax, California", Project Number 2764-01-16, dated February 26, 2018.

Conclusions

Based on our site reconnaissance and document review, it is our opinion that the geotechnical report suitably demonstrates the site may be safely developed, and that all of our Planning-Level comments have been suitably addressed.

Recommendations

We recommend that project processing be continued at the planning level. As stated in his report, the Geotechnical Engineer should also review the project civil and structural plans and provide a letter confirming they sufficiently incorporate the intent of his recommendations at the building level. We should also review the building-level structural and civil plans to confirm they reflect the approved planning documents.

MILLER PACIFIC
ENGINEERING GROUP

May 25, 2021

We trust that this letter contains the information you require at this time. If you have any questions, please call. We will directly discuss our comments with the applicant's consultants if they wish to do so.

Yours very truly,
MILLER PACIFIC ENGINEERING GROUP

CERTIFIED ENGINEERING GEOLOGIST
MICHAEL JEWETT
CEG No. 2610
Exp. 1-31-23
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
SCOTT A. STEPHENS
2398
Exp 6/30/21
GEOTECHNICAL
STATE OF CALIFORNIA

Mike Jewett
Town of Fairfax Contract Geologist
Engineering Geologist No. 2610
(Expires 1/31/23)

Scott Stephens
Town of Fairfax Contract Engineer
Geotechnical Engineer No. 2398
(Expires 6/30/21)

ILS ASSOCIATES, INC.
CIVIL ENGINEERING AND LAND SURVEYING

August 6, 2021

Town of Fairfax
Planning and Building Services
142 Bolinas Road
Fairfax, CA 94930
Attention: Ben Berto, Director

Re: 79 Wood Lane
Certification of Story Poles
Our File No. 9473

Dear Mr. Berto,

On August 6, 2021 at above listed site, our survey field crew checked the locations and the elevations of 6 constructed story poles and found the elevation and position of them to be in substantial conformance with the attached Story Pole Plan sheet dated 7-20-21 prepared by Fredric C. Divine Architects. The elevations of the story pole tops are as shown on the attached Story Pole Cut Sheet prepared by this office dated 8-6-21.

If there is any other information that may be required please contact our office.

Sincerely,

arthur J. Smith

Arthur J. Smith, PLS 9580

LICENSED LAND SURVEYOR
ARTHUR JESSE SMITH
L 9580
STATE OF CALIFORNIA

Cc: Coby Friedman

79 GALLI DRIVE, SUITE A NOVATO, CA 94949-5717 (415) 883-9200

ATTACHMENT E

CUT SHEET

	A–E	F	G	H–I	J–K
1	**ILS ASSOCIATES, INC.**			CALC'D	FIELD
2	**79 GALLI DRIVE**		DATE:	8/6/2021	8/6/2021
3	**NOVATO, CA 94949**		FB #:	247	
4	PHONE (415)883-9200		DC FILE:	9473.crd	
5	FAX (415)883-2763		CREW:	JG/JPG	
6	MEA.EL. = MEASURED ELEVATION		JOB # :	9473 (Ref: 8868)	
7	D.ELEV = DESIGN ELEVATION		JOB NAME:	79 Wood Lane, Fairfax	
8					
9	TYPE OF VERIFICATION:	**STORY POLE CUT SHEET**			
10					

	A	B	C	D	E	F	G	H	I	J	K
11	SP	STORE PT.		DESIGN			CUT / FILL		COMMENT		
12	NO.	NUM	MEA.EL.	D.ELEV	DESC				O/S	NOTE	SET
13											
14	**SP-1**	**1001**	137.27	137.32	**TOP SP**	**F**	**0.05**		0	NORTHMOST	
15	**SP-2**	**1002**	142.09	142.05	**TOP SP**	**C**	**-0.04**		0	CENTER-NORTH	
16	**SP-3**	**1003**	137.59	137.32	**TOP SP**	**C**	**-0.27**		0	WESTMOST	
17	**SP-4**	**1004**	136.99	137.32	**TOP SP**	**F**	**0.33**		0	EASTMOST	
18	**SP-5**	**1005**	142.06	142.05	**TOP SP**	**C**	**-0.01**		0	CENTER-SOUTH	
19	**SP-6**	**1006**	136.90	137.32	**TOP SP**	**F**	**0.42**		0	SOUTHMOST	
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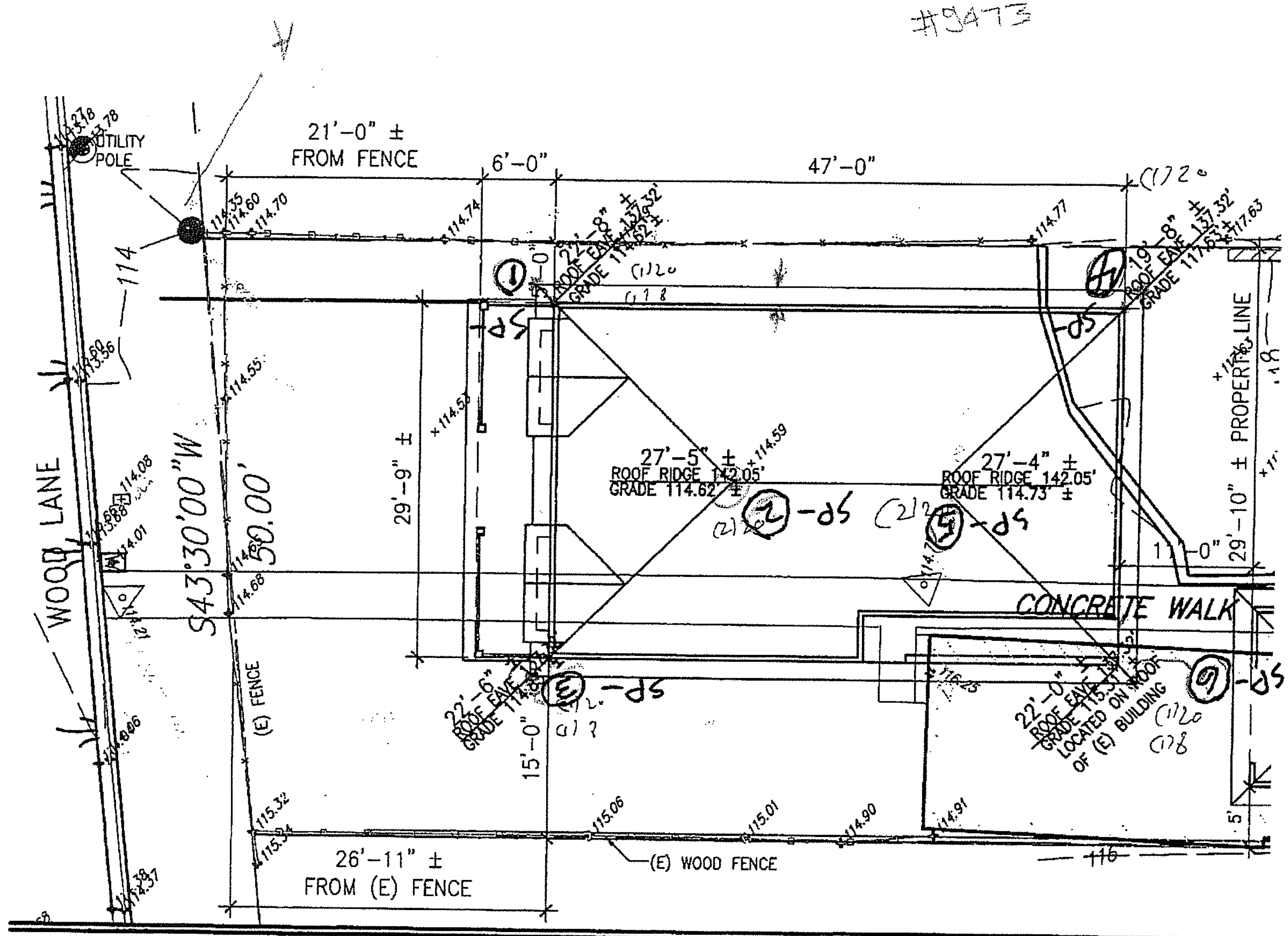


STORY POLE PLAN

79 WOOD LANE, FAIRFAX CA

@ 1" = 10'-0"

(8) 2x4x20
(10) 2x4x8'

07-20-21

TOWN OF FAIRFAX

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

Date: April 28, 2021

Permit #21-T-30

NOTICE OF TREE COMMITTEE ACTION

This action may be appealed to the Fairfax Town Council within 10 days of the Tree Committee decision. *This permit is not in effect until the 10 day appeal period is over.*

Request for a tree permit to remove: (1) Apple
(5) Olive
(1) Live Oak

Address of Tree(s) to be removed: 79 Wood Ln

Applicant's Phone: Coby Friedman (415) 310-5442

On April 26, 2021 the Fairfax Tree Committee took the following action on the above referenced tree permit application:

FOR RECOMMENDATION ONLY TO PLANING COMMISSION:

Romaidis made a motion to recommend to the Planning Commission that the permit be approved with the exception that the oak tree in the corner remain and that no tree removal commence until a building permit associated with the driveway work is issued; the motion was seconded by Richardson-Mack and voted on.

Vote:
Benson- Aye
Childers- Aye
Richardson-Mack- Aye
Romaidis- Aye

Item #11 Vote: Ayes- 4, Noes- 0

__________APPROVED

REMINDER: PLEASE KEEP PERMIT NOTICE UP DURING THE 10 DAY WAITING PERIOD

__________ CONTINUED

__________ DENIED

CONDITIONS OF APPROVAL: **For Recommendation to Planning Commission only.**

THIS APPROVED APPLICATION IS YOUR PERMIT-KEEP IT ON THE JOB SITE. FAILURE TO HAVE THE PERMIT ON THE SITE WHILE THE TREE WORK IS IN PROGRESS MAY RESULT IN THE WORK BEING HALTED UNTIL YOU SHOW PROOF OF APPROVAL.
Please verify that the tree company performing the work has a current Fairfax Business license and worker's compensation coverage.

THIS TREE PERMIT EXPIRES IN SIX MONTHS. If necessary, you may apply for an extension in writing prior to the expiration date.

FOR RECOMMENDATION ONLY TO PLANNING COMMISSION

TOWN OF FAIRFAX

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

APR 08 2021

APPLICATION FOR TREE REMOVAL OR ALTERATION

A permit is required to remove or alter one or more trees on any parcel in the Town of Fairfax. All trees for which a permit is requested shall be tagged with an orange ribbon, a minimum of 10 days prior to the Tree Advisory Committee meeting date. Applicants must also post a notice of intent to alter or remove the marked Tree(s) in a prominent location visible along the frontage of the affected property.

APPLICANT INFORMATION

OWNER (APPLICATIONS MUST BE FILED BY PROPERTY OWNER): COBY FRIEDMAN	DATE OF APPLICATION: 4.6.2021
JOB ADDRESS/ASSESSOR'S PARCEL NO. IF SITE IS VACANT: 79 WOOD LANE	PHONE NUMBER: ~~415-721-4160~~ ~~CF CON. X. 101~~
EMAIL ADDRESS: coby@cfcontracting.com	FAX NUMBER: Do not (415) 310-5442
mail here → PROPERTY OWNER'S ADDRESS IF DIFFERENT FROM ABOVE: 96 FORREST AVE., FAIRFAX	ALTERNATE PHONE NUMBER:

TREE INFORMATION

SPECIES AND DESIGNATION OF HERITAGE/SPECIMEN/UNDESIRABLE TREE: (1) Apple	CIRCUMFERENCE BREAST HEIGHT: 47" CIRCUMFERENCE, 15" Ø REASON FOR REMOVAL/ALTERATION: NEW DEVELOPMENT DRIVEWAY
SPECIES AND DESIGNATION OF HERITAGE/SPECIMEN/UNDESIRABLE TREE: (5) OLIVE — LESS than 12" circumference	CIRCUMFERENCE BREAST HEIGHT: 3" CIRCUMFERENCE 1" Ø REASON FOR REMOVAL/ALTERATION: NEW DEVELOPMENT
SPECIES AND DESIGNATION OF HERITAGE/SPECIMEN/UNDESIRABLE TREE: (1) LIVE OAK	CIRCUMFERENCE BREAST HEIGHT: 31" CIRCUMFERENCE 10" Ø REASON FOR REMOVAL/ALTERATION: NEW DEVELOPMENT DRIVEWAY
SPECIES AND DESIGNATION OF HERITAGE/SPECIMEN/UNDESIRABLE TREE:	CIRCUMFERENCE BREAST HEIGHT: REASON FOR REMOVAL/ALTERATION

Please attached a site plan to this application showing the location and species of all trees with a diameter of 4 inches (circumference of 12 inches or more), measured 4.5feet above grade at tree base, property boundaries and easements, location of structures, foundation lines of neighboring structures and paved areas including driveways, .

AGENDA ITEM # 11

Any tree company used for the removal or alteration must have a current and valid Fairfax Business license. Please include the name, address, and phone number of the person or company doing the above listed work: T.B.D.

NAME:	PHONE NUMBER:
ADDRESS:	CONTRACTOR BUSINESS LICENSE NUMBER

Please note the Tree Advisory Committee may require applicants to submit their application to a Qualified Arborist for a report or recommendation at the expense of the applicant. A Qualified Arborist is defined as a Certified Arborist, A Certified Urban Forester, a Registered Consulting Arborist, or a Registered Professional Forester.

OWNER'S STATEMENT

I understand that in order to properly process and evaluate this application, it may be necessary for Town personnel to inspect the property, which is the subject of the application. I also understand that due to time constraints it may not always be possible for Town personnel to provide advanced notice of such inspections. Therefore, this application will be deemed to constitute my authorization to enter upon the property for the purpose of inspecting the same, provided that Town personnel shall not enter any building on the property except in my presence or the presence of any other rightful occupant of such building. I understand that my refusal to permit reasonable inspection of any portion of the property by town personnel may result in a denial of this application due to the lack of adequate information regarding the property.

Coby Friedman

Signature of Property Owner

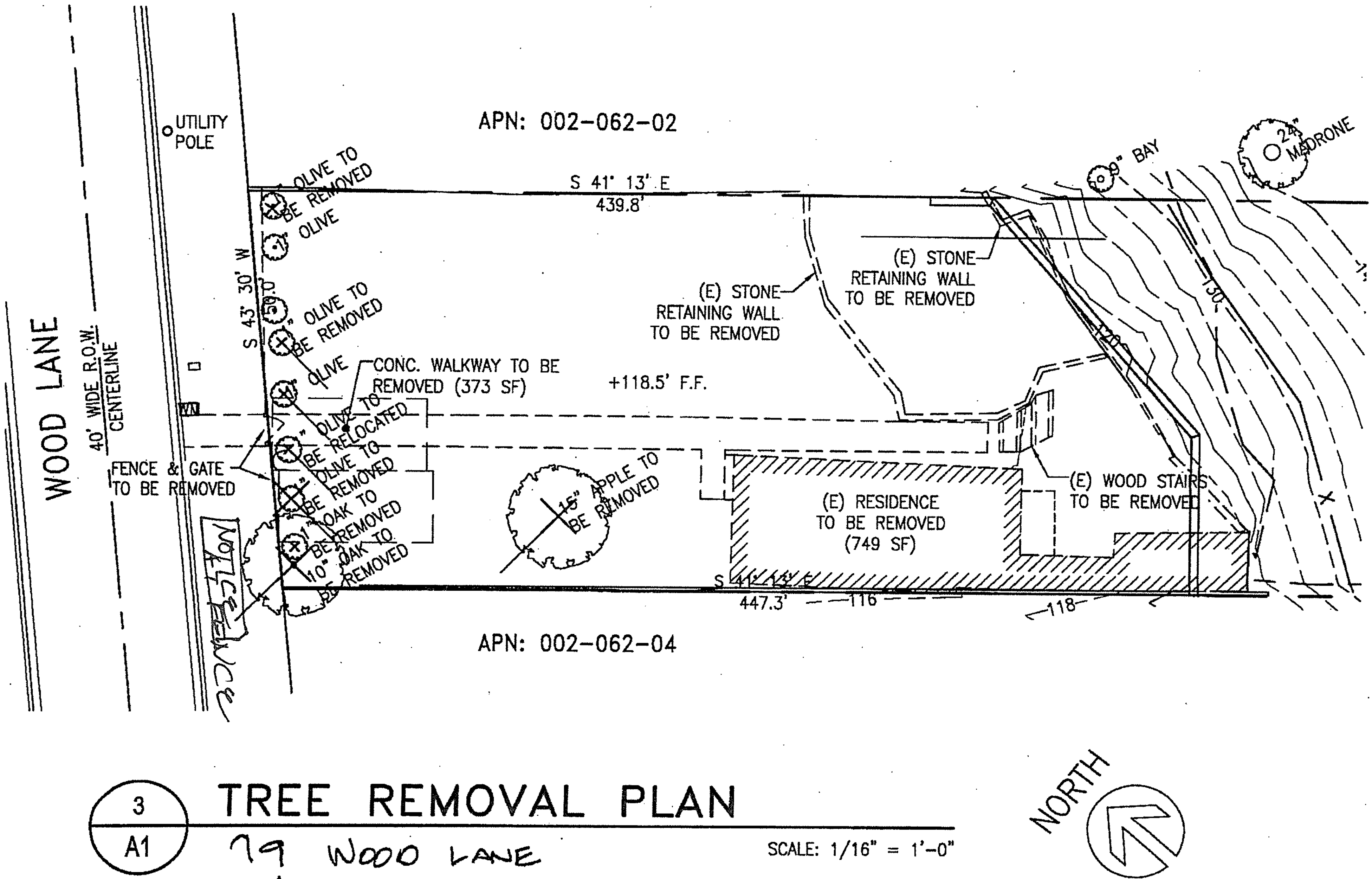
4.6.2021

Date

[AREA BELOW FOR STAFF USE ONLY]

Permit Number: 21-T-30	
Date Received: 4-6-21	Received by: S. Waters
Conditions of Approval:	
Tree Committee Action:	Date:

Tree Committee Actions can be appealed to the Town Council within 10 days of the Tree Committee Action. Contact Town Hall for more information.



3 / A1 **TREE REMOVAL PLAN**

SCALE: 1/16" = 1'-0"

79 WOOD LANE
04.06.2021