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Geotechnical Investigation Report for Four New Homes

Off McKissick Street,
Pleasant Hill, California

Prepared for

Providence Development Corporation

1055 Craddock Court
Walnut Creek, CA 94597

prepared by

THE SUTTON GROUP

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September 18, 2016

Mr. Kenneth Wu
Providence Development Corporation
1055 Craddock Court
Walnut Creek, CA 994597

Geotechnical Investigation Report for Four New Homes Off McKissick Street, Pleasant Hill, California

Dear Mr. Wu:

In accordance with our signed agreement, we investigated the soil conditions on the subject 1.1 property in Pleasant Hill. You plan to re-develop the property by building four new homes on the former walnut grove.

The significant concerns we have for successful completion of the project are the the moderately expansive soils and the possibility of seismic caused liquefaction settlement with lateral spreading. It is our opinion, however, that conventional foundations, well-engineered in accordance with current standards of local residential practice in the area, will minimize the effects of these two concerns., Thick, post tensioned slab foundations can resist the cyclic shrink and swell activity of the expansive soils, and the seismic –caused settlement. Engineered pier and grade beam foundations would also resist the expansive soils but would typically be less resistant to lateral spreading and liquefaction settlement effects.

Please read the report in detail and contact us with any questions you may have with it. Our next involvement should be to review the structural and civil engineers' plans before they are submitted to the Building Department to ensure our recommendations have been interpreted as we intended

We appreciate the opportunity to be of service to you on this interesting project. Please call or email the undersigned should you have any questions, need clarification, or if you need additional information.

Very truly yours,
THE SUTTON GROUP

REGISTERED PROFESSIONAL ENGINEER
JOHN R. SUTTON
GE 812
EXP 12/31/16
GEOTECHNICAL
STATE OF CALIFORNIA

John R. Sutton, P.E/GE, D.GE
Principal Engineer

Attachment: Geotechnical Report

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1 Introduction

We have made a geotechnical investigation of the subject property, located on the west flank of Ygnacio Valley, near Boyd Road in Pleasant Hill. The property comprises two adjoining parcels, totaling 1.1 acres, each of which will be subdivided into two lots. The property includes a driveway between the houses at Nos. 60 and 98 McKissick Street. You authorized our work by signing our agreement on June 8, 2016. We present herein our findings, and our geotechnical-related recommendations for the design and construction of the new residences, site development and landscaping.

1.1 Proposed Construction

The preliminary site layout plan provided by JES Engineers (JES, 2016) indicates that the four lots will of approximately equal size on the somewhat-level property. The plan indicates the four new houses, which may be single- or two-storied, with double garages will face a common, private driveway, providing access from McKissick Street. The site will be lightly graded to direct surface drainage to the central street, which will be paved with pervious materials and possibly underlain by an infiltration basin to reduce offsite runnoff.

2 Site Conditions.

2.1 Weather

Normal annual rainfall in Pleasant Hill is 18 inches, with a third of that falling between October and December[1]. Our investigation was performed in mid 2016, in the fifth successive year of a drought cycle. While the past winter had "normal" rainfall, water use restrictions remain in force, so the site and soil conditions we have reported are dryer than average, and expect that the groundwater table is depressed below the normal.

2.2 Surface Conditions

The property is situated on the easterly-sloping, west flank of the flat-bottomed, 11 miles wide, Ygnacio Valley (Figure 1). Ygnacio Valley is drained by several creeks, which eventually drain north into Suisun Bay. In the property area, the creeks are tributary to Grayson Creek, which is tributary to Walnut Creek.

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[1] Source: Contra Costa County Flood Control District. See References at the end of the report text.

The majority of the Ygnacio Valley floor was in agricultural use until the 1940s, mainly with fruit and nut trees. Progressively the land was converted to residential development. In the 1970s, little land remained in agricultural use. This site was formerly a walnut grove, although a majority of those trees are now gone, and there are several other trees which have volunteered. The site has a surface cover of waist-high grasses. A 1939 aerial photo shows the area as undeveloped, and a 1987 photo shows the property in a similar state as today.

The JES topographical survey (Figure 3) shows elevations on the site ranging 75 to 90 feet above sea level. The land falls diagonally across the rectangular property, northwest to southeast, on a gentle, five per cent gradient. The absence of a topsoil layer, and the presence of minor fills, discussed in more detail in "Subsurface Conditions" below, indicates that the site was graded to create the walnut grove.

Adjacent residential properties to the east and south have relatively steep downslopes against the common property boundaries, and those planar lots are as much as eight feet lower than the subject site. The steep slopes down to those planar properties indicate that the land, was levelled to create those lots. The site plan also shows a northeasterly-directed, minor creek channel meandering across the landscape on adjoining, lower, land near the southeast property corner. The land and the channel's geometric alignment are indicative that the creek's alignment was modified to accommodate development and subdivision for residential use.

2.3 Geology and Geologic Hazards

2.3.1 Regional Geology

As stated above, the land is on the westerly side of the flat-bottomed Ygnacio Valley, which is rimmed by hills on the east, south and west sides. The closest hills are the Briones Hills which rise about a mile to the west, with the boundary ridge line at about 900 feet elevation. To the southeast, Mount Diablo rises to over 3,600 feet elevation. The valley bottom is filled with alluvium, i.e. gravels, sands, silts and clays, which may be a hundred feet thick over bedrock beneath the site. While Mount Diablo is comprised of hard, basaltic rocks, the Briones Hills are comprised of sandstones and claystones, some of which are erodible and others expansive. Figure 2 is a portion of the 2005 geologic map of the Walnut Creek quadrangle by Dibblee, (Dibblee and Minch, 2005).

2.3.2 Soils

According to the US Soil Conservation Service, now part of the Department of Natural Resources, the soils in the area of the site should be of the Clear Lake series. However, the soils on the property as revealed by the borings do not resemble those. Instead, they resemble the Tierra Series as mapped further to the west. In our borings, it was seen that the topsoil layer is typically absent, suggesting that the land has been graded. The Tierra soils, with a profile about 60" thick, are moderately well drained clay loams with slow permeability. These very sandy, lean clays (borderline clay-silt) have low plasticity and about 45 per cent fine sand in the upper foot, but are underlain by sandy clays of higher plasticity. Our laboratory test results, summarized on Table A-3 in Appendix A, corroborate these published properties.

The surficial clays are typically moderately expansive but those deeper are moderately to highly expansive. After becoming saturated in winter, this soil shrinks (desiccates) during the summer as the moisture evaporates due to the heat, opening surface cracks an inch or more wide. Then with the next winter wetting, it swells again. The shrink-swell movement is somewhat cyclic and the swell pressures are sufficient to heave exterior paving slabs and lightly loaded foundations. The ongoing drought can be assumed to have dried the upper soils several feet deeper than is normal, which may take a generation or more to restore, so this will exacerbate the shrink-swell cycles. This shrink-swell behavior must be managed to avoid damage to improvements, including houses. The deeper Tierra series clays are moderately to highly acidic and should be considered moderately to highly corrosive to buried metal objects. (SCS, 1978).

As stated above, the property was graded, and we found surficial soil layerss in a different order than on the soils report. This is discussed in Subsurface Conditions below.

2.3.3 <u>Seismicity</u>

There are no active earthquake faults in the immediate vicinity of the property. While the Bay Area is crisscrossed with active faults, the nearest active fault to this property is the Concord Fault, which follows the edge of the Ygnacio Valley at the foot of Lime Ridge, about four miles to the northeast. (Active Faults are so designated by the State Geologist after detailed research). Other nearby active faults are the Calaveras fault, through the San Ramon Valley, near the foot of the westerly slope, from Sunol into Danville, about 11 miles southeast of the property and the Hayward fault along the foot of the Oakland Hills through Berkeley and Oakland, is about 11 miles to the southwest. The well-known, and highly active, San Andreas Fault is on the Peninsula, in the valley west of Highway 280, about 30 miles to the southwest. On this basis, ground break is not considered a threat at this site.

While ground breakage due to fault movement is an unlikely possibility, the property is situated in the seismically active San Francisco Bay Area, which has numerous active faults. A moderate to strong earthquake event on any one of the active faults will severely shake every structure in the entire region. Even newly built houses, engineered, and built in accordance with the most recent building code provisions, can be expected to be damaged to some degree by a strong earthquake. This is because the building code requires that new houses be engineered to be ductile, i.e. the structure will flex but will remain standing after a strong earthquake. While such a house will not collapse on its occupants, there may be substantial cosmetic damage, and the cost of repairing such damage can be very significant[2]. The framing of houses could be additionally strengthened by the structural engineer, by applying greater load factors, which the building code requires in the design of schools, hospitals, and fire stations. If that is your interest, we recommend you discuss your concerns with your architect and structural engineers.

2.3.4 Liquefaction and Lateral Spreading

Liquefaction is the loss of soil strength when saturated, granular soil (relatively clean sand) is agitated in a strong earthquake, so that it becomes a liquid, losing its shear strength and thus support capability. Strain softening is the softening of saturated, soft clay-rich soil due to seismic agitation, also resulting in loss of shear strength and support capability. Lateral spreading is another related seismic-caused condition, which occurs when agitated higher ground, underlain by liquefied materials, "floats" on the liquefied layer, then slides laterally into an adjacent void, such as a creek channel.

The Contra Costa County GIS map database indicates the southeasterly part of the property, including the proposed southeasterly lot, and parts of the adjoining planned lots, to be in a liquefaction hazard zone. The ABAG Liquefaction Potential Map (ABAG, 2014) indicates the site vicinity to have low liquefaction potential. We specifically investigated for liquefiable soils, and made an analysis of liquefaction, strain softening, and lateral spreading of the developed lots. The results of the study are presented in Subsection 2.6 "Analysis", below.

2.3.5 Other Geologic Hazards

Flooding: The site is not listed as being in a flood hazard zone. We found no indication, such as denuded soil or erosion channels, that suggests flooding is common on the property. There are no water reservoirs upslope in a situation such that flooding after breach of such storage could affect the property.

Other Seismic Hazards: The site is over 75 feet above sea level, and 12 miles from Suisun Bay and the Sacramento River. Due to the general physiography with respect to the River, the San Francisco Bay Estuary, and the Pacific Ocean, the property is not at risk from tsunami. As there are no significant water bodies upslope and nearby, the site is also safe from the seismic hazard of seiche.

2.4 Site Investigation

A Mobile B24 drilling rig mounted on a light truck was used to access the boring locations, 4-inch diameter, solid stemmed, continuous flight auger was used to advance the three test borings, to 40 feet maximum depth below ground surface.

[2]Much information about earthquakes and seismic performance of houses has been compiled on the Association of Bay Area Governments (ABAG) website: www.abag.ca.gov

The borings were continuously logged by our field engineer. Samples were collected during drilling using a 3-inch OD 'California' spoon sampler fitted with 2½-inch diameter metal liners, or with a 2-inch OD "SPT" sampler. These samplers were driven using the Standard Penetration Test procedure (ASTM standard test method D1586), with the 140 pound hammer raised by rope-and-cathead, and falling 30 inches. The number of hammer blows to drive the sampler each six-inch increment, for a total of 18 inches, or part thereof in more dense materials, and the "raw N-value", the blow count for driving the final foot, was determined. This value was corrected based on sampler area as compared to the SPT sampler is shown on the drill logs. Samples were collected from the borings at approximately 5-foot depth intervals, as directed by the field engineer. Soils were classified in the field and a pocket penetrometer was used to obtain a comparative estimate of bearing capacity.

Immediately following the completion of drilling, the borings were backfilled with cement grout under the observation of the inspector from the Contra Costa County, Environmental Health Services Division.

Figure 3 shows the boring locations on the property in relation to the general locations of the planned new houses. The logs of the borings are included in Appendix B.

Samples were returned to the laboratory, whereupon all of the samples were visually check-classified by the Geotechnical Engineer. The system we use to classify and describe soil, the Unified Soils Classification System, (ASTM methods D2486 and D2487), both on the boring logs and in this report, is summarized on Figure A1 in Appendix A. A selection of samples were then variously tested for field moisture content, dry density, particle size distribution (sieve and hydrometer analysis), Atterberg Limits (plasticity), and strength in direct shear and in unconfined compression. The test results are noted on the boring logs. The test reports for the sieve and hydrometer analysis, and Atterberg Limits are included in Appendix C.

2.5 Subsurface Conditions

2.5.1 Soils:

The soil present on the surface in the borings ranged from very stiff, fat clay (CH) to very sandy lean clay (CL) or very clayey sand (SC), typically dry/desiccated, stiff to very stiff, and gray-brown to brown. In some locations near-surface soils appeared to be disturbed, suggesting that minor grading has occurred. At two to three feet depth, the soil ranged from sandy lean clay to sandy silt, although a lens of fat clay was present at from 2 to 6 feet depth in boring SB-3. Stiff to hard, sandy, lean clay and sandy silt was predominant in the borings, except for a layer of medium dense, well graded sand (SW) in boring SB-2 at 10½ to 14 feet depth, and in boring SB-1, a 3-foot thick layer of medium dense, poorly graded (medium), silty sand (SP-SM) was present below 30 feet depth. Moisture content increased with depth, although our calculations indicate that the soil was not saturated until a depth of 28 feet.

We performed soil particle size analysis (to determine fine soil characteristics and distribution) and Atterberg Limits tests (to determine the clay's plasticity range) and more particularly to evaluate liquefaction potential, and to provide input for the analysis for post tensioned concrete foundations. We also conducted strength tests on a selection of samples.

Plasticity analysis was by ASTM Test Method D4318; Particle Size Analysis was by ASTM Test Method D422. Direct shear was conducted under a partially drained condition by a modified ASTM Method D3080 procedure, and unconfined compression was by ASTM Method D2166 for determination of foundation bearing capacity. The Unified Soils Classification (USCS, ASTM Test Methods D2487 and D2488) results from these analyses. The laboratory test results are included on the boring logs and plotted on Figure A-3.

The test results indicate that these soils have low to moderately high plasticity, with Liquid Limit (LL) in the range 35 to 59 and Plasticity Index (PI) in the range 13 to 38, but the majority of soil underlying the property would have a PI of about 20. By correlation the plasticity index values indicate moderate expansion potential. Expansive clay will shrink during summer, opening surface cracks an inch or so wide and several feet deep. When exposed to wetting, such as by winter rains or irrigation, and especially leaking pipes, they will absorb moisture, which causes the swelling. This seasonal variation is somewhat cyclical and the swell pressures in these clays are sufficient to heave lightly loaded, or insufficiently resistant foundations, and paving. Hard and tough when dry, these clays, when near saturation, have low shearing strength, appearing soft and somewhat sticky.

The clays were classified as stiff near the surface and hard below about three feet depth. This is a result of desiccation over (recent) geologic time. Unconfined compressive strength tests ranged 3,300 to 12,000 pounds per square foot (psf), averaging about 6,000 psf.

These clays are also moderately corrosive to concrete and to buried metal objects (SCS, 1977). Dense, good quality, structural concrete is typically unaffected by local soil corrosivity, and many of the local ready-mixed concrete suppliers (but not all) use a cement which meets ASTM C94 Type 2-Modified, which is resistant to chemical attack. Unprotected buried metal, such as exposed (i.e. cut ends and surface scratches in ductile iron piping may be affected and should be protected against corrosion, as discussed in the Recommendations section.

2.5.2 Ground Water

Ground water was not encountered in the borings as they did not remain open long enough for water level to stabilize before they were backfilled. We calculated the saturation moisture content from the density/moisture tests and conclude that saturation would be at about 28 feet depth. From the samples, the drill logs, and from a plot of soil moisture content versus depth, we note from the moisture content of these clays that the upper ten feet are dessicated , no doubt due to the drought.

As stated above, the borings were drilled in the year following four years of drought. Over pumping of local wells during the drought has also contributed to a depressed groundwater table. Expect groundwater levels in the area to rise at least three feet presuming normal rainfall for several years, as well as due to non-controllable causes, either manmade or natural.

2.6 Analysis

We drilled boring SB-1, to 40 feet depth, on the planned southeasterly lot and made a liquefaction and lateral spreading analysis using the laboratory test results and conservative soil properties. Liquefaction, strain softening and lateral spreading can be triggered only in saturated soil.

Groundwater beneath the site was calculated to be at 28 feet depth in Boring SB-1 in the southeast part of the site. The majority of soils encountered in our investigation were relatively stiff/dense clays and clayey sands. A silty sand lens was indicated in SB-1 at from about 31 to 35 feet depth and our analysis was based on this layer. A clean sand lens was indicated at 10 to 14 feet depth in boring SB-2. It is not feasible, based on our knowledge of local geologic conditions and the back calculation, that groundwater could rise, or has risen in relatively recent geologic time, through those clays, to saturate soil at that depth.

The calculation, using the computer code LiqSVs, by Geologismiki[3] resulted in a potential for 1.2 inches settlement due to liquefaction under the design earthquake. All the site clays are stiff, and the groundwater is relatively deep, so the strain softening is unlikely. Iwasaki's Liquefaction Index, Ii, for this soil profile is 1.95. Soils with Ii less than 5 are classified as "liquefaction not probable".

We also calculated the potential for lateral spreading. Lateral spreading, and lurching are seismic effects where the seismic impulse could shift the land laterally where it is not restrained, such as a nearby downslope, especially a creek bank. The adjoining properties to the east and south, on Hubbard Avenue and on Oakvue Court were subexcavated in the 1960s, as much as eight feet below the original site grade, to create the level lots which now obtain and such a free face was created. The downslopes begin on those adjoining lots relatively close to the common property boundaries. Our analysis for the eight feet elevation difference, as shown on the JES survey indicate that the southeast lot has a potential to spread laterally 1.3 feet in the design seismic event. Where the slope is shorter, less movement can be expected, in a somewhat linear ratio to eight feet.

Our review of the site conditions and the calculations, based on the peak ground acceleration determined using the formula in the CBC and ASCE 7-10 results in a very low risk of liquefaction and lateral spreading. Houses to be built on this property, with foundation systems engineered in accordance with the recommendations in this report, are unlikely to suffer significant structural damage from liquefaction or lateral spreading

[3] LiqSVs version 1.0.i.41: SPT-based liquefaction analysis, by Geologismiki Geotechnical Software.

at the design seismic event level. The houses would move laterally as much as the calculated distance. Under such an event, expect that underground utilities would be disconnected, however, this must be considered relative to the houses being built under the current edition of the building code, whereas the adjacent houses were built about 50 years ago under a building code developed when seismic design was in its infancy. They are now located within the liquefaction hazard zone, and with shallower groundwater, so those houses are likely to suffer substantially more damage.

3 Conclusions

1. From a geotechnical engineering viewpoint, we conclude that this property can be developed with four houses, centrally sited on the lots as planned, provided the recommendations presented herein are carefully incorporated into the design and the construction, and those improvements maintained with diligence during the life of the proposed development.

2. The site has been graded and the typical topsoil layer was absent in some locations. Expect that there are areas of the site where these more expansive topsoils have been accumulated as fill which, given that the site was graded to create an orchard, was not uniformly compacted. The firmness of these soils and their moisture content must be managed and maintained at a slightly elevated level to maintain the stability of the site soils. The majority of the shrinkage and swell will occur in the upper three feet, but can extend down to about nine feet in drought. Expect that the site soils are drier at this time than in the last half century. Expect that the site clays, unusually dry to a depth of about nine feet, will absorb substantial moisture in normal and wetter seasons, and will swell in so doing. The recovery from the drought will be cyclic in the summer, and will take years. Structure foundations and hardscaping should be designed with consideration for the expansive soil behavior. Often, economics will dictate that some compromises be made in the design details and construction. If so, expect some level of ground and structure movement due to the expansive site soils if such compromises are made.

3. The site clays are moderately corrosive, which should be considered in design and construction, especially of buried metal materials.

4. The site slopes gently. Having been a fallow orchard for many years, expect that stormwater from adjoining upslope properties may have been directed onto this property, Expect also that unless surface drainage is managed, that surface runoff will flow onto adjoining downslope properties, which are at an artificially lowered elevation. Expect that crawlspaces beneath the houses, if any, especially if excavated below current site elevations, could flood in wetter weather.

5. Strong seismically induced ground shaking should be expected to occur during the life of the project. Resistance to strong seismic shaking should be engineered into the proposed houses, as well as into any other development located in the San Francisco Bay Area.
6. Our review of the site conditions and our calculations, results in a very low risk of liquefaction and lateral spreading, in any but the maximum predicted earthquake event. We calculated a potential for liquefaction settlement of up to 1.2 inches, and lateral spread of up to 1.3 feet under the maxumum earthquake. Houses to be built on this property, with foundation systems engineered in accordance with the recommendations in this report, are unlikely to suffer significant structural damage from liquefaction or lateral spreading.
7. The house and significant structures should be founded on foundation systems that can resist the pressures of moderately expansive clay soils and the design earhtquake. Thick, post-tensioned, reinforced concrete floor slabs will typically be the most efficient and also the most economical foundation system. Deep-founding the new houses on well-constructed pier-and-grade beam foundation systems will both elevate the house floors and protect against expansive soils effects, however, pier and grade beam founded houses are less able to resist the effects of liquefaction settlement.
8. Based on the soils observed in the borings, the grading and excavation of the site soils can done be with the moderately sized equipment typically used for tract-type homes on gently sloping land. Bedrock or strongly cemented soils were not encountered. Expect the dry clays will be hard and may challenge smaller excavation or drilling equipment.

4 Recommendations

4.1 General

Our recommendations are cognizant of the moderately expansive soils on the site, which are typical of those of central Contra Costa County, and the potential for seismic liquefaction and lateral spreading. As such the homes will need to be designed to resist liquefaction-caused settlement and also protected from lateral spreading where they are adjacent to downslopes.

Appropriate foundation types for development of these lots are thick structural slabs founded on the surficial, stiff clay subsoils, and drilled, cast-in-place, reinforced concrete piers (CIDH piers) inter-connected by structural concrete grade beams. The piers should bottom at sufficient depth to avoid the influence of wetting and drying of the near-surface soils and the grade beams should be isolated from the effects of expansive soils.

Site grading will entail stripping of topsoil, filling of pits after the trees are removed, and minor excavations and placement of minor fills to level the building pads and roadway. The native clayey soils should be suitable for construction of site fills if they are constructed as engineered fills in accordance with the recommendations herein. Thin concrete slabs-on-grade, such as walkways and normal thickness garage floor slabs founded on the native soils, or on engineered fills constructed from those soils, should be expected to heave and settle as the clays become moistened, and then dry, because the weight of the relatively thin slabs will be insufficient to resist those heave pressures. Improved performance of slabs-on-grade can be gained by casting those slabs on a layer of import fill comprised of non-expansive material. Exterior concrete paving should be similarly protected against moisture content change in the clays. Refer to the recommendations presented below for design and Section 5 for construction details.

4.2 Seismic Design

As stated above, this site, being in the San Francisco Bay Area, is proximate to several recognized active earthquake faults. Seismicity and soil liquefaction must be a significant consideration in structural and foundation design. The following seismic design criteria has been developed for the site, with Site Classification "D", in accordance with §1613 of the 2013 California Building Code, per the procedures in ASCE-7-10 with 2013 errata:

Site Location: Latitude = 37.941260º N
Longitude = -122.075678º W

Site Class: D, "Stiff Soil": Fa = 1.0, Fv = 1.5

Period (sec)	Spectral Acceleration (g)
0.2	$S_S = 1.656$, $S_{MS} = 1.656$
1.0	$S_1 = 0.601$, $S_{M1} = 0.901$
Design: 0.2	$S_{DS} = 1.104$
1.0	$S_{D1} = 0.601$

PGA = 0.442g, and $F_{PGA} = 1.000$

The houses, underground utilities and other improvements should be designed cognizant of the potential for 1.2 inches of liquefaction-caused settlement. This amount of settlement should be easily resisted by a thick, post tensioned concrete slab foundation system, such as the Wafflemat®. As protection against seismic lurching and lateral spreading, we recommend the houses and significant improvements are set back from the toe of the adjacent downslope by at least three times the maximum elevation differential between the adjacent lots (i.e. three times the maximum slope height).

4.3 Foundations

4.3.1 General

We recommend that the new houses be founded on a thick, post tensioned, reinforced concrete slab, which are likely to the most economical choice. At your request, we have also provided design criteria for founding the new houses on a system of CIDH piers which bottom in the stiff clays, and are structurally interconnected so that they transfer and share vertical, lateral, and moment loadings. However, our concerns for successful and economical foundations by this latter procedure relate to their ability to resist the calculated 1.2 inches of liquefaction settlement, in addition to static settlement, unless special consideration is taken in the engineering.

4.3.2 Post-Tensioned Slab Foundations

Post-tensioned slab foundations for this site should be at least 12 inches thick. In consideration of the current drought conditions, and the former agricultural use of the site, we recommend that the house pads for post tensioned slab foundations be cross-scarified, moisture conditioned, and re-compacted as an engineered fill to provide uniform bearing. Details of site preparation are provided in Section 5 below.

A slab foundation bottoming on the site clay or on engineered fill comprised of the site clay prepared as herein may be designed using an maximum allowable bearing pressure of 1,500 pounds per square foot (psf) of contact surface for dead and long term live loads, which pressure may be increased by one third for transient (wind or seismic) loads. The soil so prepared may be designed using a modulus of subgrade reaction (k) of 50pci (pounds per square inch, per inch of deflection). If a slab with edge or interior beams, such as the Wafflemat® system[4], is to be used, grade beams for the waffle slab, whether pre-stressed or conventionally designed, should be at least 12 inches wide beneath bearing walls. Other beams of a waffle type slab system may be narrower but we recommend not less than 6 inches wide.

For sliding resistance of the slab use a friction factor of 0.3. However, if a thick moisture barrier, such as Stego®Wrap is to be used beneath the slab, use a 0.2 friction factor. Recommendations for moisture barriers are provided in Section 4.4.2.

If a perimeter skirt (edge beam) is to be incorporated into the design, it may be used advantageously for resistance of lateral loads using an equivalent fluid pressure of 300pcf equivalent fluid pressure (efp). Neglect the first foot of embedment, unless there is concrete paving at least 6" thick, which extends out at least twice the depth of skirt embedment. Without paving, use 100 pcf passive resistance for the top foot of embedment.

[4] For information about the Wafflemat® foundation system, see the website www.thebestbase.com

Conventionally reinforced foundation slabs should be reinforced with the equivalent of #4 deformed bars at no more than 18" on center, each way, in both top and bottom mats, using chairs or dobies to ensure correct bar placement. We recommend against using welded wire reinforcement in lieu of deformed bars, unless the product is delivered in sheets, and is firmly fixed in place with chairs or dobies. These recommendations are to satisfy geotechnical concerns. It shall be the purview of the structural engineer to design and detail the foundations and their reinforcing steel.

For post- tensioned slab design, the following criteria are provided in accordance with the Post-Tensioning Institute's 3rd Edition procedure (May, 2008) with Addenda ("PTI-3"). In this site location and climate, the "edge lift" and "swell condition" will control the design.

For designing the slab by PTI-3, we suggest:

- A reinforced concrete slab at least 12 inches thick;
- For the Edge Lift Condition, an Edge Moisture Variation Distance, $\mathbf{e_m}$ = 2.8 feet; and a Maximum Unrestrained Differential Soil Movement, $\mathbf{y_m}$ = 0.7 inch.
- For the Center Lift Condition, an Edge Moisture Variation Distance, $\mathbf{e_m}$ = 5.4 feet; and a Maximum Unrestrained Differential Soil Movement, $\mathbf{y_m}$ = 0.3 inch.

Note re: slab design criteria: It should be recognized that in the recent past, California designers have generally presumed that slabs will be surrounded by lawns and gardens that will be irrigated, maintaining soil moisture within reasonable limits. For these reasons, some designers applied a reduction factor to the provided values. However, the combination of drought-caused watering restrictions and the economic conditions, as exemplified by the prevalence in 2009 of property abandonment, such as by foreclosure or other market gambits, can result in soil drying conditions heretofore disregarded by designers. With this experience, we recommend that the provided design values be considered in slab design without reduction.

4.3.3 Piers, and Pier-and-Grade Beam Foundations

CIDH piers should be at least 10 feet deep, as measured from the final site grade. Piers should be at least 16 inches in diameter, and reinforced with at least four #5 bars full depth and #3 hoop or helical ties, or larger where required by the building code.

Vertical Loads: Piers as described may be designed for download support in the stiff clays using a skin friction of 500 psf of shaft surface, neglecting the topmost 5 feet below the lowest adjacent final grade. For uplift resistance use 350 psf for that part of the shaft deeper than three feet below the lowest adjacent final grade. These values may be increased by one third for short-duration live loads such as wind and seismic loads.

Lateral Loads: Lateral loads may be resisted by passive soil pressures on buried foundations and pier shafts in native clay or engineered fill using a passive resistance of 300 pounds per square foot, per foot of depth, i.e. pounds per cubic foot (pcf) equivalent fluid pressure, acting over two pier diameters, to a maximum of 2,500 psf. Neglect the upper foot below adjacent grade for passive resistance, unless there is at least 5 feet width of concrete paving adjacent to the pier.

Where there is an adjacent downslope, neglect pier resistance (vertical and lateral) until a depth where there is at least 10 feet of soil between the pier and the slope face. This is illustrated on Figure C-1 in the appendix.

Refer also to Section 5 "Construction Details" below.

Grade beams that span between/across piers should be reinforced with at least two #5 bars each, top and bottom, and structurally connected to the piers. Piers should be spaced as widely as possible to concentrate the vertical loads. If piers must be closer than three diameters between centers, reduce their design capacity in linear ratio of spacing to 3 diameters. We recommend placing piers away from structure corners as the additional corner reinforcement required of the grade beam, as well as the pier reinforcement, often precludes good concrete placement.

Minimizing the width of grade beams will reduce the potential uplift pressures on their soffits. One method to mitigate the uplift pressure due to expansive soil beneath building foundations and grade beams is to create a void beneath the grade beam soffits using a cardboard void-forming product such as manufactured by Sure Void Products, Inc. of Denver Colorado. [*www.surevoid.com*] Use a 3-inch void height. (*DO NOT* use Styrofoam type material as this has substantial structural strength, which will transmit uplift load to the grade beams. While contractors' best intentions are to remove the Styrofoam after concrete placement, history shows that this removal is oft forgotten, with dire consequences).

4.3.4 Foundation Settlement and Heave

Post tensioned foundation/floor slabs for the houses as currently envisaged, are expected to remain level, with static settlement or heave less than ½ inch from the elevations measured 72 hours after stressing lockoff. Foundations, which are supported on CIDH piers, designed and built as, described herein, will likely experience less than ¾" static settlement, which may be differential over a distance of 20 feet. This settlement may be entirely differential due to the possibility that soil wetting will be local.

Seismic liquefaction could result in 1.2" settlement in addition to the above. Expect that the lenses which liquefy are local and may not extend beneath the full house footprint. We recommend allowing that all the liquefaction settlement could be differential. Lateral spreading could yield as much as 1.3 feet displacement. This would include the liquefaction settlement.

4.4 Floors

Mat slab foundation systems will be the ground floor in a post tensioned floor system. For a pier and grade beam foundation system, structural floors, of concrete or wood, supported free of the ground and spanning between grade beams over normal height crawl spaces will not be directly influenced by expansion and/or shrinkage of subsoils.

4.4.1 Garage Floors

Garage floors may be part of the post tensioned slab system, or conventional thin slabs on grade at least 4½ inches thick. Thin slab-on-grade floors, if chosen for the garages, will be subject to some degree of movement due to the underlying expansive clays, especially along the exterior edges. Experience has shown that some differential movement between slab and foundation must be expected, no matter the level(s) of protection included in the design.

Interior slabs on grade should be founded on a 4-inch thick layer of crushed rock as a capillary break. If floors are to be covered or surface-coated, consider placing a moisture barrier between capillary break and slab. Moisture barrier recommendations are provided in Section 4.4.2.

Slabs on grade should be designed structurally independent of foundation elements, with allowance for subsequent movement of underlying expansive soils. Reinforcing details are provided in Section 4.5. Do not dowel between interior (or exterior) slabs on grade to the house foundations. We suggest separating interior slabs-on-grade from foundation elements by placing ½-inch thick bituminous joint board between foundations/walls and slabs. Differential movement between slabs-on-grade and the adjoining structure can be reduced by founding the slab on Non-Expansive Fill prepared and placed as described below under Earthwork. We recommend that the non-expansive fill be at least 18 inches thick, including the thickness of capillary break material.

Where walls for rooms with thin slab-on-grade floors are to be framed for sheet rock, allowance for vertical heave and settlement of the floor against the framing must be included in the framing details. With slab preparation including non-expansive fill as presented herein, we recommend allowing ±½ inch of movement of slab against the framing.

4.4.2 Capillary Break and Moisture Barriers

If a capillary break layer beneath the slab is planned, we recommend it be of compacted, crushed rock. This material can be 1/2" to 1" maximum size but there should be less than 5 percent by weight passing the No. 4 sieve size. A capillary break layer can reduce the potential for upward migration of moisture towards the slab. We do not recommend placing a sand layer over uniformly graded rock unless the two layers are separated by a thick membrane layer or geotextile fabric.

If a moisture barrier is to be laid to protect floor finishes, we recommend it be a flexible membrane at least **15 mils thick** such as **Stego®Wrap** complying with ASTM E1745 "Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill Under Concrete Slabs". It should be placed in accordance with ASTM

E1643 "**Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs**". This is to preclude puncture during construction. The manufacturer allows 15 mil Stego®Wrap be laid directly on crushed rock without any need for a sand or geotextile cushion layer. They also recommend that concrete be poured directly on it without a sand layer over the barrier. If a sand layer is to be placed over the moisture barrier, it should be less than 2 inches thick, and the sand **must be completely dry for its full depth at the time of concrete placement.** Otherwise, moisture trapped between the barrier and the slab may negate the moisture barrier's presence, thus damaging the floor coverings.

4.5 Exterior Slabs On Grade

Slabs on grade should be separated from the structure foundations, and be allowed to move in a vertical direction freely due to the influences of wetting and drying of the site clay soils. Slabs on grade should be at least 4½" thick, supported on at least 6" of compacted Caltrans Class 2 aggregate base (See Section 5 for site preparation and compaction details). The perimeter edges of concrete walks and decking should be thickened as a "turndown," or skirt extending at least 9" below the slab surface to better resist wetting and drying influences of the soils.

Reinforce slabs with no less than #4 bars at 16" spacing both ways, centrally located. Rigid sheet product welded wire reinforcing of no less cross section than the rebar may be used. We will not accept substitution of roll-product wire mesh reinforcement. The edge turndown should be reinforced by including at least one #4 bar at the lower edge of the turndown, and continuing the horizontal slab reinforcement with lapped L-bars.

Contraction joints should be provided in concrete paving at 10-foot maximum centers, each way with reinforcing carried through the joints. Do not make T-joints in paving as they will generate cracks. Place at least two #4 bars, 4 feet long diagonally across re-entrant corners and slab openings.

Expansion joints should be provided at 30-foot maximum spacing and between paving slabs and adjacent structure concrete. The reinforcing should be cut at these joints. Install joint keys or place smooth dowels of #6 bar, 3 feet long across these joints, at 24" maximum spacing, with one end grease-wrapped, capped or coated. There should not be dowels or rebar connecting structures to exterior paving.

Expansion joints should be at least ¾" wide, and sealed with an elastomeric sealant such as W.R. Meadows' Deck-O-Seal® placed per the manufacturer's installation instructions. The lower part of the joint should be cast with a compressible backing strip which extends the full depth of the slab. We recommend that contraction joints also be sealed with a bead of the elastomer.

4.6 Grading and Drainage

Permanent slopes in the site soils will be stable if they are of engineered fill, are drained, and no steeper than 3 horizontal : 1 vertical. Ensuring a coverage of deep-rooted, dry-climate grasses, shrubs and trees will conserve the slopes against creep and erosion.

Cut slopes in the clay less than about four feet high, including utility trench walls, will be temporarily stable as vertical cuts. Taller cut slopes should be laid back per Cal-OSHA slopes for a Type B soil, or temporarily shored. In hot weather, moisture in the clay will quickly evaporate, shrinkage cracks will develop, and this apparent strength will be lost, resulting in slope collapse, so keep the clay surface moist and tarped until the structures are built. Further, dry clay will wick the moisture from freshly applied concrete causing it to dry too quickly and crack. In winter, the trenches must be protected from rains with tarps and bermed to direct runoff elsewhere.

The site should be graded to ensure that surface waters are directed away from the house and other structures. Adopt the Building Code grades as a minimum. Allowing soils to become saturated, such as in landscape planters, will result in soil expansion, which will cause nearby hardscaping to heave.

Grade new concrete paving to intercept surface waters and direct them to inlet basins and/or strip drains. Direct roof downspouts to this in-ground piped drainage system. Use 4" diameter, rigid-walled, PVC *non*-perforated, drainage pipe (Class SDR 35 or stronger), with glued joints, to discharge at the lowest elevations possible towards the downslope site boundaries. To control erosion at the outlet provide a gravel bed similar to the detail herein or seed with straw wattle. Screw a drain grate to the outlet end(s) to preclude rodent entry to the pipes. If "bubble-ups" must be used, site them and their drywell bleed pits down gradient of, and at least 10 feet distant from any structure. Typical drainage system details are included in Appendix D.

Landscaping and plantings will be necessary on all bare ground to avoid erosion. As you have engaged a recognized landscape architect, who will address these issues, we have not addressed them directly herein.

5 Construction Details

5.1 Earthwork

All earthwork and grading should be performed under the onsite oversight of the Geotechnical Engineer so that the work can be tested, verified and properly documented, as will be required by the Building Official for acceptance of the project.

5.1.1 Site Preparation

Site preparation should comprise clearing and grubbing to remove vegetation, debris, and organic-rich root zones including tree roots over ¼ inch diameter from the entire area to be re-developed. As this site was formerly a walnut grove, we recommend digging more deeply to search for, and remove decayed root mass. Any zones of organics or otherwise unsuitable fill, including any manmade fill encountered should be removed from the site.

Old concrete foundations, drains, and septic tanks, if located within the general area to be developed for the pads of the new houses, should be removed entirely and off-hauled too. These sub-excavations should be completed under the observation of the Geotechnical Engineer. The resulting low areas should then be restored to rough site grade as Engineered Fill under our oversight, as detailed below.

Then the subgrade soil should be cross-scarified (in two directions at right angles) to no less than 8 inches depth, and moisture-conditioned, i.e. dried or moistened, to between +2 and +5 percent wet of the optimum moisture content according to ASTM D1557. We recommend mixing the soil thoroughly using a disc harrow, rototiller, or similar to uniformly distribute the moisture, at least a day prior to planned compaction. Place and compact as described below under "*Compaction*".

5.1.2 Engineered Fill

The on-site or similar clay soils may be used for general site grading and backfilling. When placed and tested under the oversight of the Geotechnical Engineer's representative, in accordance with the recommendations herein, it constitutes Engineered Fill.

Cull any rock or concrete chunks greater than 3 inches in size, as well as any organic or otherwise deleterious matter. No more than 20 percent by weight should exceed 1½ inches size. Clay-rich soils should be moisture conditioned in advance of placement, and placed at between +2 and +5 percent wet of optimum. Place and compact as described below under "*Compaction*".

Compact the on-site or similar clay as engineered fill to no less than 88 percent <u>and no greater than</u> 92 percent of maximum dry density, at between +2 and +5 percent over the optimum moisture content.

5.1.3 Non-Expansive Fill

Non-expansive fill should be sands or gravels less than 3 inches maximum size, with no more than 20 percent retained on the 1½-inch sieve. There should be at least 12 percent fines (soil passing the #200 standard sieve), and the plasticity index should be less than 10. Caltrans Class 3 Aggregate Subbase is an acceptable non-expansive fill material. Recycled Class 2 Aggregate Base, if it has fines content more than 8 per cent, may be acceptable too. (Sections 25 and 26, State of California (Caltrans) Specifications). Non-expansive fill should be moisture conditioned before placement to no less than the optimum moisture content according to ASTM D1557. It should be placed and compacted as described below under "*Compaction*," at no less than 95 percent of maximum dry density, under the Geotechnical Engineer's observation.

5.1.4 Compaction

Prior to placing fill, the soils in areas to be filled should be thoroughly scarified, then moistened, and compacted as described in *Site Preparation* above. To ensure uniformity, pre-moisten all fill soil, blend by discing/thoroughly mixing, such as with a rototiller; then compact from one end of the fill area to the other, completing each individual lift in a single effort prior to beginning the next. Place each lift horizontally, not on a slope. Re-moisten previously compacted fill prior to placing a new lift on top. Care should be exercised to prevent over-compacting against grade beams and walls.

Compact in eight inch maximum loose lifts for "ride-on" compactors or four inches maximum for hand operated compactors, unless it can be demonstrated that thicker lifts can be compacted for their full thickness using the available onsite compaction equipment. Compact larger areas with no less than four passes per lift. For clays use a sheepsfoot type roller, having tamping feet no less than three inches long. (A BOMAG model BW124PD has proven successful on other small projects). Compact smaller areas in 4-inch maximum loose lifts with a Wacker type vibratory compactor or "jumping jack." Ensure that water is available on site to wet the in-place soils and trench walls between lifts, especially in hot weather. Uniformity of moisture content and firmness are most important. Compact the entire surface to be filled with uniform compactor coverage to an unyielding surface.

5.2 Utility Construction

The site soils are reportedly moderately corrosive to concrete and buried metals (SCS, 1977). Using concrete with Type 2-modified/Type 5 cement provides protection against sulfate attack. The majority of cement used in the Bay area meets this standard. We did not conduct soil corrosivity tests at this site. Where metal, such as ductile iron pipe, must be buried, it should be protected from soil corrosion by plastic wrapping. Refer to the CCCSD 2011 standards for appropriate protection.

Buried water, sewer and drainage lines occasionally leak, which can result in soil expansion. Where possible, avoid placing such lines beneath ground-supported floors on expansive clays, or alternatively, double-contain them and provide a safe outlet for the containment. Where such lines are to be placed beneath ground-supported floors, they should be pressure-tested prior to backfilling. We recommend against placing plastic lines beneath concrete slab-on-grade floors due to the possibility of undetected damage during installation and backfilling, and during construction of the slab. Take special care to protect all plastic lines against damage during backfill placement.

Bed utility lines in sand, thoroughly working it around and beneath the pipe in accordance with utility company standard specifications. Do not use sand made from crushed concrete for bedding metal pipes as it will likely be corrosive. Utility trenches should be backfilled in accordance with the requirements of Utility trenches that must parallel the sides of buildings should be more than two feet away from foundations, and above a down-sloping, 1.5H:1V plane, drawn from a line 9" above the foundation bearing level. Where this cannot be avoided, backfill with controlled density fill (CDF, also called CLSM, slurry fill and liquid backfill). Where pipes pass beneath foundations, the trench should be backfilled for 4 feet, centered on the footing, with CDF or lean concrete. Trenches should be backfilled with native site soils, except that trenches through non-expansive fill should be backfilled with like materials and compaction. Backfill compaction must be to the same specifications as Engineered Fill and Non-expansive Fill as above.

5.3 CIDH Pier Foundation Construction

Pier foundation penetration into the bearing strata should be verified and approved at the time of drilling by this Geotechnical Engineer. Should drill rig *refusal* preclude penetration to the design depth, this Engineer must be consulted. Refusal should only be determined by this Engineer and on an individual (pier-by-pier) basis.

Pier shafts should be cleaned of all loose soil, and any water removed to the satisfaction of this Engineer immediately prior to concrete placement, otherwise a tremie pipe shall be used for concrete placement. Concrete placement should occur on the day of drilling. Have a commercial-sized trash pump or bailing system on site with backups because ground water is expected. Do not leave holes open overnight if standing water is present. Instead, bail then backfill such holes with soil at the end of the day to higher than the water depth. Then, re-drill/re-inspect before re-bar and concrete placement.

Concrete overpour around the pier heads called "mushrooms" provide a shoulder against which the expansive soils will act. To prevent forming 'mushrooms' at pier heads, use cylindrical tubes to form the tops of the piers. Cardboard "Sonotube", PVC pipe, and sheet metal air ducting have been used with success. These forms should extend two feet below grade. Sonotube should be removed before backfilling but plastic and metal pipe forms can be left in place.

Grade beam and tie beam excavations should be cleared of loose soil and debris, and their bearing surfaces should be firm and unyielding to a 1" diameter probe. The side wall cuts should be formed if they are not stable. Excess (mushroom) concrete at the pier tops should be broken away before void forms are installed.

Use "dobies" or other spacers to maintain side and bottom cover to rebar during concrete placement. Concrete in foundation elements should be placed with a maximum water/cementitious materials ratio of 0.55. Use approved retarders and/or plasticizers to extend the set time as necessary and to provide necessary fluidity. Addition of water to the concrete mix on site should not be permitted without written authorization of the concrete supplier.

The Geotechnical Engineer must inspect and approve the bearing soil/rock, diameter, and depth of <u>every</u> pier at the time of drilling. Retroactive inspection is not acceptable to us. Please have the contractor give us at least three full business days' advance notice prior commencement of pier drilling, with a follow up confirmation call the day before, so that we can schedule our inspections in a manner cost effective to you. The building permit will require that we observe and approve each pier and foundation excavation immediately prior to concrete placement.

The crawlspace beneath elevated wood floors should be covered with a plastic sheet, at least 10 mils thick. This is to maintain the high soil moisture content (thus preventing soil shrinkage and swell), and to minimize evaporation, which creates a moist atmosphere in which mold can germinate. The crawl space should be well ventilated for this reason.

5.4 Crawl Spaces

The presence of water or even damp soil in crawl spaces will result in damage to wood floors, and foster the growth of mold. Place a thick plastic sheet over the crawlspace floor soon after pier drilling. To limit vapor in the crawlspace, use a polyethylene sheet at least 10 mils thick, 15 mils preferred. Thinner plastic will be destroyed during construction. Cut neat circles in the plastic at each pier. Lap the plastic up the grade beams as a seal.

We recommend crawlspaces be drained by gravity or by grading to a sump that is provided with both a primary pump and a backup/high level pump. See the drain detail D4 in Appendix D for use where the crawlspace can drain by gravity. Realize that dependence on public utilities for power to the pump could result in the pump being unavailable at a critical time, so consider a battery backup.

5.5 Slabs-on-Grade

Interior slabs on grade (i.e. conventionally reinforced, not post tensioned) should be underlain by non-expansive fill, as detailed in Section 4 and described in Section 5.1.3 above. Fill beneath concrete slabs-on-grade should be uniform in consistency and degree of compaction to provide uniform support. Proof rolling is recommended to ensure uniform firmness. Immediately following compaction, the subgrade should be moistened by sprinkling with water, then covered with a heavy plastic membrane (which may be the vapor barrier) to ensure the soil moisture content remains over the optimum until concrete placement.

All concrete slabs should be detailed with trimmer bars at internal corners to limit crack propagation. Place and cure the concrete in accordance with ACI-308, current version. Conventionally reinforced slabs should be provided with contraction/expansion joints at the spacing stated in Section 4 above. Locate control joints at all internal corners to control cracking. Do not make T-joints in slabs as they will generate cracks.

5.6 Exterior Paving

Driveways, turnarounds and parking areas may be paved with a variety of materials, including hot-mixed asphalt concrete (HMA), Portland cement concrete (PCC), pervious concrete, permeable interlocking block pavers and similar materials. Recommendations for HMA and permeable and interlocking block pavers are provided below. For recommendations for PCC paving refer to the section “Concrete Slabs On Grade” above. Recommendations for pervious concrete, and similar materials are material- and situational-dependent and will be provided upon request.

5.6.1 Hot Mixed Asphalt Paving

Hot-mixed asphalt (HMA) paving will be used on the driveway which will see infrequent heavy traffic, such as fire trucks. We used a Traffic Index (TI) of 4, a subgrade R-value of 40, and the Caltrans design procedure to result in a minimum section of 2” HMA on 6” AB. This section, presuming it is properly drained, will have a 20-year design life.

Hot mixed asphalt (HMA) should meet the specifications in use by the Caltrans District 4 Engineer for the local area. We recommend using a “½-inch max medium” mix. Aggregate base (AB) should meet the State of California (Caltrans) specifications, Section 26 for Class 2 AB. AB should be compacted to 95 percent of maximum dry density per ASTM D 1557. See also section 5.1.4 below for additional construction details.

5.6.2 Interlocking Block Pavers

Areas to be paved with interlocking block pavers (IBP) should be prepared for paving in accordance with the recommendations herein for asphalt pavements. We recommend that the IBP areas be secured by a 12” deep, concrete perimeter curb, reinforced with #5 bars top and bottom, lapped 40 diameters.

AB beneath IBP-paved areas should be at least 6” thick. The sand bed thickness and specification should be not less than as recommended by the block manufacturer.

Permeable IBP should be placed over a layered system of clean crushed rock in accordance with the manufacturer’s guidelines. Usually this comprises layers of #8 and #57 aggregate (ASTM C33) on Class 2 Aggregate Base, per Section 26, or Class 2 Permeable (crushed product), per Section 68-2.02F. of the Caltrans Specifications. As permeable paving directs infiltration to the aggregate base with the intent of saturating the subgrade, we recommend that the IBP-paved areas that will be subjected to vehicular traffic, be underlain by a structural geotextile fabric, placed between the prepared subgrade and the aggregate base. We recommend use of a structural/drainage geotextile equivalent to Ten Cate Mirafi™ RS380i.

At the junction between the IBP-paved section and the HMA-paved driveway, we recommend a subdrain, comprised of a 4” perforated pipe at least 12” deeper than the top of subgrade, and surrounded by Class 2 Permeable Material, extended to an appropriate outlet (See Appendix D5).

6 Wetting of Foundation Soils

Wetting of foundation soils always causes some degree of volume change in soils and should be prevented during and after construction. Methods of doing this include compaction of impervious fill around structures, installing water proof membranes, providing adequate grades for rapid runoff of surface waters, and collecting roof discharge water in non-perforated pipes diverting the flow to a subsurface piping system, or directing the flow well beyond the limits of the construction. Provision of paving and burying an impervious membrane with positive grade around structures provides an excellent protection against wetting of foundation soils, provided they are properly sloped and maintained. The membrane should be covered by several inches of an obvious 'non garden' material such as gravel. In garden areas, the membrane should be heavier and set deeper than 'shovel depth' to ensure its integrity.

Garden and lawn irrigation water is a leading source of foundation water. Periodically look for overly wet soil, heaved or depressed paving, distressed plants and presence of moss as problem indicators. Observe the system periodically, minimize watering time, and observe that spray heads are properly directed, and not directed against building walls. Dry-climate plantings and irrigation systems, such as drip irrigation significantly reduce the potential for over-watering, however they must be maintained. The 'spaghetti tubes' are easily dislodged inadvertently by gardening activities. These go undetected, leading to accumulation of water.

7 Plan Review and Construction Monitoring

We should be requested to review the final grading and foundation plans prior to submittal for permits. This is in order to confirm that geotechnical conditions have been interpreted with the intent of these recommendations.

This Engineer should also be requested to observe all site grading, pier and foundation excavations to verify that conditions revealed are compliant with the basis and intent of our recommendations, and to observe, and if appropriate, test the compaction of fill and backfill.

Please have the contractor give us at least three full business days advance notice prior to commencement of any pier drilling, with confirmation the preceding day, so that we can schedule our inspections and approve each of the excavations as required by the building department in a timely manner. We would then be able to prepare a final report describing and documenting the work for submittal to the Chief Building Official as part of the project documentation.

8 Limitations and Responsibilities

8.1 Engineer

The recommendations presented herein are based on the soil conditions revealed by our test borings and laboratory procedures according to state-of-the-practice methods. Samples were evaluated for the existing conditions and proposed construction according to ordinary soil-engineering standards of practice, but we did not do an investigation for hazardous materials.

Underground conditions can be expected to vary between exploration points; therefore, if any unusual conditions are encountered during construction, or if the actual construction will differ from that planned at the present time, we should be notified so we can provide supplemental recommendations. The conclusions and recommendations for this report should not be assumed to apply to properties beyond the project area.

The items in the report are valid as of the present time. However, the future may change conditions due to natural processes, works of man, legislation and the broadening of knowledge. Therefore, this report is subject to review and should be relied upon less with the passage of time in the event that delayed construction is considered. For instance, the building code is revised every three years and our recommendations may need to be altered to address new design criteria.

The geotechnical engineer may be consulted in the future to review designs and to observe and test soil-related aspects of construction. None of this subsequent work should be construed as supervision of the builders by the engineer, and no responsibility will be accepted for others' actions. The engineer will only act as an advisor to his client, and then only when he has been given sufficient notice to provide the advice.

8.2 Owner

This report is issued with the understanding that the owner chooses the risk he wishes to bear by the expenditures and savings involved with the construction alternatives and scheduling that are chosen. For this report to be valid, the owner should ensure that necessary steps are taken to carry out the recommendations of the report during design, construction, and maintenance of the project.

8.3 Builders

It is the responsibility of the contractors to investigate the soil conditions and to provide their own specific conclusions and designs regarding site preparation, excavation, trenching, filling, temporary drainage, foundation forming, erosion control, job security and safety, etc. This report is intended primarily for the use of the designers, although it should be useful as a source of construction information.

The recommendations in this report are general in nature and are subject to adoption or revision as the construction circumstances warrant. We may be consulted for supplemental advice, or to provide assistance in interpreting our findings and recommendations, or to inspect various aspects of construction. It should be recognized, however, that we are experienced geotechnical and foundation engineers but that we are not structural, hydraulics, or safety engineers.

9 References

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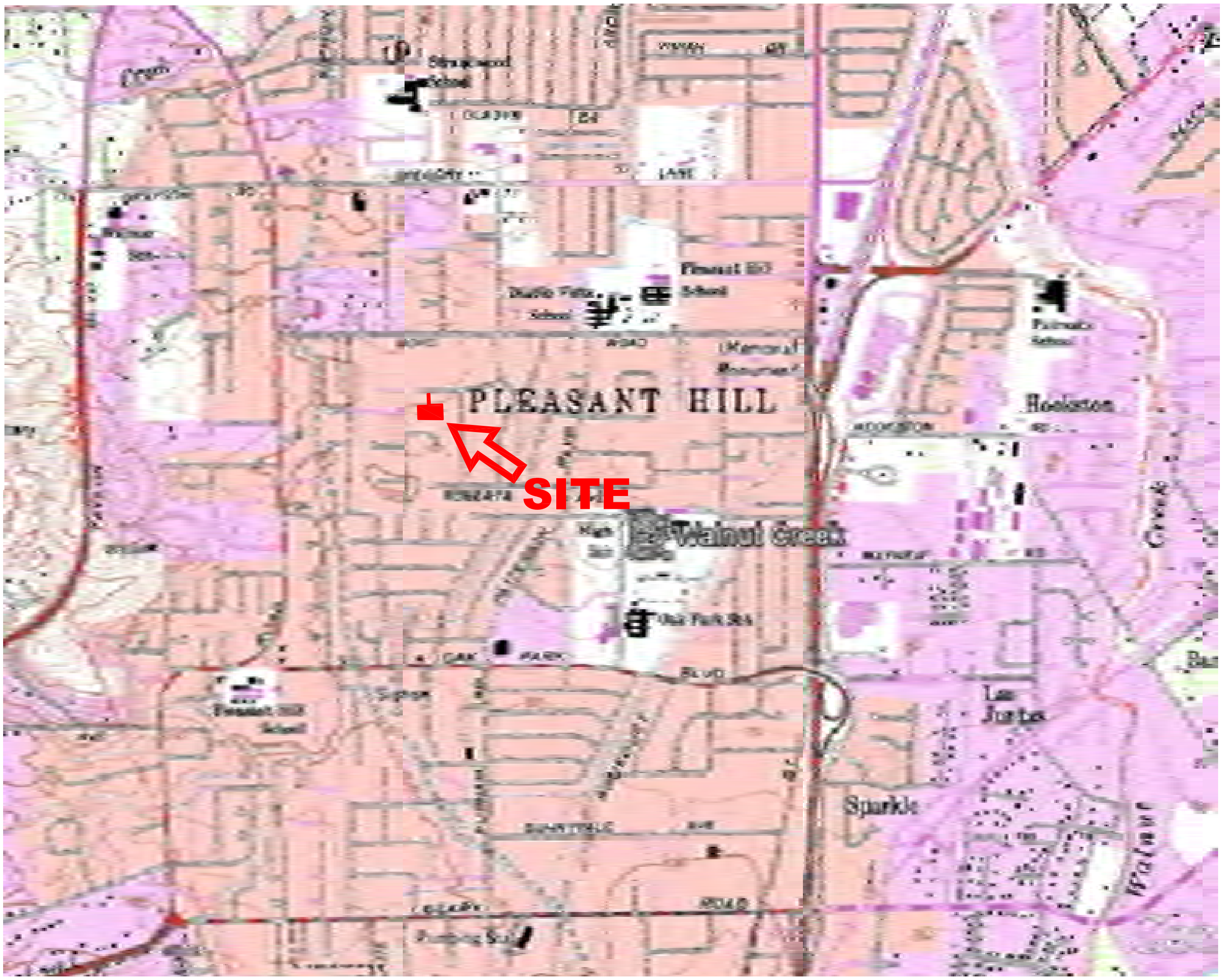
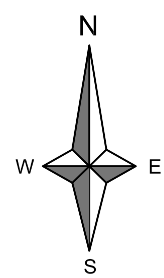
FIGURES

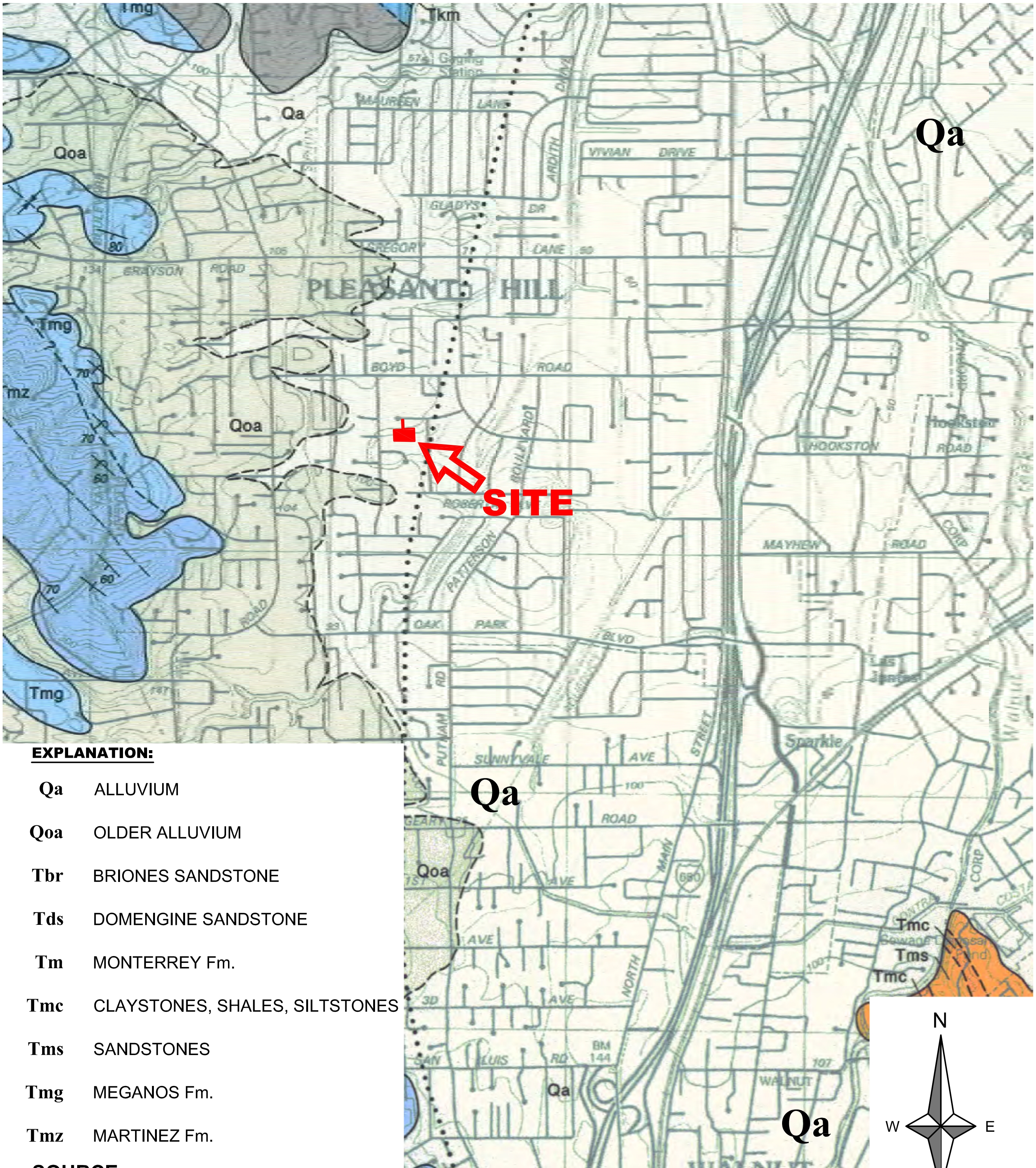




SOURCE: USGS Walnut Creek Quadrangle Map at 1:24,000

APPROXIMATE SCALE
1" = 2000'

THE SUTTON GROUP *GEOTECHNICAL ENGINEERS* 3708 Mount Diablo Blvd, Suite 215 Lafayette, CA, 94549 P: (925) 284-4208 www.suttongeo.com	**SITE LOCATION MAP** Providence Development Corp. 4 Lots on McKissick Street Pleasant Hill, CA	Job 7326 Drn: 8/31/2016 Figure No. **1**



EXPLANATION:

Qa	ALLUVIUM
Qoa	OLDER ALLUVIUM
Tbr	BRIONES SANDSTONE
Tds	DOMENGINE SANDSTONE
Tm	MONTERREY Fm.
Tmc	CLAYSTONES, SHALES, SILTSTONES
Tms	SANDSTONES
Tmg	MEGANOS Fm.
Tmz	MARTINEZ Fm.

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SOURCE:
Dibblee and Minch, 2005: Geologic Map of the Walnut Creek Quadrangle, Contra Costa and Alameda Counties, California, Thomas W. Dibblee and John A. Minch, Dibblee Geological Foundation map DF-149 at 1:24,000.

APPROXIMATE SCALE
1" = 2000' (1:24,000)

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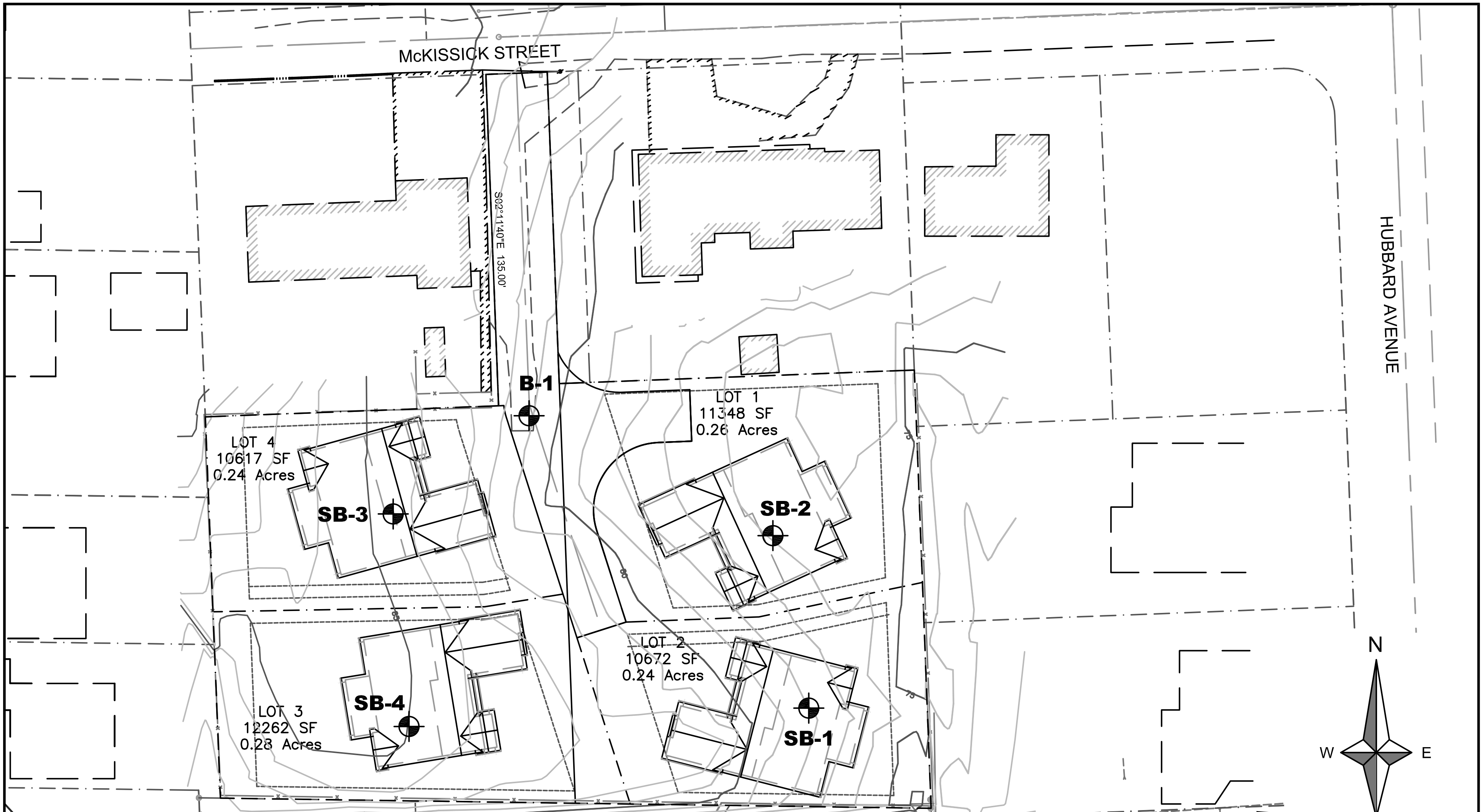
GEOLOGIC MAP
Providence Development Corp.
4 Lots on McKissick Street
Pleasant Hill, CA

Job 7326 Drn: 8/31/2016

Figure No. **2**



APPROXIMATE SCALE
1" = 60'

EXPLANATION:

B-1 = BORING BY TERRASEARCH (2004)

SB-1 = BORING, THIS INVESTIGATION

SOURCE:
Preliminary Site Plan
Providence Development Corporation
By JES Engineering

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SITE PLAN WITH BORINGS
Providence Development Corp.
4 Lots on McKissick Street
Pleasant Hill, CA

Job 7326

Drn: 8/31/2016

Figure No. ***3***

Geotechnical Investigation Report
for Four New Homes
Off McKissick Street,
Pleasant Hill, California

APPENDICES

APPENDIX A

SOIL CLASSIFICATION SYSTEM
AND
LABORATORY TEST RESULTS

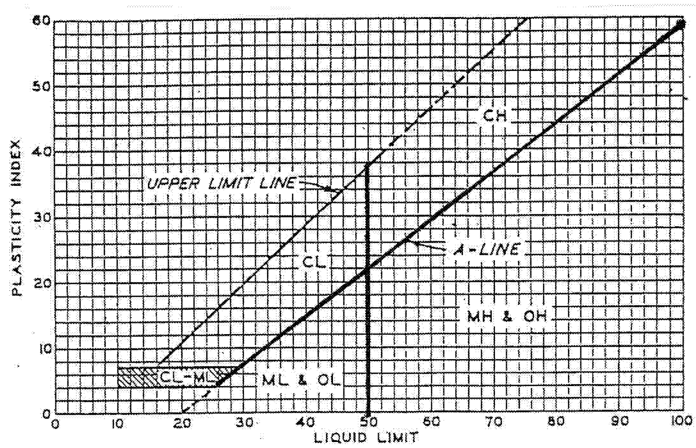
UNIFIED SOILS CLASSIFICATION SYSTEM

AFTER US ARMY CORPS OF ENGINEERS, AND ASTM D2287

Coarse Grained Soils (more than half of soil > No. 200 sieve)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Sandy gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-silt mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	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	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
Fine Grained Soils (more than half of soil < No. 200 sieve)	Silts and Clays LL = < 50	ML	Inorganic silts and very fine sands, rock flour, silty fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic silts of high plasticity, fat clays
		OH	Organic clays of high plasticity, organic silty clays, organic silts
Highly Organic Soils		Pt	Peat and other highly organic soils

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size In Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 7.76
coarse	3" to 3/4"	76.2 to 4.76
fine	3/4"to No.4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
coarse	No.4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

Grain Size Chart



Plasticity Chart

RELATIVE DENSITY / FIRMNESS, BASED ON SPT BLOW COUNT

SANDS AND SILTS

Blows / Foot	Relative Density
0 - 4	Very Loose
4 - 8	Loose
8 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

CLAYS

Blows / Foot	Relative Stiffness	Unconfined Strength (q_u, ksf)
< 2	Very Soft	< 0.5
2 - 4	Soft	0.5 – 1.0
4 - 8	Medium Stiff	1 - 2
8 - 20	Stiff	2 - 4
> 20	Very Stiff	4- - 8

WEATHERED BEDROCK

Blows / Foot	Relative Hardness
< 20	Severely Weathered
20 – 30	Firm
30 – 50	Medium Hard
50 – 80	Hard
> 80	Very Hard

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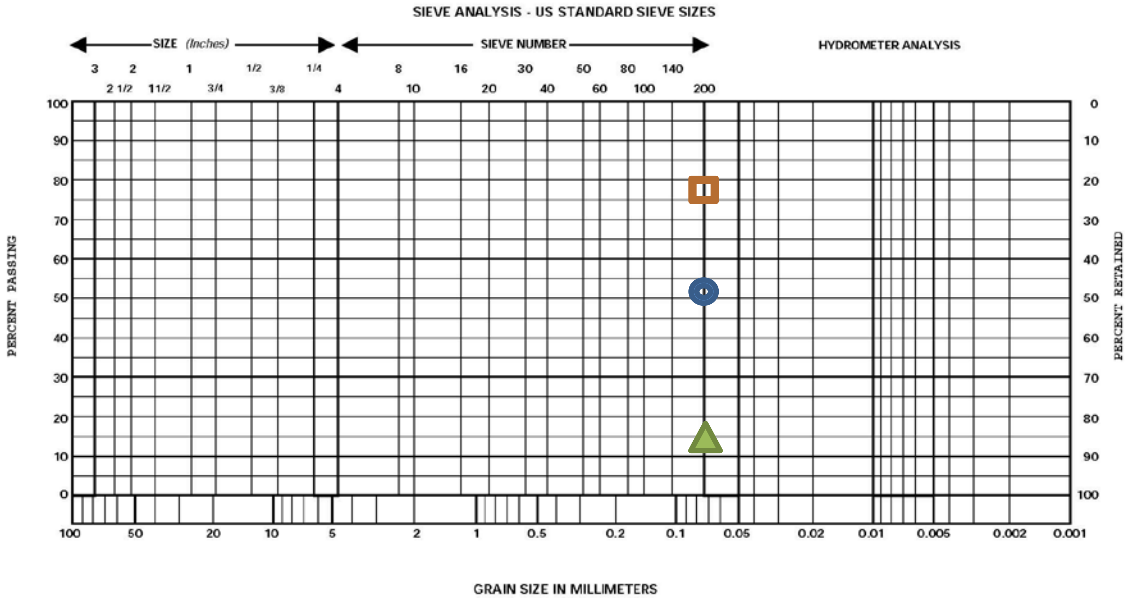
SOILS CLASSIFICATION SYSTEM

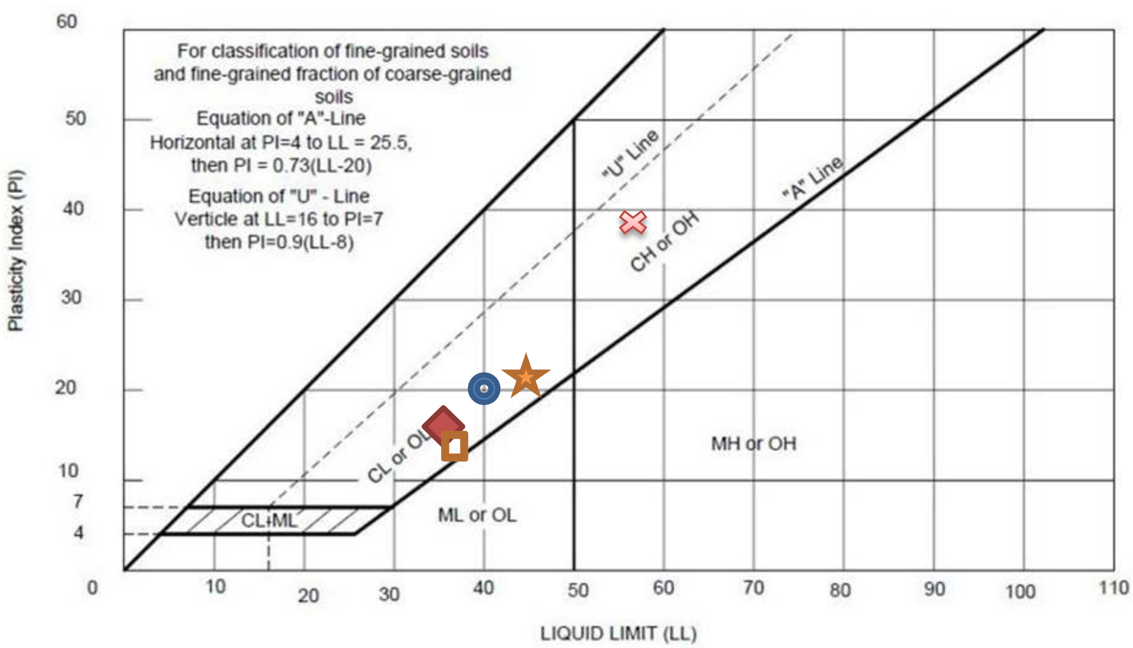
SOILS, FOUNDATIONS, DRAINAGE, LANDSLIDES, CIVIL & GEOTECHNICAL ENGINEERS

FIGURE No. **A-1**

GRAIN SIZE ANALYSIS RESULTS



PLASTICITY ANALYSIS RESULTS



	BORING/ SAMPLE NO.	DEPTH (FT)	LL	PL	PI	PASS # 200, %	SOIL DESCRIPTION
★	1-2	5	45	24	21	-	Lean CLAY, yellow brown (CL)
◆	1-5	20	35	20	15	-	Sandy Lean CLAY, yellow/olive (CL)
▲	1-8	35	-	-	-	13	Silty SAND, poorly graded (medium), grey (SP-SM)
◎	2-0	½	41	21	20	66	Lean CLAY, very sandy, dark brown (CL-SC)
▢	2-2	9	37	24	13	78	Lean CLAY w/ fine sand, dark olive brown (CL)
✕	3-1	5	59	21	38	-	Fat CLAY w/ fine sand, yellow brown (CH)

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GRAIN SIZE AND PLASTICITY ANALYSIS RESULTS

Soils, Foundations, Drainage, Landslides, Earth Retention, Geotechnical and Foundation Engineering,

File No ***7326***

Figure No. *A-3*

APPENDIX B

BORING LOGS

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BORING LOG

SB-1

Project No. **7326**	Date Drilled **7/6/2016**	Drilling Company	**Hillside Geotechnical Drilling**
Client	Providence Development Corp	Drill Rig Model	Mobile B-24 Driller John C, Tony
Site address	**McKissick Street, Pleasant Hill**	Drilling Method/ Dia.	4" Continuous Flight Auger
		Sampling Method	Calif 2.5" ID, SPT
Boring Location:	Center of southeast lot	Start Drilling @	8:30 am End Drilling@ 10:15 am
Surface Elevation 79	Datum JES Topo	water level	~ 18' / 21'
Logged By	JRS	Time/Date	Drill / 12:45pm

DEPTH FEET	SAMPLE #, TYPE	BLOWS/FT N*	USCS CLASS	DESCRIPTION	WATER DEPTH FEET
			CH	FAT CLAY, very stiff, dry(desiccated), gray brown	
1					1
@2'					
			ML/CL	SILT/Lean CLAY, very stiff/hard, dry, with caliche nodules to ¼", light yellow-brn	
@3½'	1-1 C-25	22		DD=103, w=16.2%, UCS=12,300 psf...........................PP > 5tsf	
	1-2			As above, more moist	
5	C-25	32		DD=101, w=23.0%,LL=45, PI=21, UCS=9,700 psf......PP > 5tsf	5
@8½'					
	1-3			As above, but moist	
10	C-25	39		DD=96, w=20.5%, ...PP > 5tsf	10
@13½					
	1-4		CL	Lean CLAY, w/ fine sand, stiff to very stiff, very moist, yellow brown	
15	C-25	24		DD=102, w=22.2 %, UCS=4,500 psf.............. ....PP 4¼ tsf	15
@18½					∇
	1-5			As above, wet	
20	C-25	18		DD=100, w=23.8 %, LL=35, PI=15, UCS=4,500 psf.........PP 2¼ tsf	20
				Continued on Sheet 2	∇

Note: Blowcounts are corrected for sampler dia., only

SAMPLER Type: SPT = 2" OD SPT; C20 = 2" ID California, C25 = 2½ " ID California, ST = Shelby, P = Pitcher, PP=Pocket Penetrometer—tsf.
LABORATORY TESTS: DD = dry density, (dry unit weight) pcf; w = water content, %; LL= liquid limit, PI = plasticity index; EI =Expansion Index
UC=Unconfined strength, psf. DS= strength by direct shear, TS= strength by triaxial shear Φ= friction angle °, c=cohesion intercept, psf.

THE SUTTON GROUP Geotechnical Engineers 3708 Mt. Diablo Blvd, Suite 215 Lafayette, CA, 94549 (925) 284-4208	Project No. 7326 Project Name McKissick Street	BORING LOG SB-1 Sheet 2 of 2

DEPTH FEET	SAMPLE #, TYPE	BLOWS/FT N*	USCS CLASS	DESCRIPTION	WATER DEPTH FEET
				Continued	∇
@23½					
25	1-6 C-25	16		As above, wet DD=99, w=24.8 %, LL=35, PI=15, UCS=3,400 psf.........PP 2¾ tsf	
@28½					
30	1-7 SPT	19	CL	Lean CLAY, very sandy, stiff, w/ ½" thick sand layers @ 6" apart w=25.3 % Driller: Layers of sand and clay to deeper than 32 feet.	
			SP-SM	Silty SAND, poorly graded (medium), medium dense, wet, grey W= 15.9%, 13% < #200	
@33½					
35	1-8 SPT	22	CL	Lean CLAY, very stiff/hard, very moist, yellow brown W= 24.8%,	
@38½			CL	Lean CLAY, stiff, wet, red brown w/ caliche zones	
40	1-9 SPT	27		W= 31.6%,	
				Boring terminated at 40 ft depth Tremie grouted with cement	
45					

SAMPLER Type: S = 2" OD SPT; C20 = 2" ID California, C25 = 2½ " ID California, ST = Shelby, P = Pitcher Sample

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BORING LOG

SB-3

Sheet 1 of 1

Project No. **7326**	Date Drilled **7/6/2016**	Drilling Company	**Hillside Geotechnical Drilling**
Client	Providence Development Corp	Drill Rig Model	Mobile B-24 Driller John C, Tony
Site address	**McKissick Street, Pleasant Hill**	Drilling Method/ Dia.	4" Continuous Flight Auger
		Sampling Method	Calif 2.5" ID, SPT
Boring Location:	Center of NW lot	Start Drilling @	12:25 End Drilling@ 12:45
Surface Elevation 85	Datum JES Topo	water level	Dry
Logged By	JRS	Time/Date	Drill

DEPTH FEET	SAMPLE #, TYPE	BLOWS/FT N*	USCS CLASS	DESCRIPTION	WATER DEPTH FEET
1			CL	Very Sandy Lean CLAY, very stiff, dry, dark gray brown. DISTURBED GROUND, Possibly FILL to about 2 ft. depth.	1
@3½'			CH	Fat CLAY w/ fine sand; very stiff, desiccated, dry, yellow brown	
5	3-1 C-25	28		DD=107, w=19.5%, LL=59. PI=38.. < #200=82%...PP > 5tsf	5
			CL/ML	Lean CLAY, very stiff to hard, dry, olive brown	
@8½'					
10	3-2 C-25	42		As above, but w/ caliche to ½" thick	10
@13½					
15	3-3 SPT	32		As above, w/ caliche W= 17.2%	15
				Boring terminated at 15 ft depth Tremie grouted with cement	
20					20

Note: Blowcounts are corrected for sampler dia., only

SAMPLER Type: SPT = 2" OD SPT; C20 = 2" ID California, C25 = 2½ " ID California, ST = Shelby, P = Pitcher, PP=Pocket Penetrometer—tsf.
LABORATORY TESTS: DD = dry density, (dry unit weight) pcf; w = water content, %; LL= liquid limit, PI = plasticity index; EI =Expansion Index
UC=Unconfined strength, psf. DS= strength by direct shear, TS= strength by triaxial shear *Φ*= friction angle °, c=cohesion intercept, psf.

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BORING LOG

SB-3

Project No. **7326**	Date Drilled **7/6/2016**	Drilling Company	**Hillside Geotechnical Drilling**
Client	Providence Development Corp	Drill Rig Model	Mobile B-24 Driller John C, Tony
Site address	**McKissick Street, Pleasant Hill**	Drilling Method/ Dia.	4" Continuous Flight Auger
		Sampling Method	Calif 2.5" ID, SPT
Boring Location:	Center of NW lot	Start Drilling @	12:25 End Drilling@ 12:45
Surface Elevation 85	Datum JES Topo	water level	Dry
Logged By	JRS	Time/Date	Drill

DEPTH FEET	SAMPLE #, TYPE	BLOWS/FT N*	USCS CLASS	DESCRIPTION	WATER DEPTH FEET
1			CL	Very Sandy Lean CLAY, very stiff, dry, dark gray brown. DISTURBED GROUND, Possibly FILL to about 2 ft. depth.	1
@3½'			CH	Fat CLAY w/ fine sand; very stiff, desiccated, dry, yellow brown	
5	3-1 C-25	28		DD=107, w=19.5%, LL=59, PI=38.. < #200=82%...PP > 5tsf	5
			CL/ML	Lean CLAY, very stiff to hard, dry, olive brown	
@8½'					
10	3-2 C-25	42		As above, but w/ caliche to ½" thick	10
@13½					
15	3-3 SPT	32		As above, w/ caliche W= 17.2%	15
				Boring terminated at 15 ft depth Tremie grouted with cement	
20					20

Note: Blowcounts are corrected for sampler dia., only

SAMPLER Type: SPT = 2" OD SPT; C20 = 2" ID California, C25 = 2½ " ID California, ST = Shelby, P = Pitcher, PP=Pocket Penetrometer—tsf.
LABORATORY TESTS: DD = dry density, (dry unit weight) pcf; w = water content, %; LL= liquid limit, PI = plasticity index; EI =Expansion Index
UC=Unconfined strength, psf. DS= strength by direct shear, TS= strength by triaxial shear *Φ*= friction angle °, c=cohesion intercept, psf.

BORING LOG

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SB-4

Project No. **7326**	Date Drilled **7/6/2016**	Drilling Company	**Hillside Geotechnical Drilling**
Client	Providence Development Corp	Drill Rig Model	Mobile B-24 Driller John C, Tony
Site address	**McKissick Street, Pleasant Hill**	Drilling Method/ Dia.	4” Continuous Flight Auger
		Sampling Method	Calif 2.5” ID, SPT
Boring Location:	Southeastern lot	Start Drilling @	11:30 am End Drilling@ 12:15pm
Surface Elevation 77	Datum JES Topo	water level	Dry / Dry
Logged By	JRS	Time/Date	Drill / 12:45pm

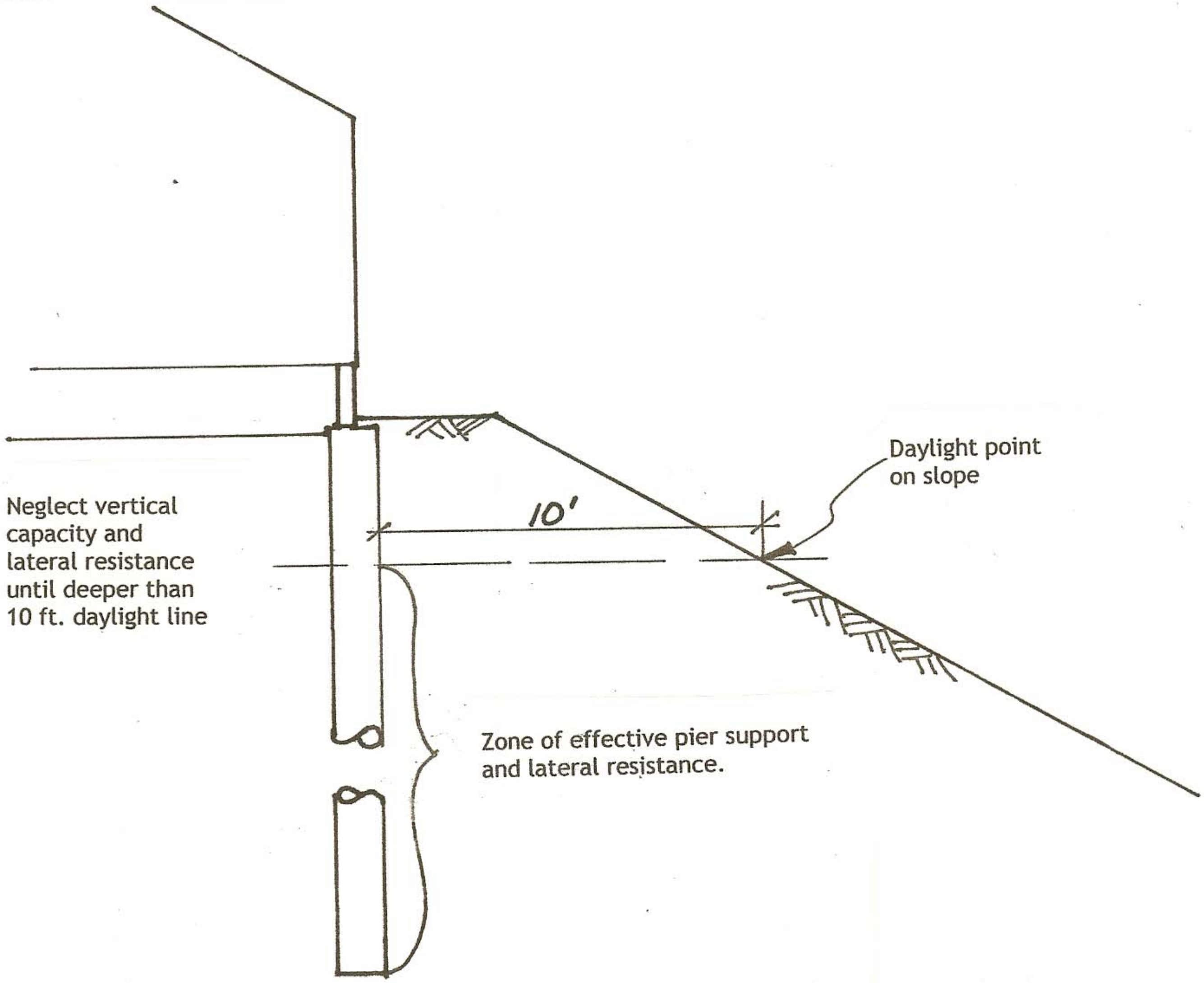
DEPTH FEET	SAMPLE #, TYPE	BLOWS/FT N*	USCS CLASS	DESCRIPTION	WATER DEPTH FEET
1			CL	Very Sandy Lean CLAY, very stiff, dry, w/ ½” desiccation cracks, dark gray brown	1
@3½			ML, CL	Interbedded lean CLAY (dark brown) and Clay/SILT (yellow brown)	
5	4-1 C-25	50/10”	CL	Lean CLAY, hard, moist, yellow brown, w/ ¼” caliche nodules DD=104, w=19.5%, UCS=9,600 psf..............................PP = 5 tsf	5
@8½’					
10	4-2 C-25	50/8”		As above but olive brown w/ caliche nodules to ¾” DD=102, w=19.8%, ...PP > 5tsf	10
@13½					
15	4-3 SPT	28	CL	As above W=17.7%	15
@18½					
20	4-4 SPT	20		As above, but very moist W= 23.4%	20
				Boring terminated at 20 ft depth Tremie grouted with cement	

Note: Blowcounts are corrected for sampler dia., only

SAMPLER Type: SPT = 2” OD SPT; C20 = 2” ID California, C25 = 2½ “ ID California, ST = Shelby, P = Pitcher, PP=Pocket Penetrometer—tsf.
LABORATORY TESTS: DD = dry density, (dry unit weight) pcf; w = water content, %; LL= liquid limit, PI = plasticity index; EI =Expansion Index
UC=Unconfined strength, psf. DS= strength by direct shear, TS= strength by triaxial shear Φ= friction angle °, c=cohesion intercept, psf.

APPENDIX C

CONSTRUCTION DETAILS



Schematic - NOT TO SCALE

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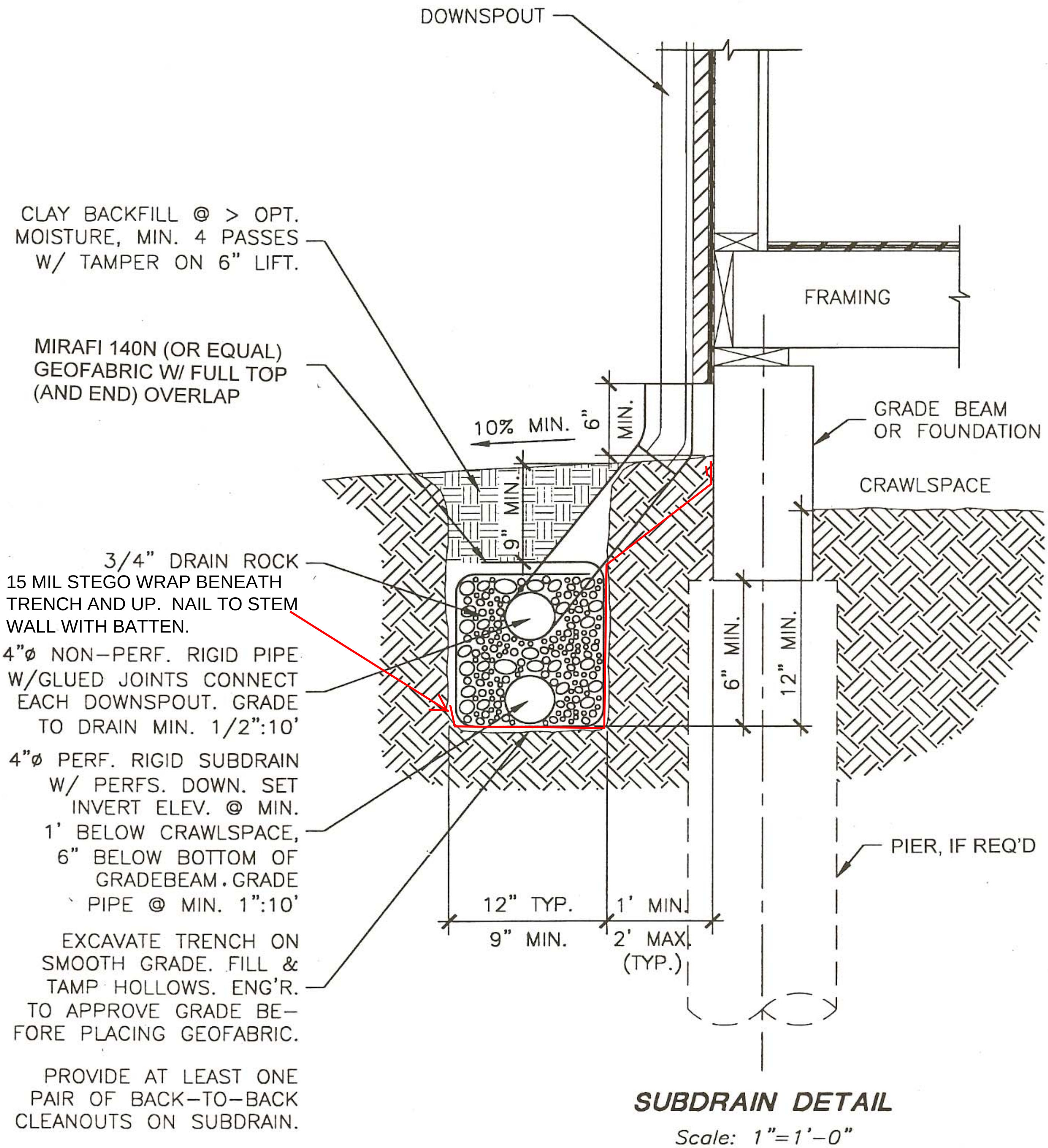
PIER DETAILS

FIGURE NO. **C-1**

Fig C1.doc

APPENDIX D

TYPICAL DRAINAGE DETAILS



SUBDRAIN DETAIL

Scale: 1"=1'-0"

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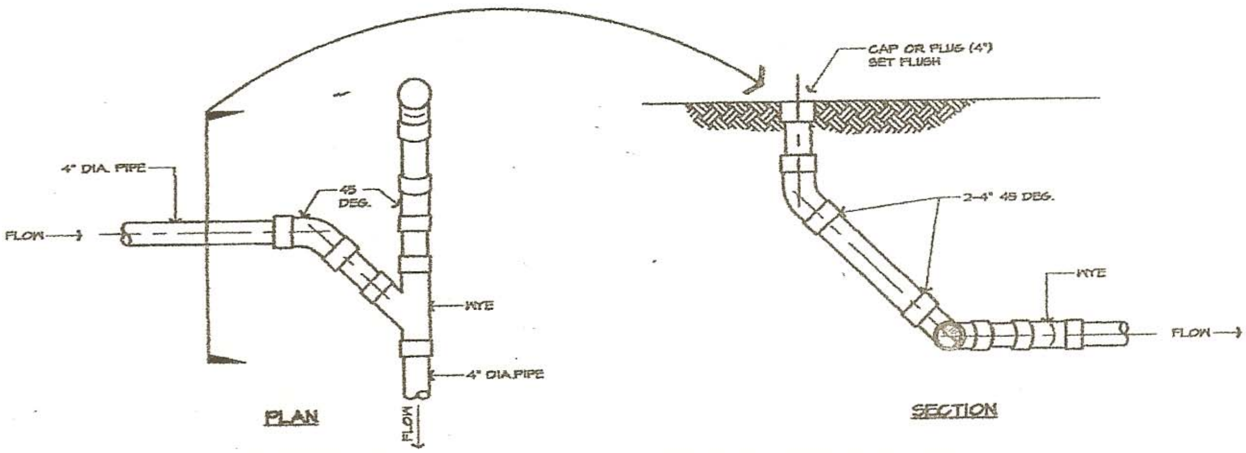
SUBDRAIN DETAILS
NEW CONSTRUCTION

PROJECT No.

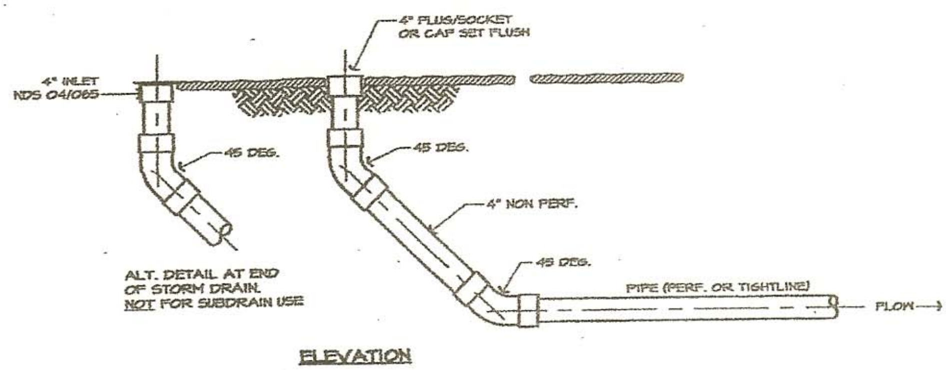
PLATE
D-1

DET-1.DWG



TYPICAL 90° BEND DETAIL ON 4" DIA. DRAIN

NOTE:
ALL PIPE TO BE BELL & SPIGOT



TYPICAL CLEANOUT DETAILS

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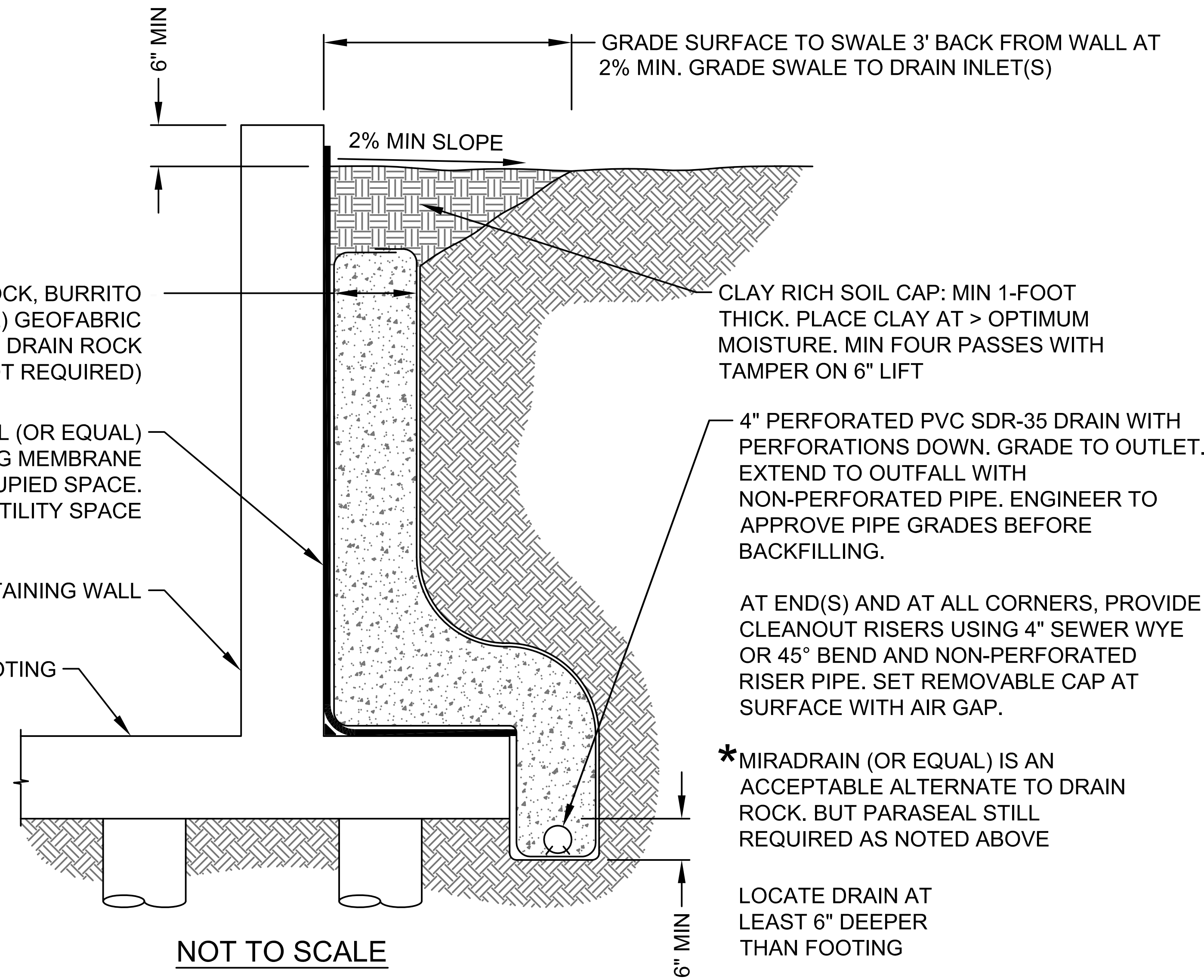
TYPICAL DRAIN DETAILS

PROJECT No.

D-2

10/20/03



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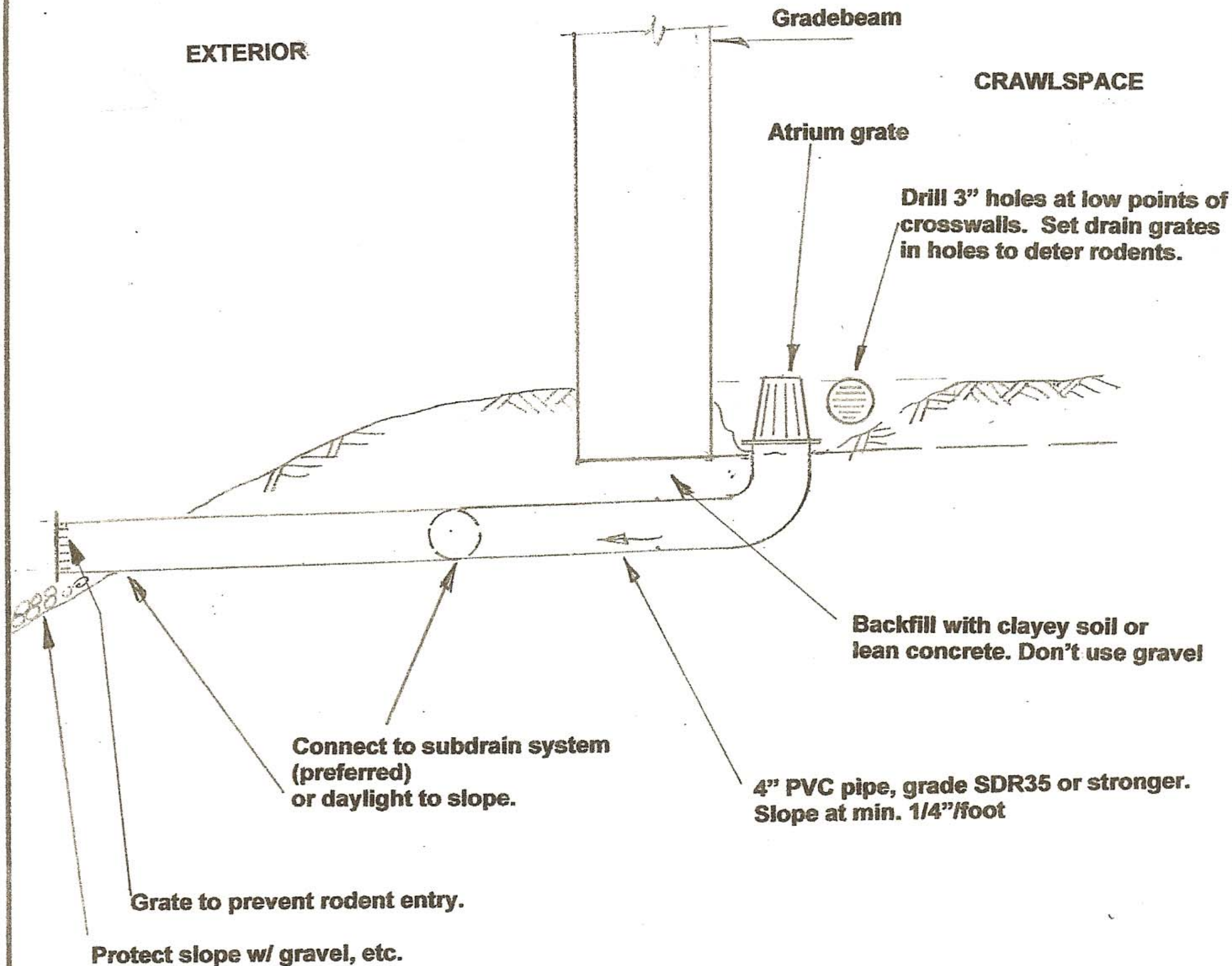
RETAINING WALL BACKDRAIN PIER-FOUNDED WALL

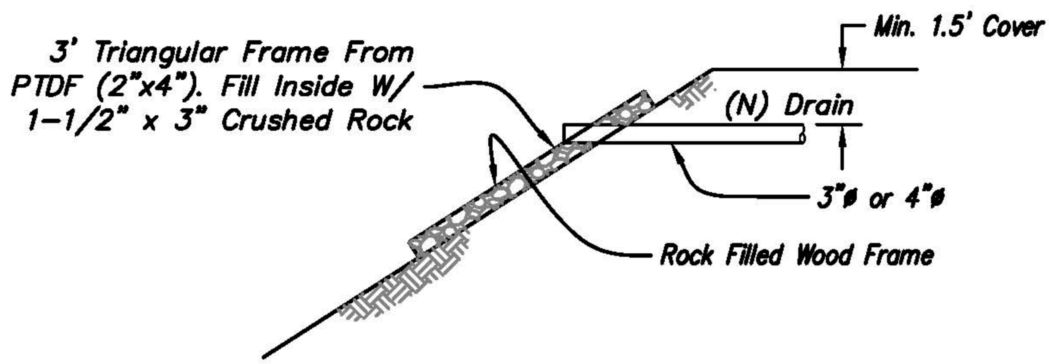
Project No.

Drn: 10/14/15

Figure No. **D-3A**



THE SUTTON GROUP *3708 Mount Diablo Blvd, Suite 215* *Lafayette, CA, 94549* *phone (925) 284-4208*	**CRAWL SPACE DRAIN DETAILS**	PROJECT *7086* FIGURE NO. **D-4**



DETAIL 1 - TYPICAL DRAIN OUTLET/ ENERGY DISIPATOR ON SLOPE

N.T.S.

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DRAINAGE OUTLET ENERGY DISSIPATOR DETAIL

FIGURE NO. **D5**

Geotechnical Investigation Report
for Four New Homes
Off McKissick Street,
Pleasant Hill, California

Back Page of Report