HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

September 5, 2007
Project Number 1866-03-07

Mr. Ted Pugh
P.O. Box 99485
Emeryville, California 94662-9485

RE: Geotechnical Review of Design Concept
Acacia Road Lot (APN 001-112-31)
Fairfax, California

Dear Mr. Pugh:

This presents the results of our geotechnical review of the proposed design and construction methodology for the planned residence at the referenced Acacia Road lot in Fairfax, California. Herzog Geotechnical previously performed a geotechnical investigation for the project and summarized results in our report dated July 25, 2007.

We reviewed the August 22, 2007 letter by BHW Engineers, LLC entitled *Acacia Road Lot & Proposed Residence*. Based on our review, we conclude that the proposed structural design and drainage methodology for the 32-foot wide retaining wall and foundation system outlined in this letter conforms to the intent of our geotechnical recommendations, and constitutes an appropriate means of mitigating continued slide hazard to the site and roadway.

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.

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
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HERZOG

GEOTECHNICAL
CONSULTING ENGINEERS

July 25, 2007
Project Number 1866-03-07

Mr. Ted Pugh
P.O. Box 99485
Emeryville, California 94662

RE: Report
Geotechnical Investigation
Acacia Road Lot (APN 001-112-31)
Fairfax, California

Dear Mr. Pugh:

This presents the results of our geotechnical investigation for the proposed residence at the referenced Acacia Road lot in Fairfax, California. The scope of our investigation was to review selected geologic references, review our previous work at the site, observe exposed site conditions, drill three additional test borings, perform laboratory testing and engineering analyses, and develop geotechnical conclusions and recommendations for the project. Our scope of work was outlined in our professional services agreement dated January 30, 2007.

PROJECT DESCRIPTION

The project will consist of constructing a three-story, wood-framed residence over a garage on the existing vacant parcel. Retained cuts for the project will range to about 30 feet in total height. The project is shown on the preliminary plans by Jeff Kroot, Architect dated May 2007.

WORK PERFORMED

Prior to performing our investigation, we reviewed our previous work in connection with a landslide within the lower portion of the site, along with selected geologic references. In accordance with our recommendations, a retaining wall (Phase 1) was constructed along the downslope edge of the property at 13 Acacia Road in March 2007. We explored the subsurface conditions in the project area on June 12, 2007 to the extent of three additional test borings extending between approximately 9-1/4 and 12 feet deep, and extending into bedrock. Due to limited access, the test borings were drilled with portable drilling equipment. The locations of our previous and recent test borings are shown on the attached *Site Plan*, Plate 1.

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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 samplers were driven 18 inches, and the number of blows was recorded for each 6 inches of 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 our previous and recent test borings are presented on Plates 2 through 6. The soils encountered are described in accordance with the criteria presented on Plate 7. Bedrock is described in accordance with the *Engineering Geology Rock Terms* presented on Plate 8. 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 7.

FINDINGS

Site Conditions

The site is situated on the southwestern (upslope) side of Acacia Road in Fairfax, California. The portion of the roadway below the site was created by excavating into the hillside. The resultant cut bank for the road ranges to approximately 15 feet high, and is inclined at between approximately 1/2:1 and 1:1 (horizontal:vertical). The cut bank generally exposes gravelly clay colluvium which washed or slid down from upslope areas. The lower 4 to 5 feet of the bank is blanketed with rock rip-rap which was reportedly installed by the Town of Fairfax immediately following landsliding during the intense winter of 2005/2006. The portion of the slide extending onto the property is approximately 45 feet long, and consists of slumping of the approximately 15-foot high roadway bank (see Plate 1). Debris from the failure reportedly extended into the roadway and was removed.

The portion of the lot above the bank is a tree-covered hillside. The property is bounded on the southeast and northwest sides by developed residences. Upslope and southwest of the lot the ground surface extends up to residential properties on Bay Road. During our investigation we noted the presence of some fill material within the subject property which appears to be associated with previous grading of level benches on the adjacent property to the southeast.

Subsurface Conditions

The site is within the Coast Range Geomorphic Province which includes San Francisco Bay and the northwest-trending mountains that parallel the coast of California. These features were formed by tectonic forces resulting in extensive folding and faulting of the area. Bedrock has been mapped previously within the site (Rice, 1976) as consisting of Jurassic to Cretaceous aged sandstone of the Franciscan Assemblage.

Our test borings encountered fill, topsoil, colluvium (slopewash) and residual soils overlying bedrock. The fill encountered consisted of loose to medium dense silty sand derived from previous grading activities near the southeast edge of the property. The topsoil encountered consisted of loose silty sand and soft sandy silt which contained varying amounts of organics. The colluvial soils encountered consisted of medium dense clayey gravel and soft sandy and gravelly clay. The residual soils encountered consisted of medium dense clayey gravel derived from the in-place weathering of the underlying bedrock. The fill and native soils encountered are relatively weak and compressible, and are subject to gradual downslope creep on hillsides. In addition, portions of the soils encountered are expansive. Expansive soils undergo changes in volume with changes in moisture content, and can cause slabs and lightly loaded foundations to heave and crack. Bedrock encountered in the borings generally consisted of firm to moderately hard sandstone and shale.

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

	Depth (feet)			
Boring	**Fill**	**Topsoil/Colluvium**	**Residual Soil**	**Bedrock**
B-1A	---	0-6.0	---	6.0-9.2±
B-2A	0-3.0	3.0-6.0	---	6.0-10.5±
B-3A	---	0-7.5	---	7.5-12.4±
B-2	---	0-4.0	4.0-5.2	5.2-7.0±
B-3	---	0-1.2	1.2-3.5	3.5-4.5±

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

Groundwater

Free groundwater did not develop in the borings prior to backfilling. Groundwater levels at the site are expected to fluctuate over time due to variations in rainfall and other factors. Rainwater percolates through the relatively porous surface soils. On hillsides, the water typically migrates downslope in the form of seepage within the porous soils, at the interface of the soil/bedrock contact, and within the upper portions of the weathered and fractured bedrock.

CONCLUSIONS

Based on the results of our investigation, we conclude that the project is feasible from a geotechnical standpoint provided the recommendations presented in this report are incorporated into the project. The primary geotechnical concerns are discussed below.

Foundation and Slab Support

Our investigation indicates that most of the planned excavations should expose weathered bedrock which will be suitable for support of footings and slabs. In areas where excavations do not expose bedrock, it will be necessary to utilize either deepened footings or drilled piers to extend support into the underlying bedrock. We estimate that post-construction differential settlements of foundations designed in accordance with the recommendations contained in this report will be on the order of half an inch.

In areas where slab subgrade exposes soils, it will be necessary to overexcavate these soils, segregate and remove expansive materials, and replace the removed material non-expansive compacted fill which is founded on bedrock. Alternatively, slabs may be designed to structurally span between bedrock supported foundations.

Excavation and Shoring

We anticipate that planned cuts will expose relatively weak soil and bedrock with bedding, fracture and shear surfaces which may slope adversely into the planned excavations. Excavations must therefore be shored to laterally support the banks and to maintain stability of adjacent areas. Among possible shoring alternatives are cantilevered or tiedback soldier piers with lagging, tiedback shotcrete walls, or internally braced walls. Shoring should be designed by the Contractor's engineer to resist lateral earth pressures and surcharge loads from adjacent structures using the design criteria presented in this report. Adequate drainage facilities should be provided to prevent hydrostatic buildup behind the shoring. Unless non-yielding support is provided for shoring (i.e. tiebacks or rigid bracing), it will be necessary to underpin adjacent house foundations.

During construction, cuts should be closely monitored for the presence of adverse bedding, fracturing conditions, or lithologic contacts that could promote slope instability. As excavation proceeds, conditions may be exposed which require design modifications.

Our investigation indicates that excavations will expose areas of hard bedrock which will necessitate the use of heavy-duty, hydraulically-driven excavation equipment. Resistant blocks of hard rock may require hoe-ramming. Hard drilling or coring will be required to achieve the required penetrations for drilled piers, including soldier piers.

Geologic and Seismic Hazards

Landsliding

Regional mapping by Rice (1976) indicates that the site lies at the southeastern margin of an earthflow landslide which encompasses most of the hillside to the northwest. A map by Davenport (1984) of slope failures resulting from the severe 1982 storms does not indicate that sliding was reported at the site at that time. The southeastern portion of the site lies within Slope Stability Zone 2, and the northwestern portion of the site lies within Zone 4 as defined in "*Geology for Planning: Central and Southeast Marin County*" (Rice, 1976). Zone 2 includes narrow ridge and spur crests that are underlain by relatively competent bedrock, but which are flanked by steep, potentially unstable slopes. Zone 4 includes areas of existing active or inactive landslides, and areas subject to downslope creep. The zones range from 1 to 4, with Zone 4 being least stable.

As discussed previously, the roadway cut bank at the base of the site experienced sliding during the heavy winter of 2005/2006. We judge that this slide will be mitigated by excavation and retaining wall construction for the proposed residence. In order to buttress potentially unstable upslope areas, it would be desirable to extend retaining walls for the new structure laterally to encompass as much of the width of the lot as possible. In order to address the potential impact of sloughing and instability from upslope of the project, the upslope foundation of the residence should be extended at least 3 feet above finished grade to provide catchment for slough debris.

Fault Rupture/Ground Shaking

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.

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 System	Distance From Site (Miles/Km)	Direction From Site to Fault	MCE Moment Magnitude	Peak Ground Acceleration (g's)
San Andreas	6.9 / 11.1	Southwest	7.9	0.44
San Gregorio	8.1 / 13.0	South	7.3	0.33
Hayward	10.9 / 17.5	Northeast	7.1	0.24
Rodgers Creek	15.0 / 24.1	Northeast	7.0	0.17

Deterministic information generated for the site considering the proximity of active faults and estimated bedrock accelerations are presented in the table above. The estimated ground accelerations were derived from mean attenuation relationship presented by Abrahamson and Silva (1997; Rock Site) and are based on the published estimated Maximum Credible Earthquake moment magnitudes (MCE) for each fault (Petersen, 1996), the shortest distance between the site and the respective fault, the type of faulting, and the estimated shear wave velocities of the on-site soils. The MCE, also referred to as the Upper Bounds Earthquake, is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework. The deterministic evaluation of the potential for ground shaking assumes that a maximum magnitude earthquake produces fault rupture at the closest proximity to the site. This evaluation does not take recurrence intervals or other probabilistic effects into consideration.

Data presented by the Working Group on California Earthquake Probabilities (USGS, 2003) 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 62 percent. Consequently, we judge that the site will likely be subject to strong earthquake shaking during the life of the improvements.

Liquefaction

During severe 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 (Seed and Idriss, 1982). 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

During severe ground shaking from earthquakes, 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 high silt and clay content of the soils encountered in the test borings.

RECOMMENDATIONS

Seismic Design

Based on the results of our investigation, the following seismic design criteria were developed in accordance with the *Uniform Building Code* (1997):

Seismic Zone Factor (Z)	0.4
Seismic Source Type	"A"
Soil Profile Type	S_C
Near Source Factor N_a	1.00
Near Source Factor N_v	1.16
Seismic Coefficient C_a	0.40
Seismic Coefficient C_v	0.66

Shoring and Underpinning

Where excavations will extend below a 1-1/2:1 line projected down from the ground surface adjacent to existing foundations, the foundations should be underpinned unless non-yielding (i.e. tiedback or rigidly braced) shoring is provided. Underpinning piers should consist of drilled, cast-in-place, reinforced concrete piers, or of shored, hand-excavated pit footings designed in accordance with the criteria presented in the *Foundation Support* section of this report. The underpinning should be designed to resist lateral loads acting above a 1-1/2:1 line projected up from the base of excavations as outlined in the *Retaining Walls* section of this report, and downslope creep forces as outlined in the *Foundation Support* section of this report.

The Contractor should install shoring as excavation proceeds in order to maintain lateral support. All underpinning, temporary slopes, and shoring should be contractually established as solely the responsibility of the Contractor, and these items is specifically excluded from our scope of work. In addition to lateral earth pressures, shoring should be designed to resist surcharge loading from structures and retaining walls as outlined in the *Retaining Walls* section of this report. Design criteria for alternative types of shoring are presented below.

Soldier Pier Wall

Support for excavations may be provided using cantilevered, tiedback or braced soldier pier and lagging walls. Cantilevered soldier piers and lagging should be designed to resist an active lateral earth pressure equivalent to a fluid weighing 40 pounds per cubic foot (pcf) for level backfill, and 60 pcf for backfill at a 2:1 slope. For intermediate slopes, interpolate between these values. If tiebacks or bracing are used with the soldier piers, the design pressures presented in the *Tiedback Wall* section of this report should instead be used.

Soldier piers should consist of drilled, cast-in-place, reinforced concrete piers which may be provided with steel wide flange beams. Drilled piers should be at least 18 inches in diameter and should extend at least 8 feet into competent bedrock. Actual design pier depths and diameters should be calculated by the design 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 portion of the piers extending into competent bedrock below the level of planned excavations can impose a passive equivalent fluid pressure of 450 pcf acting over two pier diameters, and vertical dead plus real live loads of 1000 pounds per square foot (psf) in skin friction. End bearing should be neglected due to the uncertainty of mobilizing end bearing and skin friction simultaneously.

If groundwater is encountered, it may be necessary to dewater the holes and/or to place concrete by the tremie method. Hard drilling or coring will be required to achieve the required penetrations.

Lagging should be installed promptly as the excavation progresses. Voids behind the lagging should be tightly backfilled with free-draining crushed rock or gravel (drain rock) to prevent yielding behind the wall. Vertical spacers should be provided between the lagging to allow seepage through the face of the wall. If the wall is to act as a permanent structure, at least 1 foot of drain rock or Caltrans Class 2 Permeable Material should be placed between the lagging and the cut face. If crushed rock or gravel is used, a filter fabric such as Mirafi 140N or equivalent should be provided between the drain rock and the cut face. If Class 2 Permeable Material is used, the filter fabric may be omitted. The upper 1 foot of the wall backfill should be compacted clayey soil to exclude surface water.

Tiebacks may be used in conjunction with the soldier piers to generate additional lateral resistance. Tiedback walls should be designed using the criteria presented below. The downward thrust from the tiebacks should be included when calculating the vertical load on the soldier piers.

Tiedback Wall

Tiebacks should be inclined downward at an angle of at least 15 degrees from the horizontal. Tiebacks should have minimum unbonded lengths of 10 and 15 feet for bars and strands, respectively. Tiebacks should have minimum bonded lengths of 12 feet. Tieback support should only be derived in competent bedrock located at least 5 feet beyond an imaginary 45 degree line extended upwards from the base of the wall. The allowable skin friction of 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 pounds per square foot (psf). The contractor should be responsible for determining the actual length of tiebacks necessary to resist design loads based on their familiarity with the installation method utilized. Our field engineer should be present to observe conditions during drilling. The wall should be provided with a base footing excavated into rock at least 12 inches below finished downslope grade.

The tiebacks and facing should be designed to resist a uniform outwards lateral pressure of 20xH psf for level backfill, and 30xH for backfill at a 2:1 slope (where H is the height of the wall in feet). For intermediate slopes, interpolate between these values. Shoring should be designed for additional surcharge loading from structures as outlined in the *Retaining Walls* section of this report.

The walls should be backdrained with a drainage media such as Miradrain 6000, or equivalent. The drainage media should extend from 1 foot below the top of the walls to the bottom of the walls. Water from the backdrains should be outletted utilizing rigid perforated PVC or ABS pipe (Schedule 40, SDR of 35 or better) at the base of the walls, or weep holes spaced 4-foot on-center through the base of the wall. Waterproofing of the walls should be as specified by the wall designer.

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 should be double corrosion protected. The bars 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 may be initially grouted over their entire length. When the grout has attained the required compressive strength, 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. After testing, the tension in the anchor should be reduced to the design load and locked off. Replacement tiebacks should be installed for tiebacks that fail the load testing.

Braced Shoring

Braced shoring should be designed using a lateral pressure of 20xH psf for level backfill, and 30xH for backfill at a 2:1 slope (where H is the retained height). For intermediate slopes, interpolate between these values. Thrust blocks for the shoring should extend into competent bedrock, and should be designed using passive equivalent fluid pressures of 450 pcf. The upper 12 inches of material should be neglected when calculating passive pressures. Provisions should be made to allow free drainage through the shoring as outlined previously.

Foundation Support

Spread Footings

Spread footings should be at least 18 inches wide, should be bottomed at least 12 inches into competent bedrock, and should extend at least 12, 18 and 24 inches below lowest adjacent finished grade for 1, 2 and 3 story structures, respectively. Footings should be stepped as necessary to produce level tops and bottoms, and should be deepened as necessary to provide at least 5 feet of horizontal clearance in rock between the portion of footings designed to impose passive pressures and the face of the nearest slope or wall. Spread footings extending into competent bedrock can be designed to impose dead plus code live load bearing pressures and total design load bearing pressures of 4000 and 5300 psf, respectively.

Resistance to lateral pressures can be obtained in rock from passive pressures against the sides of footings and from friction along the base of footings. We recommend the following criteria for design:

Passive Pressures*	=	450 pcf equivalent fluid pressure
Friction Factor	=	0.40 times net vertical dead load

* Neglect passive pressure in the top 12 inches where the surface is not confined by slabs.

Drilled Piers

Drilled piers should be at least 18 inches in diameter and should extend at least 8 feet into competent bedrock located below a 1-1/2:1 line projected up from the base of banks or retaining walls. Required pier depths and diameters should be calculated by the Project Structural Engineer using the criteria presented below. For planning purposes, the depth to bedrock may be estimated based on the boring logs.. The materials encountered in pier excavations should be evaluated by our representative in the field during drilling. Drill spoils should be removed from the site or properly compacted and retained.

Piers should be interconnected with grade beams to support structural loads and to redistribute stresses imposed by the creeping soils. Piers and grade beams should be designed and reinforced to resist creep forces acting from the ground surface to bedrock located below a 1-1/2:1 line projected up from the base of unretained banks, 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 bedrock below a 1-1/2:1 line projected up from the base of banks can impose a passive equivalent fluid pressure of 450 pounds per cubic foot (pcf) acting over 2 pier diameters, and vertical dead plus real live loads of 1000 pounds per square foot (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.

In areas where expansive soils are encountered, a compressible void form product (Econo-Void or equivalent) should be provided beneath grade beams for protection against expansive soil uplift. Expansive soils exert uplift forces on concrete overpours. Grade beams should be formed above the trench to prevent overpours, and care should be taken to prevent overpours (mushrooming) at the tops of piers.

If groundwater is encountered, it may be necessary to dewater the holes and/or to place concrete by the tremie method. Caving soils may be encountered, in which case it may be necessary to case the holes. Hard drilling or coring will be required to achieve required penetrations.

Retaining Walls

Retaining walls should be supported in rock on drilled pier or spread footing foundations designed in accordance with the recommendations presented in this report. A minimum factor of safety against instability of 1.5 should be used to evaluate static stability of retaining walls.

Retaining walls should be supported on foundations designed in accordance with the recommendations presented in this report. Free-standing retaining walls should be designed to resist active lateral earth pressures equivalent to those exerted by a fluid weighing 40 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. The upslope wall of the residence should be provided with at least 3 feet of slough catchment height. The wall design should include an equivalent fluid pressure on the catchment area of 125 pcf.

The seismic stability of walls should be evaluated based on an additional uniform lateral earth pressure of 20xH psf (where H is the height of the wall in feet). 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 and foundation loads. 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. Where an imaginary 1-1/2:1 plane projected downward from the outermost edge of a surcharge load or footing intersects a retaining wall, we should be contacted to provide appropriate lateral surcharge criteria.

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 outlet. Accessible subdrain cleanouts should be provided, and should be maintained on a routine basis. 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 at least 1 cubic foot of permeable material or fabric-wrapped crushed rock per lineal foot of wall. 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 Project 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.

Slab Support

In areas where slab subgrade excavations do not expose bedrock, slabs should be structurally supported, or else underlain by compacted fill which is founded on bedrock and which is retained on the downslope sides. For non-structural slabs, existing soils beneath planned slabs-on-grade should overexcavated as necessary to create level benches in bedrock. Topsoil and expansive soils should be segregated and disposed of off-site. Approved non-expansive fill should then be placed in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned to within 3 percent of optimum moisture content, and compacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of a material expressed as a percentage of the maximum dry density of the 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.

We anticipate that on-site soils other than topsoils and expansive soils will generally be suitable for reuse as engineered fill. However, wetting or drying 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 Herzog Geotechnical prior to use. Imported fill should have a plasticity index of 15 or less, a liquid limit of 40 or less, and should be free of organic matter and of rocks larger than 4 inches. Herzog Geotechnical should observe and approve fill material prior to importing. Slab-on-grade subgrade should be rolled to provide a firm, unyielding surface.

Slab subgrade should be sloped to drain into a 12 inch deep trench excavated in the downslope direction 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.

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 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.

If expansive subgrade soils are encountered, structural slabs should be underlain by an approved void forming product for protection from expansive soil heave. The void forms should consist of at least a 2-inch thick degradable and compressible paper product (SureVoid®, or equivalent). The capillary moisture break should be installed beneath the void form, and the moisture barrier should be carefully installed over the top of the void form.

Slabs-on-grade should be at least 5 inches thick, and should be reinforced at least with #4 reinforcing bars spaced at 12 inches on-center each way to control cracking due to differential settlement. Slabs should be structurally separated from foundation supported elements to accommodate differential movement, and control joints should be provided as determined by the Structural Engineer. Reinforcement should be continuous across joints. Slabs should be as designed by the project structural engineer.

Geotechnical Drainage

Positive drainage should be provided away from foundations, slopes and retaining walls. Ponding of surface water should not be allowed. Runoff should be intercepted along the upslope side of improvements with concrete lined ditches, and fail-safe drainage should be provided to prevent flooding of the project if drains become clogged. Roofs should be provided with gutters and downspouts. Downspouts and surface drains for the project should be connected into closed conduits which discharge at an approved outlet at the street. 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 retaining wall backdrains and slab underdrains.

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 intermittent observation and appropriate field and laboratory testing during pier drilling, footing excavation, tieback drilling and testing, slab subgrade overexcavation and backfill compaction, wall backfilling, void form installation, and subdrainage installation to evaluate if subsurface conditions are as anticipated and to check for conformance with our recommendations. We should also be notified to observe the completed project. Steel, concrete, slab moisture barriers, shoring, underpinning and/or waterproofing should be inspected by the appropriate party, and are not part of our scope of 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 Mr. Ted Pugh and his 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 dates 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 locations of the test borings were established in the field by reference to existing features, and should be considered approximate only.

Our investigation 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 mold hazard at the site.

There is an inherent risk of instability associated with all hillside construction. For houses constructed on hillsides, we recommend that the owner obtains the appropriate landslide and earthquake insurance.

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
Exp. 9/30/09
GEOTECHNICAL
STATE OF CALIFORNIA

Craig Herzog, G.E.
Principal Engineer

Attachments: References
Plates 1 - 8

REFERENCES

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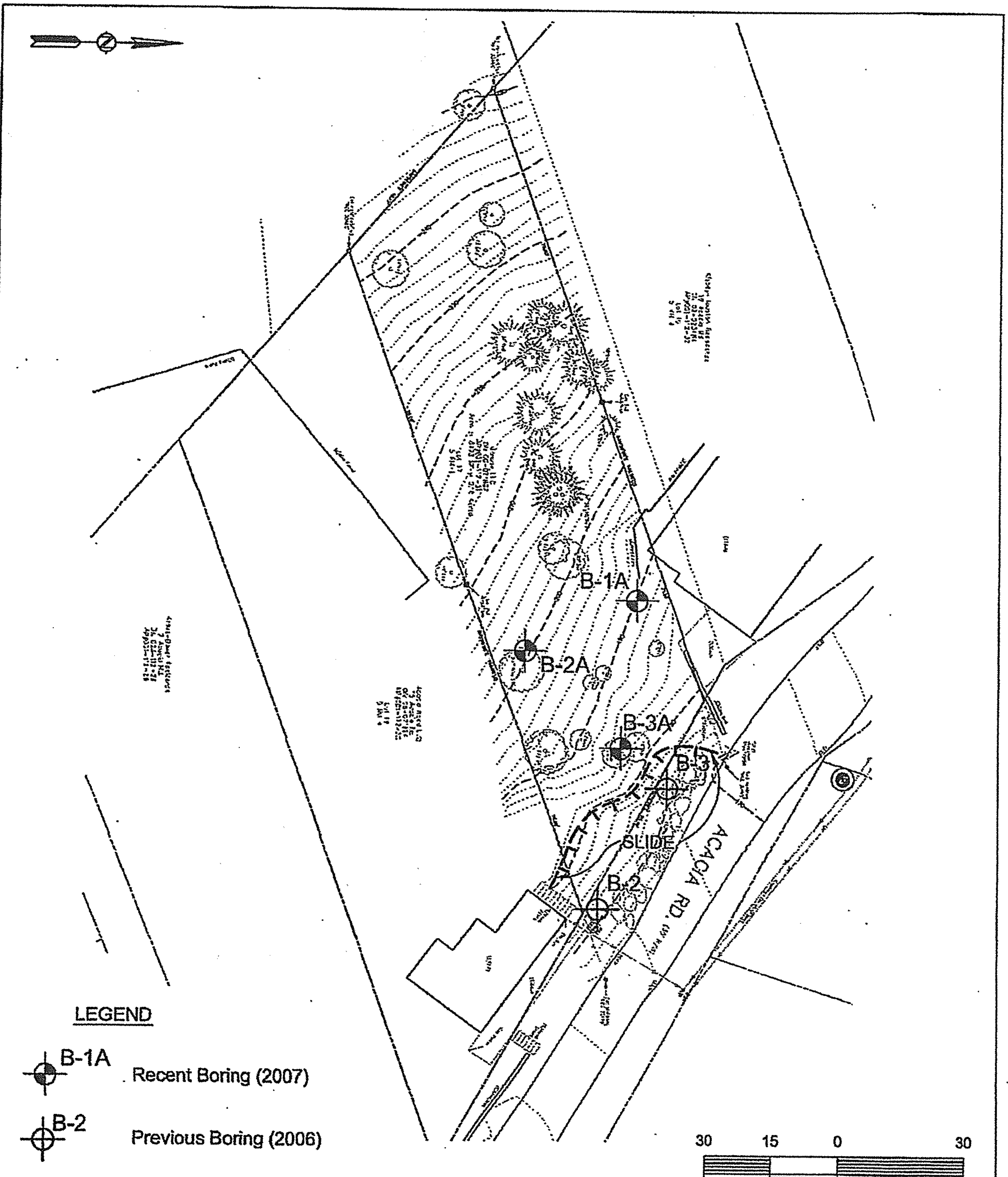


Reference: Topographic Survey by J.L. Engineering, dated April 2007.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS	Job. No: 1866-01-07 Appr: Drwn: LPDD Date: JUN 2007	SITE PLAN 15 Acacia Road Fairfax, California	PLATE 1

EQUIPMENT: 4" Flight Auger

LOGGED BY: C.H.

ELEVATION: * *

START DATE: 6-12-07

FINISH DATE: 6-12-07

Other Laboratory Tests	TorVane (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	Description
							0	BROWN SANDY SILT (ML), soft, dry, with roots
							1	
							2	
		13.4	95		9			MOTTLED ORANGE-GRAY-BROWN GRAVELLY CLAY (CL), medium stiff, dry to moist
							3	
							4	
							5	
		14.2	83		16			
							6	ORANGE-GRAY SANDSTONE, moderately hard, weak, highly weathered
							7	
					65/8"		8	drilling refusal at 8 feet
								becomes hard, strong below 8-1/2 feet, fractures oriented N40°W, 80°NE to vertical
							9	

BOTTOM OF BORING 1A @ 9.2 FEET
No Free Water Encountered

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

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 1866-03-07

Appr:

Drwn: LPDD

Date: JUN 2007

LOG OF BORING 1A

15 Acacia Road

Fairfax, California

PLATE 2

EQUIPMENT: **4" Flight Auger**
LOGGED BY: **C.H.**
ELEVATION: **
START DATE: **6-12-07**
FINISH DATE: **6-12-07**

Other Laboratory Tests	TorVane (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	Description
							0	ORANGE-BROWN SILTY SAND (SM), loose to medium dense, dry, with roots (Fill)
		9.9	103		11			
							3	ORANGE-BROWN CLAYEY GRAVEL (GC), medium dense, moist
		13.4	106		20		6	ORANGE-BROWN SANDSTONE WITH INTERBEDDED SHALE, moderately hard, weak, highly weathered, fractures oriented N-S, 45°E
					52		9	drilling refusal at 9 feet sub-horizontal shear fabric at 9 feet
							10	GRAY SANDSTONE, hard, strong, moderately weathered

BOTTOM OF BORING 2A @ 10.5 FEET
No Free Water Encountered

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

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 1866-03-07
Appr:
Drwn: LPDD
Date: JUN 2007

LOG OF BORING 2A

15 Acacia Road
Fairfax, California

PLATE 3

Other Laboratory Tests	TorVane (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: 4" Flight Auger · LOGGED BY: C.H. · ELEVATION: ** · START DATE: 6-12-07 · FINISH DATE: 6-12-07
							0	BROWN SILTY SAND (SM), loose, dry, with roots and occasional gravel
							4	ORANGE-BROWN CLAYEY GRAVEL (GC), medium dense, moist
							7.7	ORANGE-GRAY SHALE WITH INTERBEDDED SANDSTONE, firm, weak, highly weathered, sub-horizontal bedding
					27		9	
					59/ 10"		11	
							11.5	becomes moderately hard below 11-1/2 feet

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

BOTTOM OF BORING 3A @ 12.4 FEET
No Free Water Encountered

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

Job No: 1866-03-07
Appr:
Drwn: LPDD
Date: JUN 2007

LOG OF BORING 3A

15 Acacia Road

Fairfax, California

PLATE 4

EQUIPMENT: 4" Flight Auger
LOGGED BY: G.M.
ELEVATION: **
START DATE: 6-6-06
FINISH DATE: 6-6-06

Other Laboratory Tests	TorVane (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	DEPTH (FEET)	Description
						0	ORANGE-BROWN SANDY CLAY (CL), soft, moist
		20.6	102		5		
						1.8	YELLOW-ORANGE-BROWN GRAVELLY CLAY (CH), soft, moist
						3.9	ORANGE-BROWN CLAYEY GRAVEL (GC), medium dense, moist (Residual Soil)
					15	5	
						5.2	ORANGE-BROWN SANDSTONE, firm, friable, highly weathered
					27		
		19.1	105			6	
						7	BOTTOM OF BORING 2 @ 7 FEET No Free Water Encountered

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

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 1866-01-06
Appr:
Drwn: LPDD
Date: JUN 2007

LOG OF BORING 2

15 Acacia Road

Fairfax, California

PLATE 5

EQUIPMENT: 4" Flight Auger
LOGGED BY: G.M.
ELEVATION: **
START DATE: 6-6-06
FINISH DATE: 6-6-06

Other Laboratory Tests	TorVane (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	DEPTH (FEET)	Description
						0	ORANGE-BROWN SANDY CLAY (CH), medium stiff; moist
						1	ORANGE-BROWN CLAYEY GRAVEL (GC), medium dense, moist (Residual Soil)
		16.4	108		13	2	
					23	3	ORANGE-GRAY-BROWN SANDSTONE, firm, friable to weak, highly weathered
						4	

BOTTOM OF BORING 3 @ 4.5 FEET
No Free Water Encountered

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

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

Job No: 1866-01-06
Appr:
Drwn: LPDD
Date: JUN 2007

LOG OF BORING 3

15 Acacia Road
Fairfax, California

PLATE
6

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

HERZOG GEOTECHNICAL CONSULTING ENGINEERS	Job No: 1866-03-07 Appr: Drwn: LPDD Date: JUN 2007	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA 15 Acacia Road Fairfax, California	PLATE 7

ROCK SYMBOLS

SHALE OR CLAYSTONE

SILTSTONE

SANDSTONE

CONGLOMERATE

CHERT

PYROCLASTIC

VOLCANIC

PLUTONIC

SERPENTINITE

METAMORPHIC ROCKS

DIATOMITE

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

ILS ASSOCIATES, INC.
CIVIL ENGINEERING AND LAND SURVEYING

February 22, 2016

Town of Fairfax
142 Bolinas Road
Fairfax, CA 94930
Attention: Linda Neal, Senior Planner

RE: 15 Acacia Road, Fairfax
Our File No. 8207

Dear Linda,

This letter is to advise you that the Hydrology and Hydraulics Study dated February 25, 2008 prepared by this office is suitable for current project review by the Town of Fairfax.

Sincerely,

Irving L. Schwartz, C.E.

Cc/Ted Pugh

79 GALLI DRIVE, SUITE A NOVATO, CA 94949-5717 (415) 883-9200 FAX (415) 883-2763

ATTACHMENT E

ILS ASSOCIATES, INC.
CIVIL ENGINEERING AND LAND SURVEYING

March 27, 2008

Ted Pugh
P.O. Box 99485
Emeryville, Ca 94662-9485

Re: 15 Acacia Road, Fairfax
Our File No. 8207

Dear Mr. Pugh,

You have requested our input in regards to your proposed home's lot drainage system and the lower patio area below the retaining wall. BHW Engineers is responsible for actual wall and drainage design, as shown on J.L. Engineering's sheet C1. We have reviewed sheet C1 in line with our hydrology report and have the following information for you.

The lot's steep topography and need for adequate drainage provisions drives the design. The proposed features are:

- Two separate drainage system components, one for the house's foundation, which is also a retaining wall, and a second system further upslope to capture runoff behind the house.
- Inteceptor trenches are located behind the site walls upslope of the house.
- The drainage collection pipes are routed to flow into a cistern that is located in the home's lowest level below grade.
- A second cistern, if needed, can be placed under the lower patio.

We recommend a stormwater collection cistern to accommodate the difference between the pre and post-construction 100-year storm flow be installed under the lower patio, as this patio provides a level area for the cistern. We agree that combining this cistern with the graywater/roof rainwater cistern is feasible. However, the detailed designs are still evolving. For one thing, we believe that the combined systems may have maintenance issues, therefore we recommend you keep the patio location available for a separate cistern of at least a 500-gallon capacity.

Sincerely,

Irving L. Schwartz, C.E.

79 GALLI DRIVE, SUITE A NOVATO, CA 94949-5717 (415) 883-9200 FAX (415) 883-2763

ILS ASSOCIATES, INC.
CIVIL ENGINEERING AND LAND SURVEYING

Hydrology & Hydraulics Study

for

15 Acacia Road
Fairfax, CA

Prepared for:
Ted Pugh

Prepared by:
ILS Associates, Inc.
79 Galli Drive – Suite A
Novato, CA 94949

REGISTERED PROFESSIONAL ENGINEER
IRVING LOUIS SCHWARTZ
18221
CIVIL
STATE OF CALIFORNIA

Prepared on:
February 25, 2008

Job No. 8207

DRAINAGE NARRATIVE

The site is located at 15 Acacia Road. The property is narrow and long. The terrain is steep and wooded and faces northeasterly. Acacia Road is paved and slopes toward the site. The concentrated roadway runoff flows along the edge of the roadway in an old concrete gutter. This gutter has been mostly filled in by asphalt due to overlaying the roadway asphalt pavement. Approximately 120' southeasterly from the subject site is an existing storm drain system in the street which consists of grated drop inlets, a concrete headwall inlet structure and 12"+/- diameter outlet pipes, which presently collects the drainage from the subject site.

Our Hydrology analysis is based on; Cal-Trans Rainfall Intensity-Duration-Frequency Analysis, County of Marin Hydrology Manual, Marin County Rational Method Computation form (Revised August 2, 2000), the memorandum by Ray Wrysinski dated February 15, 2008 and our follow up conversations with Mr. Wrysinski on February 22 and 25th 2008.

For the post development condition we have calculated a weighted value for coefficient of runoff. We take a conservative approach in our analysis by assuming hardscape areas are impervious even though pervious type materials are to be used.

We modify the Marin County Rational Method computation, by not adding five minutes to the time of concentration. This change is based on conversations with Mr. Wrysinski and on review of other methods such as the Kipich formula. The resulting time of concentration for both pre and post development runoff is less than five minutes. We use Caltrans rainfall intensity curve chart for determining rainfall intensity (I), which has a minimum time of concentration of five minutes. The Hydrology calculations show a small increase in post development runoff; from 1.25 cubic feet per second (c.f.s.) to 1.32 c.f.s., for a 100 year storm event. We calculate that there would be an additional 430 gallons of runoff from the site during a 100-year storm event. This increased runoff can be mitigated with an onsite detention facility. We recommend that the capacity of this detention facility be oversized to handle the increase in runoff plus any accumulation of silt. If the detention facility is incorporated into the Improvement Plans, then the development of this site as proposed will not increase downstream flows.

ILS ASSOCIATES, INC.
CIVIL ENGINEERING AND LAND SURVEYING

BY: AJS JOB NO. 8207

DATE: 2/22/08 SHEET NO: 1

RATIONAL METHOD COMPUTATION FORM

(From Cal-Trans Rainfall Intensity-Duration-Frequency Analysis & County of Marin Hydrology Manual Revised 8/2/00)

EXISTING PRE-DEVELOPME HYDROLOGY 15 ACACIA RD. FAIRFAX

$$Q = C \times I \times A$$

Watershed TRIBUTARY 'A' At Point P.O.C. 1

Area = 18,548 sq. ft. = 0.42 acres.

Time of Concentration (+C)

$$t_c = \frac{1.8\,(1.1-C)\sqrt{L}}{[S(100)]^{1/3}} \cancel{+5\text{ Min.}} = \frac{1.8(1.1-0.7)\sqrt{330}}{[0.47(100)]^{1/3}} = 3.6 \text{ min.}$$

TRY KIRPICH METHO

$t_c = 0.0078(L)^{0.77}(S)^{-0.}$

$t_c = 0.9$ min.

C = Runoff Coefficient* = 0.70

L = Longest run in feet = 330'

S = Average Slope in ft/ft = $\frac{\Delta H}{L} = \frac{154}{330} = 0.47$

Intensity

P_{60} (chart I) = 1.5 zone (chart V) = C subzone (chart v) 2

C2
.7 | .67

I_{100} (chart k) = 4.25 Rd_{10} (chart k)

$I_{10} = I_{100}$ 4.25 x Rd_{10} (chart k) 0.7 = 2.98 in/hr.

I ____ = Rd ____ (from Chart R) x I_{100} ____ = ____ in/hr.

Coefficient of Runoff

Relief = 0.40

Soil infiltration = 0.10

Vegetal cover = 0.05

Surface storage = 0.15

C = 0.70 (0.70 minimum)*

Peak Discharge Q = C x I x A

Q 10 = 0.70 x 2.98 x 0.42 = 0.88 c.f.s.

Q 100 = 0.70 x 4.25 x 0.42 = 1.25 cfs.

N:\GEM\CORRESPO.OFF\RationalMethodComputationForm_100305.DOC

ILS ASSOCIATES, INC.
CIVIL ENGINEERING AND LAND SURVEYING

BY: AJS JOB NO. 8207

DATE: 2/22/08 SHEET NO: 2

PROPOSED
POST-DEVELOPMENT
HYDROLOGY
15 ACACIA RD.
FAIRFAX

RATIONAL METHOD COMPUTATION FORM

(From Cal-Trans Rainfall Intensity-Duration-Frequency Analysis & County of Marin Hydrology Manual Revised 8/2/00)

$$Q = C \times I \times A$$

Watershed TRIBUTARY 'A' At Point P.O.C. 1

Area = 18,548 sq. ft. = 0.42 acres.

Time of Concentration (+C)

$t_c = \frac{1.8\,(1.1\text{-}C)\sqrt{L}}{[S(100)]^{1/3}}$ ~~+5 Min.~~ $= \frac{1.8\,(1.1-.74)\sqrt{330}}{[0.47\,(100)]^{1/3}} = 3.3$

C = Runoff Coefficient* = 0.74

L = Longest run in feet = 330

S = Average Slope in ft/ft = $\frac{\Delta H}{L} = \frac{154}{330} = 0.47$

USING KIRITCH METH
$t_c = 0.9$ m
See Sheet 1

Intensity

P_{60} (chart I) = 1.5 zone (chart V) = C subzone (chart v) = 2

C2
.7 | .67

I_{100} (chart k) = 4.3 Rd_{10} (chart k) ______

$I_{10} = I_{100}$ 4.3 x Rd_{10} (chart k) 0.7 = 3.0 in/hr.

I ______ = Rd ______ (from Chart R) x I_{100} ______ = ______ in/hr.

Coefficient of Runoff

[NEW IMPERVIOUS AREA = 2,022 sq. ft = 0.046 Acres $\frac{.046}{.42} = 11\%$]

Relief = .40 = .40

Soil infiltration = .89(.10) + .11(.20) = .11

Vegetal cover = .89(.05) + .11(.20) = .07

Surface storage = .89(.15) + .11(.20) = .16

C = 0.74 (0.70 minimum)*

Peak Discharge Q = C x I x A

Q 10 = 0.74 x 2.98 x 0.42 = 0.93 c.f.s.

Q 100 = 0.74 x 4.25 x 0.42 = 1.32 c.f.s.

79 GALLI DRIVE, SUITE A NOVATO, CA 94949-5717 (415) 883-9200 FAX (415) 883-2763

ILS ASSOCIATES, INC.

CIVIL ENGINEERING AND LAND SURVEYING
79 GALLI DRIVE, SUITE A
NOVATO, CA 94949-5717
Ph (415) 883-9200 • Fax (415) 883-2763
www.ilscels.com

PROJECT 15 Acacia PROJECT# 8207
SHEET NO. 3 OF 3
CALCULATED BY A J S DATE 2/22/08
CHECKED BY ______ DATE ______
SCALE ______

$T_c = 5$ min.

Time (min)	t_c	%	Ex. (GPM)	Prop. (GPM)
5.0	25	6	34	35
4.0	20	9	50	53
3.0	15	12	67	71
2.0	10	19	106	112
1.5	7.5	26	145	153
1.0	5	41	229	242
0.5	2.5	72	402	425
0	0	100	559	590
0.5	2.5	78	436	460
1.0	5	52	291	307
1.5	7.5	35	196	207
2.0	10	24	134	142
3.0	15	11	61	65
4.0	20	4	22	24

Q_{100} Ex = 1.25 c.f.s. × 7.48 $\frac{Gal}{cf}$ × 60 $\frac{sec}{min}$ = 559 G.P.M.

Q_{100} Proposed = 1.32 c.f.s. × 7.48 $\frac{Gal}{cf}$ × 60 $\frac{sec}{min}$ = 590 G.P.M.

CHART I

RUNOFF COEFFICIENTS FOR AGRICULTURAL AND OPEN AREAS *

RUNOFF PRODUCING CHARACTERISTICS	WATERSHED CHARACTERISTICS A RELIEF	 B SOIL INFILTRATION	 C VEGETAL COVER	 D SURFACE STORAGE
EXTREME	0.40 Steep rugged terrain average slopes greater than 30%	0.20 No effective soil cover: either rock or thin soil mantle negligible infiltration capacity	0.20 No effective plant cover; bare or very sparse soil cover	0.20 Negligible: surface depression few and shallow; drainage ways steep and small, no ponds or marshes
HIGH	0.30 Hilly with average slopes of 10 to 30%	0.15 Slow to take up water; clay or other soil of low infiltration capacity such as heavy gumbo	0.15 Poor to fair; clean cultivated crops or poor natural cover; less than 10% of area under good cover	0.15 Low; well defined system of small drainage ways; no ponds or marshes
NORMAL	0.20 Rolling with average slopes of 5 to 10%	0.10 Normal, deep loam	0.10 Fair to good; about 50% of area in good grass land, woodland or equivalent cover	0.10 Normal; considerable surface depression storage; typical of prairie lands; lakes, ponds and marshes less than 20% of area
LOW	0.10 Relatively flat land average slopes 0 to 5%	0.05 High; deep sand or other soil that takes up water readily and rapidly	0.05 Good to excellent; about 90% of area in good grass land, woodland or equivalent cover	0.05 High; surface depression storage high; drainage system not sharply defined, Lg. flood plain storage; large number of ponds and marshes

NOTE: Runoff coefficient is equal to sum of coefficients from the appropriate block in Rows A, B, C and D.

* After H. L. Cook, as published in Engineering for Agricultural Drainage, by Harry B. Roe and Quincy C. Ayres, McGraw-Hill Book Co., Inc., New York, 1954, p. 105.

Q = CIA

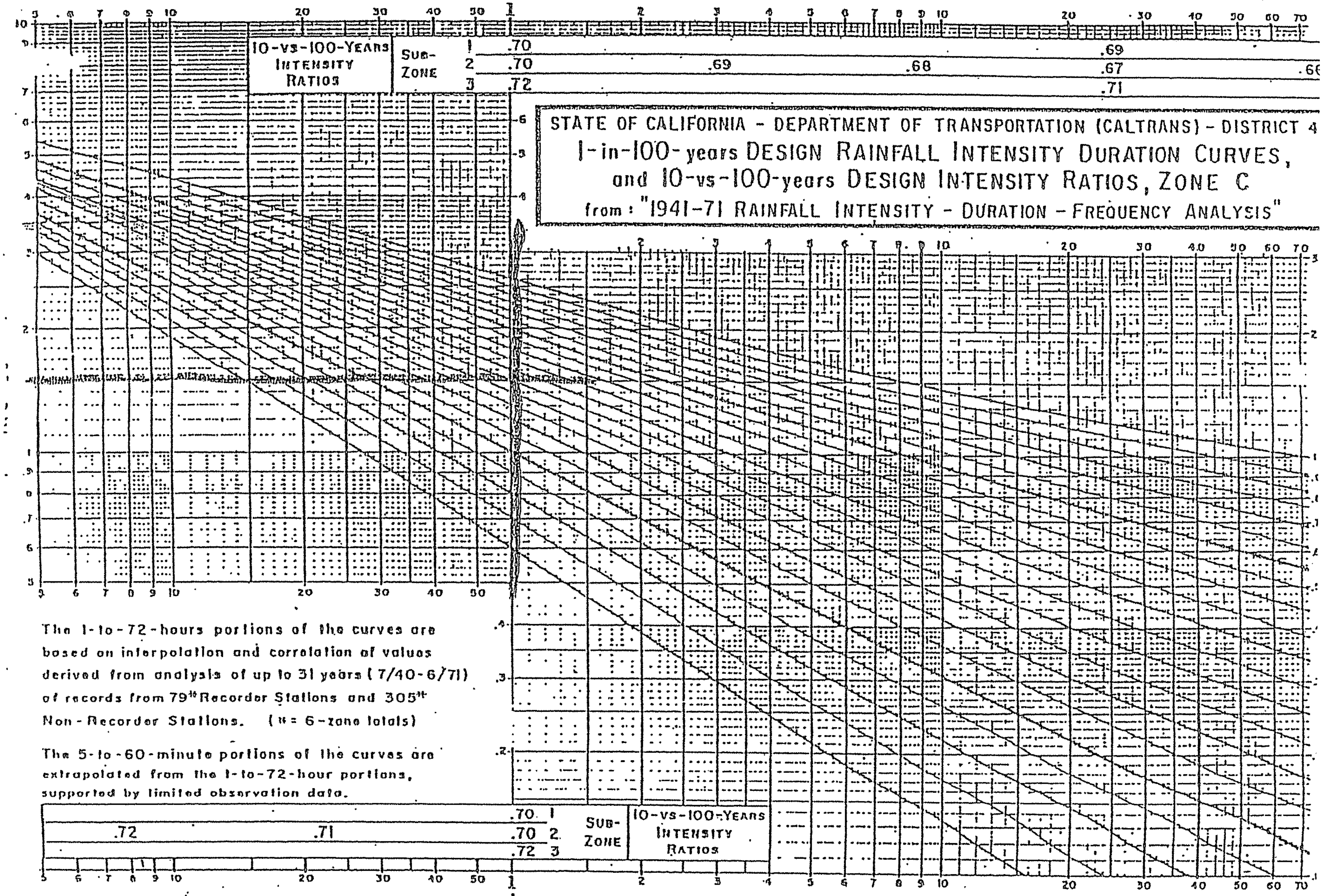
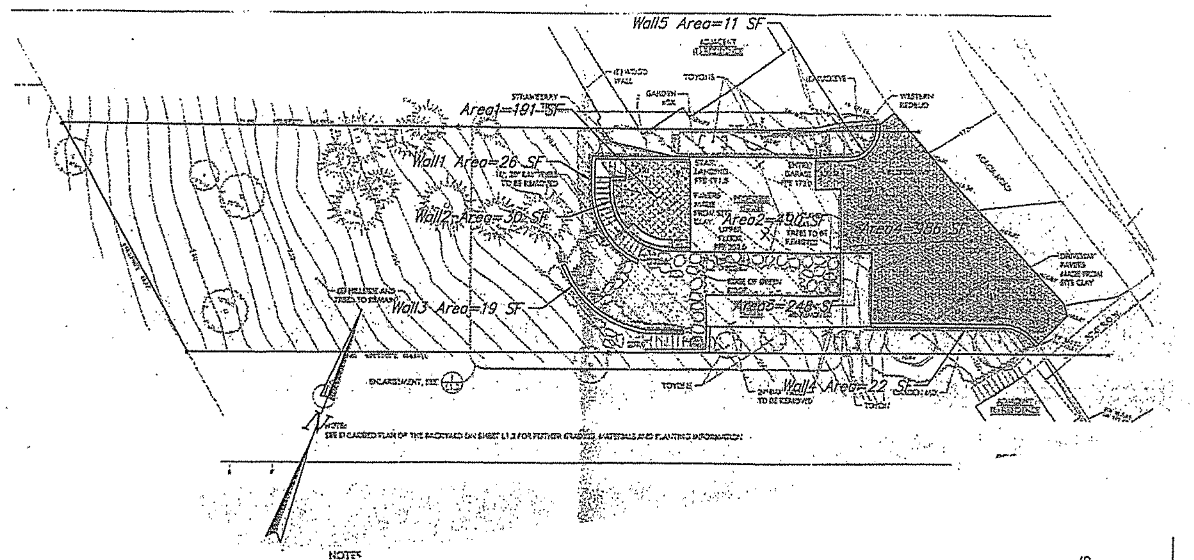


CHART "K" . ZONE C

EK/ERR
9/74



SITE MAP
NOT TO SCALE

ID	Description	Area S.F.	C value
Area1	Paver Terrace	191	0.8
Area2	Roof	490	0.9
Area3	Roof	248	0.9
Area4	Paver Driveway	986	0.8
Wall1	Concrete	26	0.9
Wall2	Concrete	30	0.9
Wall3	Concrete	19	0.9
Wall4	Concrete	22	0.9
Wall5	Concrete	11	0.9

A.P.N.: 001-112-31
FIELD BOOK NO.: ###

8207DR.dwg

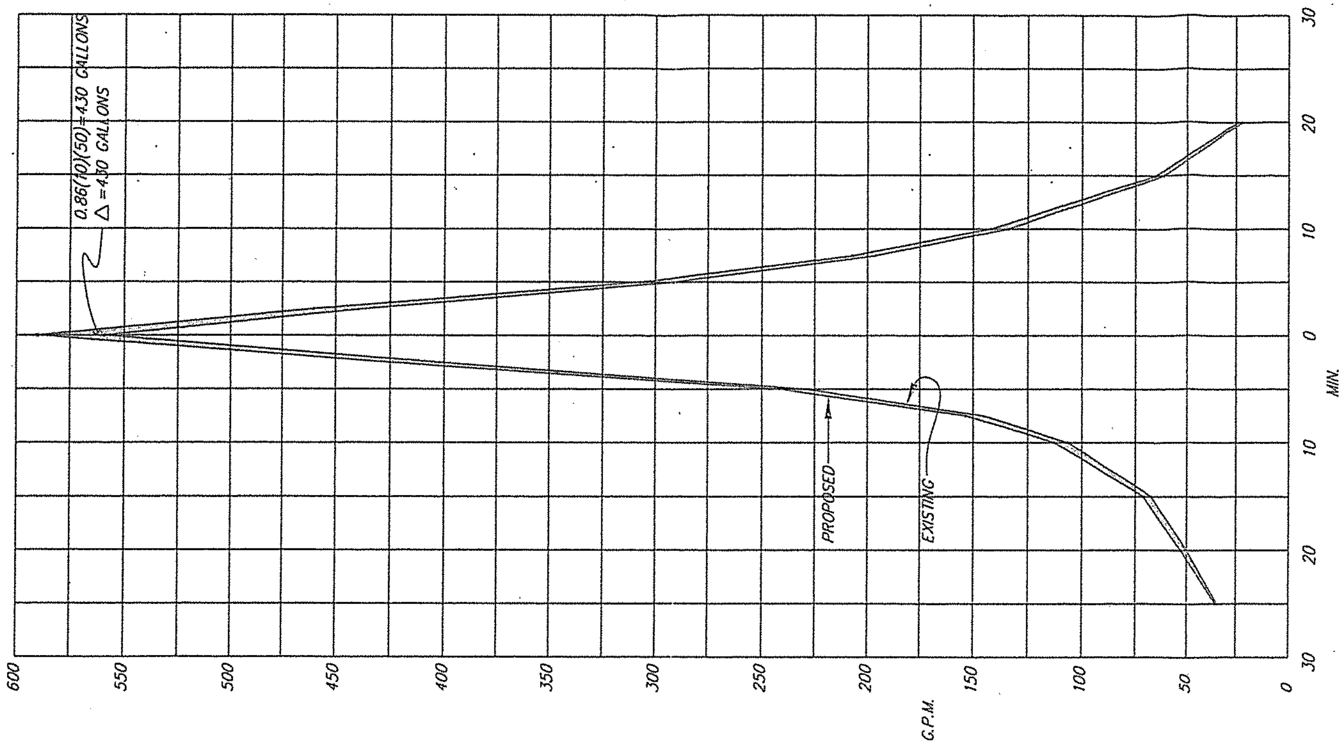
Irving L. Schwertz, C.E.
R.C.E. 18221

ILS ASSOCIATES, INC.©
CIVIL ENGINEERING AND LAND SURVEYING
79 GALLI DRIVE, SUITE A NOVATO, CA 94949-5717 (415)883-9200 FAX (415)883-2763

PUGH RESIDENCE
15 ACACIA ROAD
FAIRFAX CALIFORNIA

RUNOFF COEFICIENT WORKSHEET

DRAWN: A.J.S.
DATE: 12-11-2007
JOB NO. 8207
SHEET NO. 1 OF 1



A.P.N.: 001-112-31
FIELD BOOK NO.: ###

8207DR.dwg

Irving L. Schwartz, C.E.
R.C.E. 18221

ILS ASSOCIATES, INC.©
CIVIL ENGINEERING AND LAND SURVEYING
79 GALLI DRIVE, SUITE A NOVATO, CA 94949-5717 (415)883-9200 FAX (415)883-2763

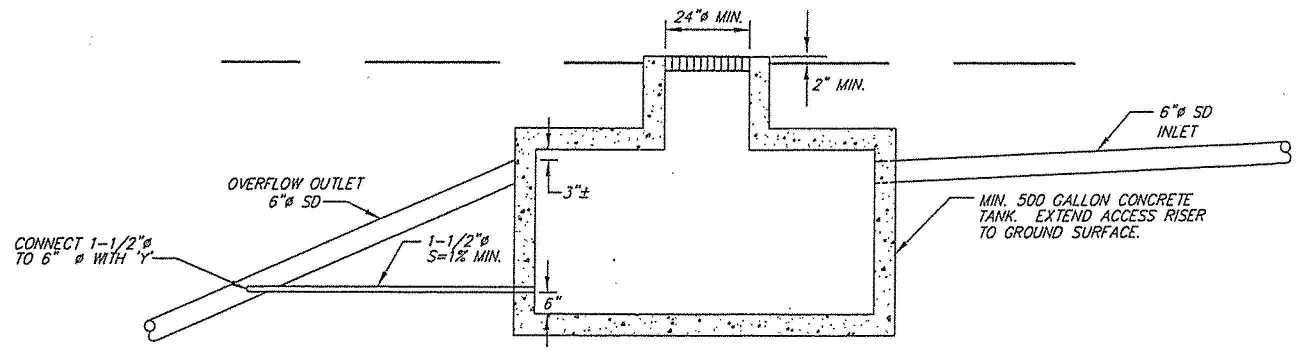
PUGH RESIDENCE

15 ACACIA ROAD

FAIRFAX CALIFORNIA

HYDROGRAPH WORKSHEET

DRAWN:	A.J.S.
DATE:	2-22-2008
JOB NO.	8207
SHEET NO.	1 OF 1



DETENTION TANK/CISTERN DETAIL

NOT TO SCALE

A.P.N.: 001-112-31
FIELD BOOK NO.: ###

8207DR.dwg

Irving L. Schwartz, C.E.
R.C.E. 18221

ILS ASSOCIATES, INC.©
CIVIL ENGINEERING AND LAND SURVEYING
79 GALLI DRIVE, SUITE A NOVATO, CA 94949-5717 (415)883-9200 FAX (415)883-2763

PUGH RESIDENCE
15 ACACIA ROAD
FAIRFAX CALIFORNIA

SCHEMATIC DETENTION SYSTEM DETAIL

DRAWN:	A.J.S.
DATE:	2-25-2008
JOB NO.	8207
SHEET NO.	1 OF 1

BHW ENGINEERS LLC
consulting structural design engineers
5 Bon Air Road, Suite 222
Larkspur, CA. 94939

T: 415-945-9587 F: 415- 945-9387 E: bhwengineers@sbcglobal.net

Mr. Ted Pugh
PO Box 447
Fairfax, Ca. 94978

March 3, 2016

Re: New House @ 15 Acacia Road / preliminary design plan approval info. request
X Town of Fairfax, CA.

Ted ,

This letter is requested to re-review the preliminary design memorandums that were issued from our office regarding the feasibility of the proposed design for planning approvals in and around 2008 for the project referenced above.

The house is proposed to be sited and cut deep into existing hillside grades as a multilevel home over a garage subterranean construction. The hillside site has deep seated loose soils from a past landslip and the proposed construction will remove nearly all of this material and the new concrete construction of the foundations required will substantially improve all stability and drainage issues on the parcel that now exists.

The critical concern is to carefully detail the concrete phases of construction to safely retain all hillside cuts with braced and tieback anchors installed in a top down construction sequence provided with safely shored vertical cuts of the levels of foundation that are required.

I am not aware of any Code changes to concrete construction since the 2007 CBC to the current 2013 code in effect now. The only increased structural requirements are under section 1704 + 1705 CBC now required on -site / 3rd party testing agencies approve concrete placement of all 3000 psi materials used and epoxy set field boltings if needed.
Geotechnical parameters are similar with only seismic base coefficients have been revised for the site soil class conditions should be updated in the Geotechnical report as needed for the base shear EQ design of the structure.

We hope this the updated review information requested at this time, if any additional information is required, please do not hesitate to contact me.

Paul J. Pieri
Senior design Engineer
@ BHW Structural Engineers

ATTACHMENT F

BHW ENGINEER LLC
consulting structural engineer
5 Bon Air Road, Suite 222
Larkspur, CA. 94939

T: 415-945-9587 F: 415- 945-9387 E: bhwengineers@sbcglobal.net

May 6, 2008

Re: Acacia Road lot development

Dear Mr. Pugh:

In response to your request, I have reviewed the May 1, 2008 memo from the Fairfax Town Engineer for your proposed home. Regarding my comments provided in the March 31, 2008 letter, a stamped copy of which is enclosed, and a response to Mr. Wrysinski's latest concerns:

1. We have addressed the purpose for, location and design of the rear patio retaining walls. It is clear that with (not excluding) the information submitted in my March 31st letter, there is a necessity for this staged construction, particularly for added site constructability and workman safety, also recognizing the benefits of separated drainage systems to propose this location of a site retaining wall.

2. My comment about potential land sliding is being misconstrued and has nothing to do with a new landslide area. The lot had a landslide that is described in Herzog's July, 2007 soils report. The upslope soil is already unstable from that landslide and subject to new creep effects from the altered soil mass below.

3. Tieback anchors will be used for the main foundation/retaining wall stability and will be located at an elevation well below the redwood tree. The design tieback layout will need to be dimensioned so as to stagger off of the drilled pier placement of the upper site wall which will employ the use of cantilever vertical drilled piers that will not impact this tree. The site cut will be evaluated for any roots exposed during construction and the project arborist can direct the final placement of this wall to ensure the health of the tree.

I hope this information re-clarifies the construction procedure proposed for this project. The progression of these ideas into structural design details and supporting calculations is required to elaborate much further into intensive reviews and plan checking responses at this phase of the design review process.

Sincerely,
Paul J. Pieri / Senior Engineer @ BHW

REGISTERED PROFESSIONAL ENGINEER
PAUL JOSEPH PIERI
3-09
C038638
CIVIL
STATE OF CALIFORNIA

BHW

To: Mr. Ted Pugh
Po Box 99485, Emeryville, CA. 94662

March 31, 2008

From: Paul Pieri
Senior Engineer @ BHW

re: Acacia Road project/ soils excavations
Fairfax, CA

These comments are in reply to your request to provide more information on the concept of how the lower retaining wall and adjacent rear patio at level 191.0 are to function and stabilize the project site vs. the appearance of more than needed excavations. This memo will focus on this component of the project.

The soils report from Herzog Geotechnical (p. 3 + plate 4) indicates the extent of the lot's unstable soil that is moving and creeping downslope. The soils boring log # 1A local to this area defines the loose soils depth around 6 to 9 foot of depth to firm and stable site rock. It is important to create a stable " Head" wall on the site and create a benched area to stage construction from on such a steep site while stabilizing the potential landslide situation. This 191.0 level area will be the primary site cut and this wall will be installed first to stabilize the upper site prior to the deeper/ tied back wall construction of the house required below.

The location and design of the lower patio retaining wall is intended to accomplish the following:

~Impede further soil creep from the upper most area of the site.
~ Remove the loose soil zone so as to keyway into bedrock the midsection of the site
~Strengthen the area in front of the redwood trees so soil won't move
~Arching shape of the wall outlay aids to increase the wall's stiffness and will conform to the hillside's natural slope and cut.
~Provide a safety margin as it separates the upper hillside soil from the house construction
~Reduces soils pressure on the house's retaining wall foundation because it will be placed with deeper drilled piers

The lot's drainage requirements and the wall's purpose should also be factored in. this wall will help cutoff subterrain water and pressure by controlling it with a higher level of backdrainage so that not only one subdrainage system exists.

I understand you will obtain more comments from ILS Associates with this patio area also be used for the site water control cistern /overflow tank location.

I hope this is the clarifications you require at this time and please don't hesitate to contact me if you have any additional questions or concerns.

Sincerely,
Paul J. Pieri / Senior Engineer @ BHW

REGISTERED PROFESSIONAL ENGINEER
PAUL JOSEPH PIERI
3·09
C033638
CIVIL
STATE OF CALIFORNIA

BHW

BHW ENGINEER LLC
Consulting structural engineer
5 Bon Air Road, Suite 222
Larkspur, CA. 94939

T: 415-945-9587 F: 415- 945-9387 E: bhwengineers@sbcglobal.net

Mr. Ted Pugh
P.O. Box 99485
Emeryville, CA 94662

Jan. 10, 2008

Re: Acacia Road lot & proposed Residence, Fairfax, CA.

Dear Mr. Pugh,

This letter is in reply to a request for information clarifications by the Fairfax town Engineer, Ray Wrysinki's Nov. 1 memo to the town in response to your application submittal information of October, 2007 to the Town of Fairfax.

Four structural issues in need of clarification;

A). comments pg. 2
No work in terms of excavations should be allowed during winter months where wet soil conditions increase the stabilization, shoring and excavation risks. We suggestion a start date of no earlier than April 15 and concrete construction and drainage substantially completed by October 15.
(6 month window for foundation work required.)

B). comments pg.3
Staged and phased levels of excavation cuts are required (top- down the site) to safely, temporarily stabilize and remove cut site soils. The cuts will be stabilized by the permanent wall construction or temporary gunnite shoring locally if loose soils zones are uncovered. Phased excavation work point grades will be detailed into the plans to indicate the soil removal depths and stage the shoring cuts for safety during the site excavation procedure.

C). comments pg.3
The soils pressures recommended by Herzog Geotechnical for "restrained" wall conditions of 75 pcf / equivalent fluid pressure is conservative for the entire height of the wall to be designed. The soils borings show hard rock at 8 ½ foot depths and this pressure (wet loose soils) will be significantly less where hard rock excavations (+/- 12 foot depths or more) will be encountered and possibly require jack hammering for removal by hand. The overall design is very conservative for tie- back pull out safety anchors proposed with each wall tie to be pull tested for performance. With the full continuous wall back drainage systems and an overflow release weep pipe cast into the basement areas of the lower wall construction the event of subsurface water flow increasing the design pressure on the wall system is unlikely.

D). comments pg. 3
Tiebacks at the northern side of the excavations will not be required because we propose using

braced " pile construction, where the final steel floor girders will be used as struts across the site to braced the phases of top down site excavations. Again the structural details will accurately define the place ments of theses beams to shore the pile used to brace the soils cuts of the north side in the structural plans and details.

REGISTERED PROFESSIONAL ENGINEER
PAUL JOSEPH PIERI
3-09
C036638
STATE OF CALIFORNIA

Sincerely,

Paul J. Pieri/ Senior Engineer @
BHW Engineers, LLC

BHW ENGINEER LLC
Consulting structural engineer
5 Bon Air Road, Suite 222
Larkspur, CA. 94939

T: 415-945-9587 F: 415- 945-9387 E: bhwengineers@sbcglobal.net

Mr. Ted Pugh
P.O. Box 99485
Emeryville, CA 94662

August 22, 2007

Re: Acacia Road lot & proposed Residence, Fairfax, CA.

Dear Mr. Pugh,

You recently retained BHW Engineers to serve as consulting structural engineer for your Residential project in Fairfax, California. The purpose of this letter is to provide preliminary technical comments and design recommendations for use in your application to the town for approval of a single family home on the above referenced lot.

We have reviewed the following documents that will be made part of the application:

- Topographical boundary survey, April, 2007, prepared by J. L. Engineering:
- Geotechnical Investigation Report, July 25, 2007, prepared by Herzog Geotechnical, Consulting Engineers;
- Preliminary development plan, August, 2007, consisting of site and floor plans, and elevations, prepared by Jeff Kroot, Architect.

Current Situation

The lot is a wooded, narrow property of about 6,400 square feet, 42 feet wide, extending an average of 152 feet upslope from the road frontage set at an acute angle to Acacia Road.
Existing rock rip rap is at the foot of the property slope, placed by the town as a temporary soil stabilization measure after a landslide in December, 2005, during a heavy rainstorm that sent soil on to Acacia Road.

The existing hillside grade exposes a landslide scarp that varies in depth up to about 20 feet upslope. There is evidence the soil is creeping downhill. The lot's surface soils are unstable and the site requires permanent mitigation to stabilize the hillside, as indicated in the Herzog report.

In early 2007, an adjacent retaining wall was built to help stabilize the slope and existing home at 13 Acacia Road that was also undermined by the landslide. In the next phase, mitigation measures to stabilize the landslide situation on your property should be designed into the new proposed residence foundation parameters. This will require a foundation system capable of retaining high pressures and a large factor of safety above most standard retaining wall designs. Herzog's report suggests that along with excavation and removal of loose soil below the scarp, a new retaining wall be designed that extends laterally (up to the 42-foot width of the lot), to buttress potentially unstable upslope areas, designed into the home's foundation. The engineered solution will have to incorporate drilled stabilizing grouted tie back rods deep into the hillside bedrock depths to stabilize the slide zone pressure areas exposed to the house foundations.

The proposed home's first two levels incorporate a 32-wide retaining wall/ 2 story tall system as the foundation, keyed into the hillside which will support the house in a more stable bedrock cut condition and incorporate tied back wall restraint rods also deep into the site bedrock. This approach represents the higher level safety needed to restrain the upslope site soils, that when detailed into the foundation design and properly drained, would mitigate the dangerous soil conditions while providing a stable foundation for the home.

We recommend the retaining wall/foundation system's design feature the following:

✓ Over excavated "benched" cuts with keyed sub-drains installed above the main 2 level ht. retaining walls to stabilize the looser upper site soils remaining; new tieback shot walls, following Herzog's recommendations for tieback placement and lengths achieved and drainage details;
✓ Tiebacks would be used at the rear wall and sidewall adjacent to 13 Acacia Road;
✓ Foundation wall base footing excavated into all bedrock depths;
✓ Diagonal drainage trench across and under the garage slap as a "finger" drain for subterranean pressure relief;
✓ Energy dissipater box installed below the driveway prior to street storm water release.

The slope falls away on the side facing the home at 19 Acacia Road, so tiebacks will be unnecessary. We recommend underpinning the closest corner of the home at 19 Acacia, and use braced shoring with a temporary bracing strut/ thrust block keyed into bedrock to shore the excavation along this side of the property.

The integrity of a combined concrete diaphragm and retaining wall system noted above as the new home's foundation system would ensure soil stability for your lot. Also the low site walls needed in the yard area for exit ways should use a drilled pier design to reach bedrock depths required.

An easement from the owner of 13 Acacia Road for the tieback depth below the surface and a variance from the town to allow encroachment into the side yard setback areas for wall stabilizing of the slide areas required will be needed to obtain construction permission during the approval process in the design of the proposed residence.

Let me know if you have any additional questions regarding the projects structural design or concepts for the stabilization process required.

Sincerely,

Paul J. Pieri/ Senior Engineer @
BHW Engineers, LLC

REGISTERED PROFESSIONAL ENGINEER
PAUL JOSEPH PIERI
3.09
C038638
CIVIL
STATE OF CALIFORNIA

J. L. ENGINEERING

C I V I L   E N G I N E E R I N G - L A N D   S U R V E Y I N G

1539 Fourth St, San Rafael, CA 94901   Ph: (415) 457-6647   Email: jlengrs@sbcglobal.net

Date: March 4, 2016

To: Fairfax Planning Dept., Fairfax Town Hall
142 Bolinas Rd.
Fairfax, CA 94930   Eml:
Cell:
Ph: 415-453-1585

Re: 15 Acacia Road, Fairfax: Kerner/ Pugh residence **(APN 001-112-31)** **JLE#2016-019**
Project Plan Review **Subdiv Map: 5-RM-4**

To Whom It May Concern:

I have discussed this project with the owners.
I have review our civil plans and survey maps for this project as previously approved in 2009 and which are the same as is currently being used for this re-application with no significant changes.
In my opinion the project plans are suitable for a current project review as a re-application of that which was previously approved with conditions.
Please contact me if you need additional information or have questions.

Sincerely,

Jay L. Hallberg (RCE 30465)
J.L.Engineering

REGISTERED PROFESSIONAL ENGINEER
J.L. HALLBERG
No. 30465
CIVIL
STATE OF CALIFORNIA

cc: Ted Pugh
P.O. Box 447
Fairfax, CA 94978   Eml: tedpugh@msn.com
Cell: 415-710-7161
Ph:

J. L. ENGINEERING

CIVIL ENGINEERING - LAND SURVEYING
539 Fourth Street, Suite "A", San Rafael, CA 94901 Ph: (415) 457-6647 Fax: (415) 457-2517 Email: jlengrs@sbcglobal.com

RECEIVED
SEP 10 2008
TOWN OF FAIRFAX

Date: September 10, 2008

To: Ted Pugh
PO Box 2410
San Francisco, CA 94126-2410

Eml: tedpugh@msn.com

Ph : 415-710-7161
Fax : 415-954-6820

Re: **Revised Grading Volume Calculations**
15 Acacia (vacant lot) Fairfax (APN 001-112-31) **JL Job# 2007-058**

Remarks:

In coordination with the revision of the rear patio and retaining walls for the proposed project at 15 Acacia Rd., as shown on our "Site Improvement Grading and Drainage Plans" dated as revised Sept. 2008, we have recalculated the grading volumes associated with the project as shown below:

		Cut (CY)	Fill (CY)	Off-haul (CY)
Grading Volumes	Proposed Driveway	215	0	215
	Proposed Building	601	0	601
	Proposed Rear Lawn, Patio, Planter, Stairs & Retention Basin	20	4	16
	Total	**836**	**4**	**832**

As shown above, excavation for the revised rear lawn, patio, planter, stairs, stormwater retention basin and associated retaining walls will result in approximately 16 C.Y. of net off-haul, or approximately 2% of the total off-haul resulting from the excavation for the entire proposed project. If you have any questions, please contact Steve McNeely or myself at 415-457-6647.

Sincerely,

Jay L. Hallberg (RCE 30465)
J.L. Engineering
Prepared by SMc

3/3/16

REGISTERED PROFESSIONAL ENGINEER
J.L. HALLBERG
No. 30465
Exp. 3-31-2010
CIVIL
STATE OF CALIFORNIA

J. L. ENGINEERING

CIVIL ENGINEERING - LAND SURVEYING
1539 Fourth Street, Suite "A", San Rafael, CA 94901 Ph: (415) 457-6647 Fax: (415) 457-2517 Email: jlengrs@sbcglobal.com

Date: March 31, 2008

To: Ted Pugh
PO Box 99485
Emeryville, CA 94662-9485

Eml:tedpugh@masterdevco.com

Ph : 415-710-7161
Fax : 510-231-1014

Re: **Rear Patio Excavation Estimate**
15 Acacia (vacant lot) Fairfax (APN 001-112-31) **JL Job# 2007-058**

Remarks:

This memo is in reply to your request that we provide a figure for the estimated excavation to create the proposed rear patio area. We estimate that the net off-haul resulting from the excavation for the rear patio and retaining wall will be approximately 37 CY. Therefore, excavation for the rear patio and retaining wall will account for approximately 4% of the total estimated off-haul of 870 CY for the project site. Please call us at 415-457-6647 with any questions you have.

Sincerely,

Jay L. Hallberg (RCE 30465)
J.L. Engineering
Prepared by SMc

3/31/08

REGISTERED PROFESSIONAL ENGINEER
J.L. HALLBERG
No. 30465
Exp. 3-31-2010
CIVIL
STATE OF CALIFORNIA

J. L. ENGINEERING

CIVIL ENGINEERING - LAND SURVEYING
539 Fourth Street, Suite "A", San Rafael, CA 94901 Ph: (415) 457-6647 Fax: (415) 457-2517 Email: jlengrs@sbcglobal.com

Date: July 24, 2007

To: Ted Pugh
PO Box 99485
Emeryville, CA 94662-9485
Eml:tedpugh@masterdevco.com
Ph : 415-710-7161
Fax : 510-231-1014

Re: **Volume Calculations**
15 Acacia (vacant lot) Fairfax (APN 001-112-31) **JL Job# 2007-058**

Grading off-haul volumes for the proposed residence are as follows:

		Cut (CY)	Fill (CY)	Off-haul (CY)
Grading Volumes	Proposed Driveway	215	0	215
	Proposed Building	601	0	601
	Proposed Patios and Stairs	28	4	24
	Total	**844**	4	**840**

As requested, a determination of the amount of off-haul material that is likely to consist of a soil type suitable for making bricks was made. As per your conversion with Steve McNeely of our office it was assumed that the soil type described in the Geotechnical Investigation Report as "orange-brown clayey gravel", (GC), was the most desirable and that soil types "orange-brown gravelly (or sandy) clay", (CL) and "orange-brown gravelly (or sandy) clay", (CH) would also be useful. Based on the soil boring locations and boring logs a triangular irregular network (TIN) of the depth and thickness of soil of type GC, CL, and CH was created in order to estimate off-haul of soils of these types at approximately 220 C.Y. Please call Steve McNeely or myself at 415-457-6647 if you have any questions.

Sincerely,

Jay L. Hallberg (RCE 30465)
J.L. Engineering
Prepared by SMc

2/3/16

REGISTERED PROFESSIONAL ENGINEER
J.L. HALLBERG
No. 30465
Exp. 3-31-2008
CIVIL
STATE OF CALIFORNIA

MARIN TREE SERVICE, INC.
Specializing in Tree Preservation
415-472-7105

March 4, 2016

Ted Pugh
P.O. Box 447
Fairfax, CA 94978

Dear Mr. Pugh:

On March 3, 2016 I inspected the landscape at 15 Acacia, Fairfax. The inspection of all trees was made from the ground and involved inspection of the external features only. No invasive, diagnostic or laboratory testing was carried out. The identification of these trees was based on broad features visible at the time of inspection.

Arborists are specialists who use their education, knowledge, experience, and training to provide proper care and professional evaluations and diagnosis of individual trees. Arborists attempt to minimize the risk of living near trees while enhancing and maintaining the overall beauty and health of the trees. Recommendations by the arborist may be accepted or disregarded by the client.

Trees inherently pose a certain degree of hazard and risk from breakage, failure, or other causes and conditions. Marin Tree Service makes recommendations, to minimize or reduce these hazardous conditions but cannot guarantee to eliminate them, especially in the event of a storm or other act of nature. While a detailed inspection normally results in the detection of hazardous conditions, there can be no guarantee or certainty that all hazardous conditions will be detected.

There always will be some risk involved with all trees. With proper monitoring and care, trees can be managed. The only way to eliminate all risks is to remove the trees.

Since there are to be no changes to the project originally approved in 2009, our letter opinions and assessment are also unchanged and supportable. The original letter may be submitted for the town's current review.

If you have any questions, please do not hesitate to contact Marin Tree Service for assistance.

Sincerely,

Robert Morey

Robert Morey
Certified Arborist #176

ATTACHMENT H

MARIN TREE SERVICE INC.
Specializing in Tree Preservation
34 DeLuca Place, Suite M • SAN RAFAEL, CA 94901
472-7105
ROBERT MOREY - CERTIFIED ARBORIST #176

Theodore Pugh
P.O. Box 99485
Emeryville, CA 94662

05/07/08

RE: Examination of vacant lot at 15-17 Acacia Road, Fairfax, CA, proposed for residential development.

At your request, we have reviewed the 05/01/08 memorandum to the Town of Fairfax by the town engineer, Mr. Wrysinski. Marin Tree Service, Inc. is aware of the depth and proximity of the proposed excavation to the Coastal Redwood (Sequoia sempervirens) trees. As stated in our previous letters dated 09/25/07 and 01/09/08, the recommended excavation, top down shoring method for all walls, will avoid undermining these trees. (As previously mentioned the Tree Protection Plan should be followed where applicable.)

Drilled piers for the two rear retaining walls, instead of tieback anchors used for the main wall, will be accomplished with minimal impact to the tree roots. Based on my experience, a relatively low percentage of the root system will be encountered, considering the entire root-ball. If encountered, the roots should be trimmed in one clean plane and I feel the trees will tolerate this minimal root disturbance.

The lower wall curves away from the root-ball and the majority of excavation will be over ten feet away with an eight foot wall section at eight feet away from the closet tree. Redwood trees are vigorous trees and lend themselves to moderate root loss. In this case, there will be no soil compaction and no changes to the majority of the root system. Though the construction will be closer than previously thought, the result is that with the wall design as proposed, these Redwoods will be uncompromised.

Wall construction will stabilize the unstable upslope area and allow for better drainage. Minimal impact during excavation is a sensible solution for permanent soil stability. When combined with the new residential foundation further down slope, the trees will be provided long term protection.

Should you have further questions or concerns, please feel free to contact us.

Regards,

Robert Morey

Robert Morey
Certified Arborist #176
RM/dac

MARIN TREE SERVICE INC.

Specializing in Tree Preservation

34 DeLuca Place, Suite M • SAN RAFAEL, CA 94901

472-7105

ROBERT MOREY - CERTIFIED ARBORIST #176

Theodore Pugh
PO Box 99485
Emeryville, CA 94662-9485

01/09/08

Re: Acacia Road lot, Fairfax, CA

As requested by the town engineer, this letter is in response to comments in his November 1, 2007 draft memorandum to the Town of Fairfax. Craig Herzog of Herzog Geotechnical has provided a response letter dated December 18, 2007, which we have reviewed.

As indicated in our letter of September 25, 2007, the proposed tieback systems anchored into the hillside for the main 32' (foot) wide retaining wall and integrated foundation are not anticipated to cause root damage or negative impact to the Redwood (Sequoia sempervirens) trees located 25' (foot) upslope. We are in agreement that utilization of the proposed "top down" methodology, with shoring methods for all retaining wall construction, including the patio retaining walls closest to the Redwood (Sequoia sempervirens) trees that will utilize drilled piers, should be sufficient to prevent undermining support for the Redwood (Sequoia sempervirens) trees.

During the construction phase, we will cooperate in the field with Herzog Geotechnical and BHW Engineers as required to monitor the construction progress and evaluate the construction to ensure adherence to the recommended construction methodologies.

Please feel free to contact us should you have further questions or concerns.

Regards,

Robert Morey

Robert Morey
Certified Arborist #176

RM/dac

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ROBERT MOREY - CERTIFIED ARBORIST #176

Theodore Pugh
PO Box 99485
Emeryville, CA 94662-9485

09/25/07

RE: Examination of vacant site intended for residential development.

On 9/24/07, I examined the vacant lot proposed for development located at 15-17 Acacia Road, Fairfax, CA. This site has had a history of instability and hill slides (slope of approximately 37 degrees with evidence of soils creep down slope). There are three Coastal Redwood (Sequoia sempervirens) trees in a cluster that are intended to be preserved, 37" 34" and 23" diameter. These trees are located on the upper right side of the lot and are presently healthy and of normal vitality and structural integrity.

During the process of home construction the hillside will be stabilized by constructing a 32 foot long rear retaining wall and integrated foundation and drainage system, which will be approximately 25' downhill from the three Redwood (Sequoia sempervirens) trees. The wall will utilize tie-back systems anchored into the hillside. The grade will remain the same within the root system of these three trees and I anticipate no root damage or negative impact due to the proposed construction.

If you should have any further questions or concerns please feel free to contact us.

Regards,

Robert Morey

Robert Morey
Certified Arborist #176

RM/dac

MARIN TREE SERVICE

Specializing in Tree Preservation

Tree Protection Guidelines

Marin Tree Service, Inc.

34 DeLuca Place, Suite M

San Rafael, CA 94901

Before development, avoid tree damage during construction by protecting the root zone. The following should be considered:

A) Physical protection of the trees can be accomplished in stages during the progression of work:

- Installing an inexpensive chain link, wire mesh, or wood fence around the drip line of trees is the most effective way to protect trees and help with tree preservation. This fence should be installed at the drip line during the initial stages of development.
- As development progresses, the fence can be moved to within 6 feet of the trunks.
- If continued progress requires access closer than 6 feet to the trunk, other precautions can be taken, such as placing hay bales around the trunks so the bark is not struck with equipment.

B) Signage: all sections of fencing should be clearly marked with signs that the area within is a tree protection zone and no one is allowed to disturb the area.

C) Root Pruning: Whenever roots over 1 inch (2.5 cm) in diameter must be severed, they should be cut flush to eliminate jagged edges. There are three methods of root pruning:

- Soil excavation using supersonic air tools, pressurized water or hand tools, followed by selective root cutting.
- Cutting through the soil along a determined line on the surface using a tool specifically designed to cut roots.
- Mechanically excavating (with trenching machine or backhoe) the soil and pruning what is left of the exposed roots.

D) Irrigate the root zone with a soaker hose allowing water to penetrate the soil to the depth of the tree roots, generally the upper 6-18" (15-45 cm) of soil.

E) Aerate the root zone: improve aeration and reduce compaction. Spread organic mulch or wood chips (2-4 inches) over the surface to reduce evaporation and conserve soil moisture and temperature.

F) Fertilization of the preserved trees before construction is recommended if nutrient deficiencies exist to boost the trees vigor and tolerance.

G) Preventive pesticide applications to reduce pest attacks should be initiated prior to construction and continued until trees have recovered from construction related stress.

H) Alternative trenching methods are available to avoid unnecessary root damage. Boring machines that tunnel under root systems and allow the installation of pipes and wires without root severance are a good alternative to trenching. If digging trenches is unavoidable, dig trenches and tunnels by hand to avoid unnecessary root damage.

I) Avoid adding backfill over the root zones of existing trees to avoid root suffocation and die back.

J) Avoid compacting soil over the root zones. Do not traffic with heavy equipment, pile debris or materials or leave equipment standing over the root zones of the trees.

K) Crown cleaning before construction is recommended to reduce the risk of branch failures in areas where people, structures, and equipment are within striking distance. When removing large limbs, the final cut should not be flush with the trunk of the tree. This removes the branch collar that contains a chemical barrier zone that controls rotting organisms. Traditional surgery paint should not be used. It is of no value and may promote rot.

Roots absorb oxygen from the atmosphere through the soil and in return release carbon dioxide (gas exchanges). Therefore, adding backfill, compressing soil, paving, etc. retards gas exchanges and limits water percolation through the soil to the roots, promoting root die back. This form of chronic stress may cause trees to die prematurely within five to twenty years after development, depending on the degree of impact. Compensation can be attempted through fertilizing, soil mulching and aerating the soil using high-pressure equipment.

JEFF KROOT ARCHITECT
& ASSOCIATES

Fairfax Planning Dept.
Fairfax Town Hall
142 Bolinas Rd.
Fairfax, CA 94930

March 1, 2016

RE: Kerner and Pugh residence
15 Acacia Road, Fairfax, CA

To Whom It May Concern:

I have reviewed the drawings that our office prepared for the Kerner and Pugh residence. The Kerner Pugh drawings are dated Oct, 2007 with the last revision dated 11/08. In my opinion the project plans are suitable for current project review and supportable today.

Please contact me if you need additional information or have questions.

Jeff Kroot

Jeff Kroot

P.O. BOX 246 • SAN ANSELMO, CALIFORNIA 94979 • FAX 415/456-2258 • TEL 415/456-5531
EMAIL: jkarch2@comcast.net • WEB SITE: www.JeffKrootArchitect.com

ATTACHMENT I

Print Close

resubmitting of project

From: **Ive Haugeland** (ive@shadesofgreenla.com)
Sent: Wed 3/02/16 7:03 PM
To: Ted Pugh (tedpugh@msn.com)

Hi Ted,

Since you are not changing anything regarding your project, the landscape plans should be suitable for re-approval as is.

Regards,

Ive Haugeland, ASLA

Principal

Shades of Green
Landscape Architecture

1306 Bridgeway, suite A

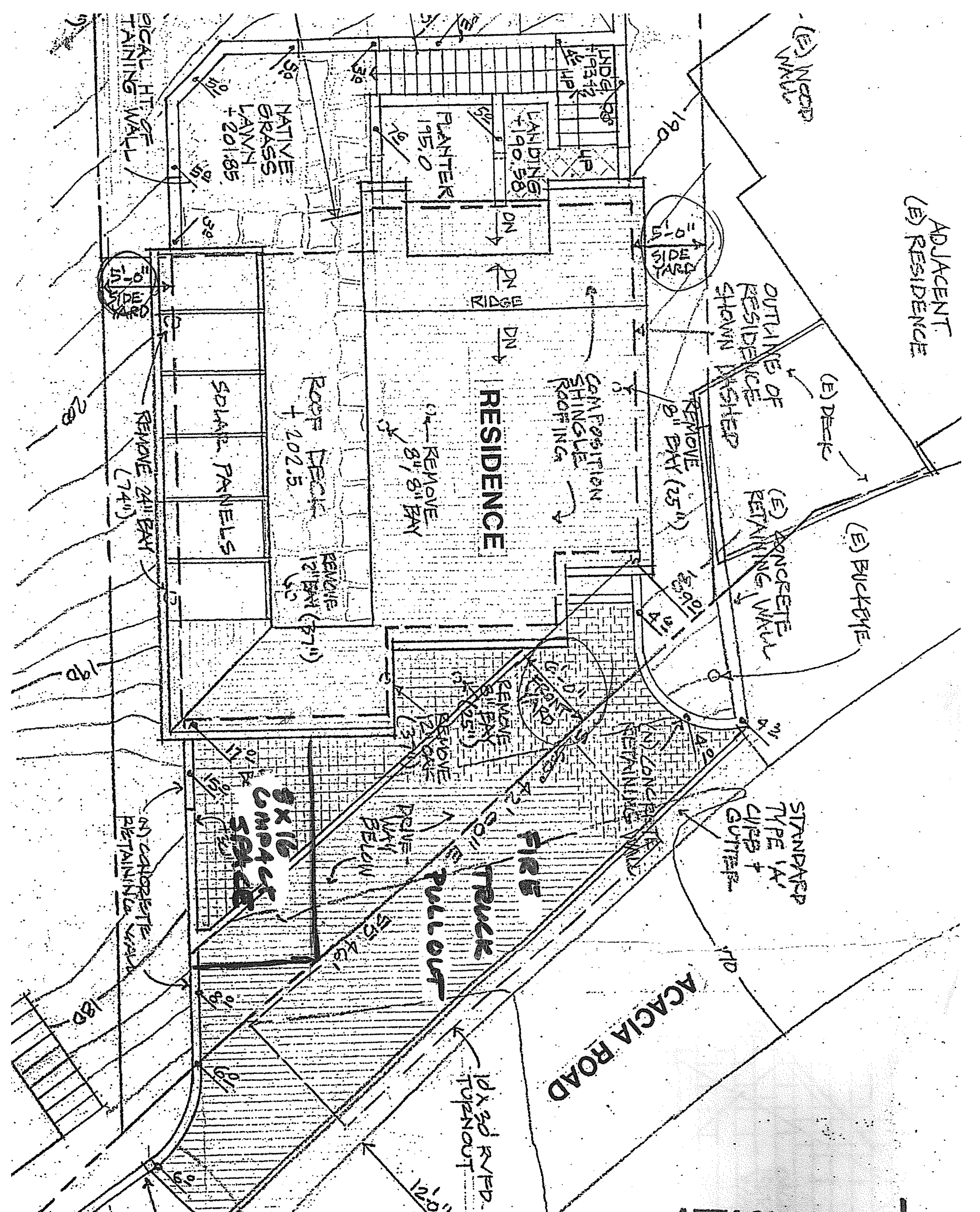
Sausalito, CA 94965

P 415 332 1485

C 415 601 1064

ive@shadesofgreenla.com

www.shadesofgreenla.com



ATTACHMENT J

15 ACACIA ROAD

Applicant's Timeline and Fact Sheet

July, 2007	Property Acquired
October,-2007	Application for new home filed
December, 2008	Planning Commission Approval
February, 2009	Design Review Board Approval
March, 2009	Town Council Approval (variance)
Summer, 2009	Construction lender IndyMac Bank failed during the financial crisis. No other financing available.
March, 2010	First one-year extension on town's approval granted
March, 2011	Second one-year extension on town's approval granted
March, 2012	Approval Expired
	From the passage of time, applicant has sustained considerable burden and financial hardship to own the property. This includes cost to obtain entitlements to build, but unable to build the home during the six-year downturn, plus costs of ownership since acquiring the property in July, 2007, a period of nine years.
	The downtown's negative effect on real estate values in Fairfax began to abate and prices improve in mid-2012.
	Construction financing for homeowner-built dwellings became available again in 2014-15 among a limited group of lenders.
December, 2015	Application for re-approval for the same and unchanged new home filed.

ATTACHMENT K

June, 2016 **Planning Commission Hearing**