Stevens Ferrone & Bailey Engineering Company, Inc

Geotechnical Engineering
Engineering Geology
Storm Water Management
Construction Observation & Testing Services

October 12, 2011

Mr. Derek Pampe
DeNova Homes
1500 Willow Pass Court
Concord, CA 94520-1010

Re: Geotechnical Feasibility Investigation
26-Acre at 451 Vine Hill Way, Martinez, CA
SFB Project No.: 155-53

Mr. Pampe

As requested, Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) has performed a geotechnical feasibility investigation for the proposed residential development on a 26-acre land at 451 Vine Hill Way in Martinez. The accompanying report presents the results of our preliminary engineering analyses. The purpose of the report is to provide information for the planning and cost estimating phases of the project.

Should you have any questions or require additional information, please do not hesitate to contact me.

Sincerely,

Stevens, Ferrone & Bailey
Engineering Company, Inc.

Ken Ferrone
Vice President

TC/KCF:ka\encl.
Copies: Addressee (1 by e-mail)

Stevens Ferrone & Bailey
Engineering Company, Inc

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October 12, 2011

GEOTECHNICAL FEASIBILITY INVESTIGATION
26-ACRES AT 451 VINE HILL WAY
MARTINEZ, CALIFORNIA
SFB PROJECT NO. 155-53

Prepared For:

DeNova Homes
1500 Willow Pass Court
Concord, CA 94520-1010

Prepared By:

Stevens, Ferrone & Bailey Engineering Company, Inc.

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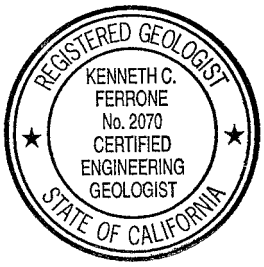

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1.0 INTRODUCTION

This report presents the results of our geotechnical feasibility investigation for a proposed new residential subdivision on 26-acres of land (APN 162-020-019) located at 451 Vine Hill Way in Martinez, California, as shown on the Site Plan, Figure 1. The purpose of our feasibility study was to evaluate the subsurface conditions and provide preliminary conclusions and recommendations regarding the geological and geotechnical engineering aspects of the project.

Based on the information indicated on the Site Plan, as well as information provided by Mr. Derek Pampe of DeNova Homes, it is our understanding that the project will consist of developing about 26 acres for a residential subdivision. The existing Pine Meadow Golf Course will be removed. Associated paved roadways and underground utilities are also proposed. Cut and fill grading is anticipated.

The conclusions and recommendations provided in this report are based upon the information presented above; Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) should be consulted if any changes to the project occur to assess if the changes affect the validity of this report.

2.0 SCOPE OF WORK

This preliminary investigation included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Performing a reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program, including drilling four exploratory borings to a maximum depth of about 11 feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing engineering analysis of the field and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the purpose of providing feasibility level geotechnical information for planning and cost estimating purposes. More detailed geotechnical studies will be necessary in the future in order to provide site specific, detailed geotechnical design and construction criteria and to confirm the preliminary recommendations provided in this report. Assessing, evaluating, and testing of onsite materials or ground water for corrosion or toxicity potential were beyond our scope of work.

3.0 SITE INVESTIGATION

A reconnaissance of the site and surrounding area was performed on October 4, 2011. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers. Four exploratory borings were drilled on October 4, 2011 to a maximum depth of about 11 feet. The approximate locations of the borings are shown on the Site Plan, Figure 1. Logs of our borings and details regarding our field investigation are included in Appendix A. The results of our laboratory tests are discussed in Appendix B. It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report.

3.1 Surface

At the time of our investigation and as shown on Figure 1, the site was bounded by Vine Hill Way on the east, Center Avenue on the south, Morello Avenue on the northwest, and existing residential developments on the west and the north. The site was occupied by the existing Pine Meadow Golf Course and associated facilities. The site was irregular in shape and had a plan area of about 26 acres with maximum dimensions of about 1600 by 900 feet. The general site grades sloped gently from the southwest toward the northeast with surface elevations ranging from about 250 feet to 170 feet (datum unknown). The existing site surface slope inclinations vary from 4:1 (horizontal to vertical) to more than 15:1. A rounded, small hill with a peak elevation of about 230 feet was located near the southeast corner of the site. A club house and associated facilities were located on top of the hill. A man-made water storage pond was located to the northwest of the hill. Large and small diameter trees were observed throughout the site with the majority locating at the site perimeter. Two water wells are reportedly located at the northeastern and southeastern corners of the site; we did not confirm their existence.

3.2 Subsurface

The near-surface materials encountered in SFB's Boring SFB-1 (located at a higher elevation) generally consisted of loose sandy fills that extended to a depth of about 1 foot. Below the surficial fills, moderately weathered sandstone of low strength and friable was encountered that extended to the maximum depth explored in the boring of about 10 feet.

Borings SFB-2, SFB-3 & SFB-4 (located at relatively low-lying areas) generally encountered 2 to 4-1/2 feet thick of surficial fills and colluvial soils below the existing ground surface. Below

the surficial soils, completely to moderately weathered sandstone of low strength and friable was encountered that extended to the maximum depth explored in these borings of about 11 feet.

These onsite surficial fills and colluvial soils encountered by SFB's borings were generally weak and potentially compressible. According to the results of laboratory testing, the near-surface more clayey materials have a high to very high plasticity and high to critical expansion potential. The onsite sandy materials generally showed no plasticity and little to no expansion potential.

Detailed descriptions of the materials encountered in the exploratory borings are presented on the boring logs in Appendix A. Our attached boring logs and related information depict location specific subsurface conditions encountered during our field investigation. The approximate locations of our borings were determined using a tape measure or landmark references and should be considered accurate only to the degree implied by the method used.

3.3 Ground Water

No groundwater was encountered in SFB's borings to the maximum depth explored of about 11 feet. All borings were backfilled with lean cement grout in accordance with Contra Costa County Environmental Heath requirements prior to leaving the site. It should be noted that our borings might not have been left open for a sufficient period of time to establish equilibrium ground water conditions. In addition, fluctuations in the ground water level could occur due to change in seasons, variations in rainfall, and other factors.

3.4 Geology and Seismicity

According to Graymer, et al. (1994), the site, below surficial soils, is underlain by Miocene Briones Sandstone (Tbr) in the northeastern half of the site and Miocene Sobrante Sandstone (Ts) in the southwestern half of the site. The Briones Sandstone consists of sandstone, siltstone, conglomerate, and shell breccia; the Sobrante Sandstone consists of gray to brown, fine- to medium-grained sandstone and minor conglomerate[1]. Nilsen (1975) mapped the low-lying area at the northwestern corner of the site as being underlain by colluvium[2].

According to U.S. Geological Survey Open-File Report 97-745 (landslide folio of the San Francisco Bay Area), the site is not mapped as having previously identified landslides or

[1]Graymer, Jones, and Brabb, 1994, *Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County*, USGS Open File Report 94-622.
[2]Nilsen, 1975, *Preliminary Photointerpretation Map of Landslide & Surficial Deposits of the Walnut Creek 7.5' Quadrangle, Contra Costa County, California*, USGS Open File Report 75-277-55.

earthflows nor is it located within an area having debris flow source potential. Based on the results of our reconnaissance and review of documents, we did not observe evidence of adverse slope stability, erosion, or drainage conditions at the site. We did not observe evidence of active, deep seated slope movement onsite or in the vicinity of the site. It is not uncommon, however, for relatively shallow slope movements to occur within the soils blanketing the site and the vicinity. These movements may include downslope creep, erosion, and slumping.

According to the Alquist-Priolo Earthquake Fault Zones Map of the Walnut Creek Quadrangle, the site is not located in an earthquake fault zone as designated by the State of California[3]. Earthquake intensities will vary throughout the San Francisco Bay Area, depending upon numerous factors including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2008) indicated that there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2008 and 2037[4]. Therefore, the site will probably be subjected to at least one moderate to severe earthquake that will cause strong ground shaking.

According to the Probabilistic Seismic Hazard Analysis (NSHMP PSHA) interactive deaggregation model developed by U.S. Geological Survey (2008), the site has a 10% probability of exceeding a peak ground acceleration of about 0.5g in 50 years (design basis ground motion based on soft rock site condition; mean return interval of 475 years). The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying unconsolidated soils.

3.5 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated, cohesionless, soil layers located close to the ground surface. These soils lose strength during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface. According to ABAG and the U.S. Geological Survey, site is located in an area mapped as having a very low likelihood of liquefaction in an earthquake and has been characterized as

[3]Hart and Bryant, *Fault-Rupture Hazard Zones in California*, CDMG Special Publication 42, Interim Revision 2007.

[4]Field, Edward H., Milner, Kevin R., and the 2007 Working Group on California Earthquake Probabilities, 2008, *Forecasting California's earthquakes; what can we expect in the next 30 years?*: U.S. Geological Survey, Fact Sheet 2008-3027, 4 p.

having very low liquefaction susceptibility[5,6]. As of the date of this report, the liquefaction potential of the site and surrounding area has not been evaluated by the State of California[7]

Based on the results of the exploratory borings, it is our opinion that the potential for ground surface damage at the site resulting from liquefaction is low due to the presence of bedrock at relatively shallow depths and the lack of saturated granular soils.

[5]Association of Bay Area Governments, 1980, *Liquefaction Susceptibility, San Francisco Bay Region.*
[6]Knudsen, Sowers, Witter, Wentworth, and Helly, 2000, "*Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California*", USGS Open File Report 00-444.
[7]Seismic Hazards Mapping Act, 1990.

4.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that development of the site is feasible for the proposed project from a geological and geotechnical engineering standpoint. The following discussion provides geologic and geotechnical considerations for development of the site.

EXISTING WEAK FILLS AND SOILS: Weak and potentially compressible fills or colluvial soils were encountered onsite that extended to depths of about 1 foot in Ts and Tbr (sandstone) areas to up to about 4-1/2 feet in low-lying Qc (colluvium) areas as shown on Figure 1. In addition, fills of various thicknesses also exist throughout the site due to existing golf course grading. Deeper weak soils should also be anticipated at the bottom of existing water storage pond. In order to reduce the potential for damaging differential settlement of overlying improvements, we recommend that these weak fills and soils, if not being removed by the proposed grading, be completely over-excavated and re-compacted.

DIFFERENTIAL EXPANSION POTENTIAL: The expansion potential of the near surface materials varies from low (sandstone and sandy soils) to very high (colluviums and more clayey soils). In order to reduce the potential for post-construction distress to the proposed foundations, we recommend that building pads be underlain by a 3 foot thick layer of engineered sandy fills mined from onsite sandstone at higher elevations. We also recommend that streets and driveways be underlain by a layer of 1 foot thick engineered sandy fills. The excavated highly expansive clayey materials can be mixed with onsite sandy materials and moisture conditioned and used as engineered fill below depths of 3 feet from finish pad grades and below depths of 1 foot from street grades, or in deeper fills and excavations.

FUTURE INVESTIGATION: A design-level geotechnical investigation including additional borings, laboratory testing, and engineering analyses will need to be performed in order to provide detailed earthwork, drainage, pavement, and foundation recommendations for use in the actual design and construction of the project. The preliminary recommendations provided below are based upon the collected data and may require modification after additional borings, laboratory testing, and analyses are performed.

4.1 Earthwork

The site should be cleared of all obstructions including existing above grade and below grade structures and their associated foundations, designated trees and their associated root systems, and the water storage pond. We estimate, at the time of our field exploration, that a stripping

depth of approximately 2 inches would be required. Holes resulting from the removal of underground obstructions extending below the finished grade should be cleared and backfilled. Water wells should be abandoned in accordance with Contra Costa County Environmental Health standards.

Weak and potentially compressible surface fills and soils exist at the site that extended to depths of about 1 to 4-1/2 feet. These fills and soils should be completely over-excavated and re-compacted if not being removed by the proposed grading. We recommend the building pads be underlain by a 3 foot thick layer of engineered sandy fills mined from onsite sandstone at higher elevations. We also recommend that streets and driveways be underlain by a 1 foot thick layer of engineered sandy fills. On-site soils and rock having an organic content of less than 3 percent by volume can be used as new fills provided the materials do not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. If required, imported fill used in building pads or streets should have a plasticity index of about 12 or less and have a significant amount of cohesive fines.

In addition to the mechanical properties specifications, all imported fill material should have a resistivity (100% saturated) no less than the resistivity for the onsite soils, a pH of between about 6.0 and 8.5, a total water soluble chloride concentration less than 300 ppm, and a total water soluble sulfate concentration less than 500 ppm.

We recommend all cut and fill slopes if no more than 5 feet high not exceed an inclination of 2:1 (horizontal to vertical) and be constructed with surface drainage. All cut and fill slopes if greater than 5 feet high should not exceed an inclination of 3:1, except for slopes consist of onsite sandstone or engineered sandy fill which can be 2:1. Structural fill should be compacted to about 90 percent relative compaction per ASTM D1557. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in uncompacted thickness.

Ponding of surface water should not be allowed at the top or bottoms of slopes, adjacent to retaining walls, foundations, or on pavement. Roof downspouts from buildings should be connected to solid pipes that transmit storm water onto paved roadways, into drainage inlets, into storm drains, or into proper storm water facilities.

4.2 Foundation Support

It is our preliminary opinion that the proposed buildings can be supported on conventional continuous and isolated spread footings bearing within the 3 foot thick layer of non-expansive engineered sandy fill layer. We anticipate that the footings can be founded at depths of about 18 inches below lowest adjacent grade depending on design bearing capacities. In conjunction with the footings, interior slabs-on-grade would be about 5 inches in thickness and supported by underlying compacted subgrade.

If post-tensioned slabs will be considered to support the buildings rather than footings, we anticipate the slabs would be approximately 10 inches in thickness.

In either case, we would recommend a vapor retarder be constructed below the slabs to reduce the potential for vapor transmission through the slabs-on-grade. Concrete should be poured directly onto the membrane.

4.3 Pavements

In areas where pavements will abut planted areas, the pavement baserock layer, pavement section subgrade soils, and trench backfill should be protected against saturation. Planned concrete slabs, sidewalks, driveways, and curb and gutters should be supported directly on properly prepared compacted sandy fills or onsite sandstone. Concrete curbs should be extended deep enough to create a water barrier between the pavement section and adjacent soil or fill.

If pavement subgrade consists of the 1 foot thick layer of engineered sandy fills mined from onsite sandstone at higher elevations, we anticipate that pavements would consist of approximately 3 inches of asphalt concrete over about 6 to 8 inches of Class 2 aggregate base.

5.0 CONDITIONS AND LIMITATIONS

Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) cannot be held responsible for the validity or accuracy of information, analyses, test results, or designs provided to SFB by others. The analysis, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from our field work and upon information provided by others. Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if changes to the project are made, SFB should be contacted immediately to evaluate the differing conditions to assess if the preliminary opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

This report is a preliminary document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of DeNova Homes and their consultants for specific application to the preliminary planning and cost estimating for the proposed residential development project at the 26-acres of land located at 451 Vine Hill Way in Martinez, California. This report is not intended to provide detailed design and construction criteria for the project. The preliminary conclusions and recommendations contained in this report are solely professional opinions.

It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. We strive to perform our services in a proper and professional manner with reasonable care and competence but it is not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; we should be consulted if it is not completely understood what the limitations to using this report are.

This report does not necessarily represent all of the information that has been communicated by us to DeNova Homes and their consultants during the course of this engagement and our rendering of professional services. Reliance on this report by parties other than those described above must be at their own risk unless we are first consulted as to the parties' intended use of this report and only after we obtain the written consent of DeNova Homes to divulge information that may have been communicated to DeNova Homes. We cannot accept consequences for un-consulted use of segregated portions of this report.

FIGURES

KEY

SFB-4 — APPROXIMATE LOCATION OF SFB EXPLORATORY BORING (10/4/11)

APPROXIMATE PROJECT LIMITS

APPROXIMATE AREA OF EXISTING FILL (ADDITIONAL AREAS LIKELY TO EXIST)

GEOLOGY (Based on Graymer, et. al., USGS OFR 94-622, 1994.)

Qc — COLLUVIUM

Tbr — BRIONES SANDSTONE (WITHIN APPROXIMATE 3')

Ts — SOBRANTE SANDSTONE (WITHIN APPROXIMATE 3')

– – –? APPROXIMATE GEOLOGIC CONTACT

44 — APPROXIMATE STRIKE & DIP OF BEDDING

NOTE: Base map was created by overlying USGS Aerial Photograph (dated 2/27/04) on Contra Costa County Assessor's Map Book 162 Page 020 (dated 1962). Locations of ground features are approximate.

APPROXIMATE SCALE: 1" = 150'

0 150' 300'

DATE: October 2011

PROJECT NO.: 155-53

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SITE PLAN & ENGINEERING GEOLOGY MAP

VINE HILL DEVELOPMENT
Martinez, California

FIGURE 1

APPENDIX A

Field Investigation

APPENDIX A
Field Investigation

The preliminary field investigation for the proposed residential development project at the 26-acres of land located at 451 Vine Hill Way in Martinez, California consisted of a surface reconnaissance and a subsurface exploration program using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers. Four exploratory borings were drilled on October 4, 2011 to a maximum depth of about 11 feet. The locations of the exploratory borings are shown on the Site Plan, Figure 1. Our representative continuously logged the soils and rock encountered in the borings in the field. The soils are described in general accordance with the Unified Soil Classification System (ASTM D2487). The logs of the borings as well as a key for the classification of the soil (Figure A-1) and rock characterization (Figure A-2) are included as part of this appendix.

Representative samples were obtained from our exploratory borings at selected depths appropriate to the investigation. Relatively undisturbed samples were obtained using a 3-inch O.D. split barrel sampler with liners, and disturbed samples were obtained using the 2-inch O.D. split spoon sampler. All samples were transmitted to our offices for evaluation and appropriate testing. Both sampler types are indicated in the "Sampler" column of the boring logs as designated in Figure A-1.

Resistance blow counts were obtained in our borings with the samplers by dropping a 140-pound safety hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of converted blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. The blow counts recorded on the boring logs have been converted to equivalent SPT field blowcounts, but have not been corrected for overburden, silt content, or other factors.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines
			SP	Poorly-graded sands or gravelly sands, little or no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, and-clay mixtures

Major Divisions		grf	ltr	Description
Soils	Silts And Clays LL < 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 silts and organic silt-clays of low plasticity
	Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
	Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZES

	U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENINGS			
	200	40	10	4	3/4"	3"	12"	
Silts and Clays	Sand				Gravel		Cobbles	Boulders
	Fine	Medium	Coarse		Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.
**Unconfined compressive strength.

SYMBOLS & NOTES

- Standard Penetration sampler (2" OD Split Barrel)
- Modified California sampler (3" OD Split Barrel)
- California Sampler (2.5" OD Split Barrel)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Ground Water level initially encountered
- Ground Water level at end of drilling

PI = Plasticity Index
LL = Liquid Limit
R = R-Value

Increasing Visual Moisture Content

Saturated
Wet
Moist
Damp
Dry

Constituent Percentage

trace	<5%
some	5-15%
with	16-30%
-y	31-49%

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KEY TO EXPLORATORY BORING LOGS

VINE HILL DEVELOPMENT
Martinez, CA

PROJECT NO.	DATE	FIGURE NO.
155-53	October 2011	A-1

ROCK MASS CHARACTERISTICS

WEATHERING

FRESH - Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer blows if crystalline.

VERY SLIGHT - Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rings under hammer blows if crystalline.

SLIGHT - Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitiod rocks, some occasional feldspar crystals are dull and discolored. Crystalline rock rings under hammer blows.

MODERATE - Significant portions of rock show discoloration and weathering effects. In granitiod rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.

MODERATELY SEVERE - All rock except quartz discolored or stained. In granitiod rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.

SEVERE - All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In some granitiod rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually remain.

VERY SEVERE - All rock except quartz discolored or stained. Rock "fabric" discernible, but rock mass effectively reduced to "soil" with only fragments of strong rock remaining.

COMPLETE - Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

STRENGTH

VERY STRONG - Resists breakage from hammer blows; but will yield dust and small chips.

STRONG - Withstands a few hammer blows; but will yield large fragments.

MODERATELY STRONG - Withstands a few firm hammer blows.

WEAK - Crumbles with light hammer blows.

FRIABLE - Can be broken down with hand and finger pressure.

LOW - Soil-like strength

DISCONTINUITY SPACING

JOINTS	BEDDING, CLEAVAGE, FOLIATION		
CRUSHED	Very Laminated	Less than 1/2 inch	Less than 1.3 cm
INTENSELY	Laminated	1/2 to 1 inch	1.3 cm to 2.5 cm
VERY CLOSE	Very Thin	1 to 2 inches	2.5 cm to 5 cm
CLOSE	Thin	2 inches to 1 foot	5 cm to 30 cm
MODERATELY CLOSE	Medium	1 foot to 3 feet	30 cm to 1 m
WIDE	Thick	3 feet to 10 feet	1 m to 3 m
VERY WIDE	Very Thick	Greater than 10 feet	Greater than 3 m

HARDNESS

VERY HARD - Cannot be scratched with a knife; metal powder left on sample.

HARD - Scratched with knife with difficulty; trace of metal powder left on samples; scratch faintly visible.

MODERATELY HARD - Readily scratched with knife, scratch leaves heavy trace of dust and is readily visible.

LOW HARDNESS - Gouged or grooved to 1/16 inch by firm pressure on knife; scratches with penny.

SOFT - Gouged or grooved readily with a knife; small thin pieces can be grooved by finger pressure.

VERY SOFT - Carves with knife; scratched by fingernail.

ROUGHNESS OF DISCONTINUITY SURFACES

SMOOTH - Appears smooth and is essentially smooth to the touch. May be slickensided.

SLIGHTLY ROUGH - Asperities on the fracture are clearly visible.

MEDIUM ROUGH - Asperities are clearly visible and fracture surface feels abrasive.

ROUGH - Large angular asperities can be seen. Some ridge and high side angle steps are evident.

VERY ROUGH - Near vertical steps and ridges occur on the fracture surface.

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KEY TO ROCK CHARACTERISTICS

VINE HILL DEVELOPMENT
Martinez, CA

PROJECT NO.	DATE	FIGURE NO.
155-53	***October 2011***	***A-2***

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY TC
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4-inch	DATE DRILLED 10/04/11

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
FILL: SAND (SM), grayish brown, fine- to medium-grained, with to silty, damp.	loose		0						
SANDSTONE, light grayish brown, moderately weathered, fine- to medium-grained, trace silt, dry. Vertical highly plastic clay seams at 1' to 2.5'.	low strength				30/2" 50/5"				
Slow drilling.	friable		5		50/2"				
Bottom of Boring = 10.2 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			10		50/2"				
			15						
			20						
			25						
			30						

EXPLORATORY BORING LOG 155-53.GPJ STEVENS FERRONE BAILEY.GDT 10/12/11

Stevens, Ferrone & Bailey Engineering Company, Inc.

1600 Willow Pass Court
Concord, CA 94520
Telephone: 925-688-1001
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EXPLORATORY BORING LOG

VINE HILL DEVELOPMENT
Martinez, CA

PROJECT NO.	DATE	BORING NO.
155-53	October 2011	SFB-1

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY TC
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4-inch	DATE DRILLED 10/04/11

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
FILL: SAND (SM), grayish brown, fine- to medium-grained, with to silty, damp.	loose		0						
FILL: CLAY (CL), mottled yellow brown, silty, with sand clasts(fine- to coarse-grained), some rock fragments, damp.	stiff				30/2" 50/2"				
SANDSTONE, light grayish brown, completely to moderately weathered, fine- to coarse-grained, trace silt, dry.	low strength friable		5		50/1"				
With ironing staining at 10'.			10		50/1"				
Bottom of Boring = 10.1 feet									
Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			15						
			20						
			25						
			30						

EXPLORATORY BORING LOG 155-53.GPJ STEVENS FERRONE BAILEY.GDT 10/12/11

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EXPLORATORY BORING LOG

VINE HILL DEVELOPMENT
Martinez, CA

PROJECT NO.	DATE	BORING NO.
155-53	October 2011	SFB-2

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY TC
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4-inch	DATE DRILLED 10/04/11

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
COLLUVIUM: CLAY (CL/CH), dark grayish brown, silty, with sand(fine- to medium-grained), some roots, dry.	stiff		0						
Change color to mottled yellow dark gray.	hard				37				
COLLUVIUM: CLAY (CL), mottled grayish brown, silty, with sand(fine- to medium-grained), trace gravel(fine, subangular), dry to damp.	very stiff				24				
SILTSTONE, light olive brown, completely weathered, sandy(fine-grained), dry.	low strength		5		51				
SANDSTONE, yellowish brown, completely weathered, fine- to medium-grained, with to silty, dry.	low strength								
SILTSTONE, light grayish brown, moderately weathered, with sand(fine-grained), dry.	low strength		10		50/4"				
Bottom of Boring = 10.8 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			15 20 25 30						

EXPLORATORY BORING LOG 155-53.GPJ STEVENS FERRONE BAILEY.GDT 10/12/11

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EXPLORATORY BORING LOG

VINE HILL DEVELOPMENT
Martinez, CA

PROJECT NO.	DATE	BORING NO.
155-53	October 2011	SFB-3

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY TC
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4-inch	DATE DRILLED 10/04/11

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
FILL: CLAY (CL), grayish brown, silty, sandy(fine- to medium-grained), dry. Roots at 1.5".	stiff		0						
COLLUVIUM: CLAY (CH), dark gray to black, silty, with sand(fine- to medium-grained), trace gravel(fine, subangular), with rootlet, dry.	very stiff				19 19	14			Liquid Limit = 52 Plasticity Index = 38
SANDSTONE, olive yellowish brown, completely to moderately weathered, fine- to medium-grained, trace silt, dry. Harder drilling below 7'.	low strength		5		92/9"				
Bottom of Boring = 10.3 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			10		50/3.5"				
			15						
			20						
			25						
			30						

EXPLORATORY BORING LOG 155-53.GPJ STEVENS FERRONE BAILEY.GDT 10/12/11

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EXPLORATORY BORING LOG

VINE HILL DEVELOPMENT
Martinez, CA

PROJECT NO.	DATE	BORING NO.
155-53	October 2011	SFB-4

APPENDIX B

Laboratory Investigation

APPENDIX B
Laboratory Investigation

Our laboratory testing program for the proposed residential development project at the 26-acres of land located at 451 Vine Hill Way in Martinez, California was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

Atterberg Limit determinations were performed on one sample of the subsurface soils to determine the range of water content over which these materials exhibit plasticity. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of the test are presented on the boring log at the appropriate sample depth.

The natural water content was determined on one sample of the subsurface soils recovered from the borings. The water content is recorded on the boring log at the appropriate sample depth.

APPENDIX C

ASFE Guidelines

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you —* should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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