GEOTECHNICAL EXPLORATION REPORT
on
PINOLE SHORES COMMERCIAL DEVELOPMENT
at
836 San Pablo Ave.
Pinole, California
for
KIEWIT INFRASTRUCTURE WEST CO.

GEOTECHNICAL ENGINEERING CONSULTANTS

KC

By

KC ENGINEERING COMPANY

Project No. VV3967

4 October 2016

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Project No. VV3967
4 October 2016

Mr. Mark Bertolero
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Fairfield, CA 94534

Subject: Pinole Shores Commercial Development
836 San Pablo Avenue
Pinole, California
GEOTECHNICAL EXPLORATION REPORT

Dear Mr. Bertolero:

In accordance with your authorization, **KC ENGINEERING COMPANY** has explored the geotechnical conditions of the surface and subsurface soils of the undeveloped northern area of the Pinole Shores commercial development at the subject site. It is noted that the site is proposed to be filled with import soils from the Pinole-Hercules Water Pollution Control Plant Upgrade project and rough graded to provide two super pads for future commercial development.

The accompanying report presents our conclusions and recommendations based on our exploration. Our findings indicate that the proposed fill site and future commercial development is geotechnically feasible for construction on the subject site provided the recommendations of this report are carefully followed and are incorporated into the project plans and specifications.

Should you have any questions relating to the contents of this report or should you require additional information, please contact our office at your convenience. Should additional criteria or analysis be required, please call or email.

Reviewed By,

Andrew L. King, P.E.
Principal Engineer

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GEOTECHNICAL EXPLORATION

Purpose and Scope

The purpose of the geotechnical exploration for the proposed fill site and future Pinole Shores commercial development in Pinole, California, was to determine the surface and subsurface soil conditions at the subject site. Based on the results of the exploration, geotechnical criteria and recommendations were established for the grading of the site, the design of foundations, slabs, retaining walls, pavement sections and the construction of other related facilities on the property.

In accordance with your authorization, our exploration services included the following tasks:

a. A review of available geotechnical and geologic literature concerning the site and vicinity;
b. Site reconnaissance by the Geotechnical Engineer to observe and map surface conditions;
c. Drilling of a total of 7 exploratory borings, excavating 6 test pits, and sampling of the subsurface soils;
d. Laboratory testing of the samples obtained to determine their classification and engineering characteristics;
e. Analysis of the data and formulation of conclusions and recommendations; and
f. Preparation of this written report.

Site Location and Description

The subject site is a located at 836 San Pablo Avenue in the City of Pinole, California as shown on Figure 1 "Aerial Vicinity Map" included in the Appendix of this report. The site consists of the undeveloped northern portion of the Pinole Shores commercial development. We understand that the commercial developer Panattoni graded the Pinole Shores project in 2006/2007 and completed the southern half of the project with the four buildings and surrounding underground utilities and pavement that currently exist. Sanitary sewer, storm drain pipelines, catch basins, and a storm water filtration system were installed around the perimeter of the subject site as shown on the attached "Operations and Maintenance Plan". Currently the northern undeveloped area consists of two rough graded pad areas separated by an approximate 12 to 15 feet high 2H:1V slope. Stockpiled materials are present on the southern upper pad. The lower northern pad is relatively flat, but improved with storm water and bio-retention ponds as shown on the Operations and Maintenance Plan. A storm drain over-flow and pipe are located in the northeastern portion of the site. A rough graded access road surrounds the site. Vegetation on the site consist of a combination of overgrown brush, weeds and native grass.

The above description is based on a reconnaissance of the site by the Geotechnical Engineer, a review of a Google aerial image dated 2016, the “Proposed Master Site Plan” by RMW Architecture dated 3/1/06, and the “Operations and Maintenance Plan” by AnWest dated 9/7/07. The Google Aerial image was used as the basis for our “Aerial Vicinity Map” and the Proposed Master Site Plan was used for our “Site Plan” included as Figures 1 and 2, respectively in the Appendix of this report.

Proposed Development

We understand that the subject site will be used as a fill disposal site for the excess spoils generated at the Pinole-Hercules WPCP upgrade project. The site is currently owned by the City of Pinole and is planned to be sold for future commercial development. The existing site is planned to be stripped of vegetation and prepared for the import materials which will be placed as well-compacted engineered fill. Two super pads are planned to be graded by Kiewit for future commercial development. The existing 12 to 15 feet high slope is planned to be extended north at roughly the same upper pad elevation, followed by placing 4 to 6 vertical feet of fill on the northern lower pad. The surface of the pads may be lime treated and erosion protection provided. All of the proposed grading by Kiewit will occur within the limits of the perimeter rough graded access road.

After rough grading and lime treating of the two super pads as described above, we anticipate that future commercial development will occur including modification of the original development plan as shown on Figure 2 “Site Plan”, and include completion of underground utilities, pavements, retaining walls and commercial tilt-up buildings similar to the existing development.

Field Exploration

The field exploration was performed on 9/7/16 and included a reconnaissance of the site and the drilling of seven exploratory test borings and six test pits at the approximate locations shown on Figure 2, “Site Plan” included in the Appendix.

The borings were drilled to a maximum depth of 29½ feet below the existing ground surface. The drilling was performed with a CME 55 drill rig using power-driven, five-inch diameter, continuous flight augers. Visual classifications were made from auger cuttings and the samples in the field. As the drilling proceeded, relatively undisturbed tube samples were obtained by driving a 3-inch O.D., California split-tube sampler, containing thin brass liners, into the boring bottom in accordance with ASTM D3550. Disturbed samples were also obtained by driving a 2-inch O.D., split-barrel SPT sampler into the boring bottom in accordance with ASTM D1586. The samplers were driven into the in-situ soils at various depths under the impact of a 140-pound hammer having a free fall of 30 inches. The number of blows required to advance the sampler 12 inches into the soil, after seating the sampler 6 inches, were adjusted to the standard penetration resistance (N-Value). The raw blow counts obtained using the California sampler were corrected to equivalent N-Values using Burmister's (1948) energy and diameter correction formula. When the sampler was withdrawn from the boring bottom, the samples were removed, examined for identification purposes, labeled and sealed to preserve the in-situ moisture content, and transported to our laboratory for testing.

The test pits were excavated with a mini-excavator utilizing a 24 inch wide bucket. The encountered soil layers and material types were logged and bulk samples obtained. At completion the pits were loosely backfilled. These pits should be over-excavated and properly backfilled during the upcoming grading operations.

Classifications made in the field were verified in the laboratory after further examination and testing. The stratification of the soils, descriptions, location of undisturbed soil samples and standard penetration resistance are shown on the respective Boring and Test Pit Logs contained within the Appendix.

Laboratory Testing

The laboratory testing program was directed towards providing sufficient information for the determination of the engineering characteristics of the site soils so that the recommendations outlined in this report could be formulated. Lab classification testing of the proposed import soils from the Pinole WPCP are also included herein. The laboratory test results from the site are presented on the respective Boring and Test Pit Logs and data sheets in the Appendix.

Moisture content and dry density tests (ASTM D2937) were performed on representative relatively undisturbed soil samples in order to determine the consistency of the soil and the moisture variation throughout the explored soil profile as well as estimate the compressibility of the underlying soils.

The strength parameters of the foundation soils were determined from a direct shear test (ASTM D3080) and unconfined compression tests (ATSTM D2166) performed on selected relatively undisturbed soil samples. Standard field penetration resistance (N-Values) and pocket penetrometer readings also assisted in the determination of strength and bearing capacity. The standard penetration resistances and pocket penetrometer readings are recorded on the respective "Log of Test Boring".

In order to assist in the identification and classification of the subsurface soils, sieve analysis tests (ASTM D6913), Atterberg Limits tests (ASTM D4318), and an Expansion Index test (ASTM D4829) were performed on selected soil samples. The Atterberg Limits and Expansion Index test results were used to estimate the expansion potential of the site and import soils.

An R-Value test (Cal Test 301) was performed on a composite bulk sample to assist in pavement section design. The bulk sample was obtained from the upper 3 feet across the site pavement areas at the locations shown on Figure 2.

A representative composite bulk sample of the near surface soils was obtained to evaluate the presence and concentration of water soluble sulfates in accordance with California Test Method 417. These test results were used to identify the corrosion potential of the soils to at or below grade concrete. Additional soil corrosion potential tests (pH, Resistivity & Chlorides) were also performed. The bulk sample was obtained from the upper 5 feet across the building pad areas at the locations shown on Figure 2.

Subsurface Conditions

Based on our field exploration and laboratory testing, the surface and subsurface soil conditions across the site generally consist of variable thicknesses of undocumented fills overlying native soil deposits and highly weathered bedrock. The undocumented surficial fills consist of highly to very highly expansive, sandy clay with claystone diatomite rock fragments ranging in thickness from 2 to 25 feet. The thickest fill encountered is at the northeastern portion of the site. Although no compaction test records were available for our review, the fills appeared to have been placed and well compacted. The fills are underlain by native soils consisting of highly expansive sandy clays. The underlying bedrock consists highly weathered and fractured, weak bedrock. Claystone, siltstone, sandstone and diatomite bedrock materials were encountered.

The existing upper pad appears to have been created by placing 2 to 14 feet of fill over native soils and bedrock, with the thickest fill on the west. The lower pad appears to have been graded by cutting the central area and lower portion of the bisecting slope. The west, north and east areas were filled as evidenced by the perimeter 2H:1V fill slope.

Groundwater was encountered in Boring 3 at 15 feet and in Boring 4 at 24 feet at the time of our field exploration. Groundwater was not encountered in the remaining borings and test pits.

Fluctuations in the groundwater conditions can occur with variations in seasonal rainfall, site irrigation and variations in subsurface stratigraphy.

A more thorough description and stratification of the soils encountered along with the results of the laboratory tests are presented on the respective Boring and Test Pit Logs in the Appendix. The approximate locations of the borings and test pits are shown on Figure 2, “Site Plan,” in the Appendix.

Soil Corrosivity

A representative composite sample of the near surface soils (upper 5 feet) was collected and transported to Sunland Analytical in Rancho Cordova for testing of water soluble sulfates, pH, minimum resistivity and chlorides per California Test Methods.

The testing indicates sulfate contents of 186 ppm (mg/kg), a chloride content of 56 ppm, minimum resistivity of 960 ohm-cm and soil pH of 5.11 for the sample collected. It is noted that the sulfate test results indicate “not-applicable” or “S0” sulfate exposure to concrete as identified in Section 1904 of the 2013 California Building Code and Tables 4.2.1 and 4.3.1 of ACI 318-11 Building Code Requirements for Structural Concrete. No cement type restriction is required, however, we do recommend that a Type I/II cement be utilized for moderate sulfate resistance.

The Caltrans Corrosion Guidelines[1] defines a corrosive site as one where the soil and/or water has a sulfate concentration of 2,000 ppm or more, a chloride concentration of 500 ppm or more, a pH of 5.5 or less, and a minimum resistivity less than 1,000 ohm-cm. According to the Electrical Design, Cathodic Protection Manual[2], soil corrosion is not likely if the minimum resistivity is above 30,000 ohm-cm. If the resistivity is between 10,000 and 30,000 ohm-cm the corrosivity is mild, between 2,000 and 10,000 the corrosivity is moderate to severe, and lower than 2,000 it is severe. Based on these criteria, the soils at the site are considered to have a severe corrosion potential to buried metal.

KC Engineering Company is not a corrosion engineering firm. Therefore, to further define the soil corrosion potential and interpret the above test results, or to design cathodic protection or grounding systems, a licensed Corrosion Engineer should be consulted.

[1] California Department of Transportation Corrosion Technology Branch, Materials Engineering and Testing Services, *Corrosion Guidelines*, Version 2.0, November 2012.
[2] Technical Manual TM 5-811-7, *Electrical Design, Cathodic Protection*, prepared by Headquarters, Department of the Army, dated 22 April 1985 also known as UFC 3-570-02A, *Cathodic Protection*, by Department of Defense.

Site Geology

The geologic materials underlying the site are mapped as lower Cretaceous aged Rodeo shale and older diatomite and sandstone as shown on the Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County[3]. A portion of this map showing the site is presented on Figure 3 "Geologic Map" in the Appendix. The subsurface deposits encountered during our exploration generally correlate with previous mapping, except for the overlying native alluvium and artificial fills.

Geo-Hazards

Seismicity & Ground Motion Analysis

The site is not located within an Alquist-Priolo Earthquake Fault Zone[4]. There are no known active faults crossing the site as mapped and/or recognized by the State of California. Pinole is located in a seismically-active region and earthquake related ground shaking should be expected during the design life of structures constructed on the site. The California Geological Survey has defined an active fault as one that has had surface displacement in the last 11,000 years, or has experienced earthquakes in recorded history.

Based on our review of the Fault Activity Map of California[5] and the USGS Fault Database[6], the nearest active faults are the Pinole Fault and the Hayward-Rogers Creek Fault located approximately 0.6 miles northeast and 2.9 miles southwest, respectively.

The 2013 CBC specifies that the potential for liquefaction and soil strength loss should be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with an adjustment for site class effects in accordance with American Society of Civil Engineer (ASCE 7-10)[7]. The MCE_G is peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated

[3] Graymer, R.W., Jones, D.L., and Brabb, E.E., 1994, *Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California*, United States Geological Survey, OFR-94-622
[4] Hart, E.W. and Bryant, W.A., 1997, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps*, California Department of Conservation, Division of Mines and Geology, Special Publication 42, Interim Revision 2007.
[5] Jennings, C.W. and Bryant, W.A., 2010, *Fault Activity Map of California*, California Geological Survey Geologic Data Map No. 6, scale 1:750,000
[6] U.S. Geological Survey and California Geological Survey, 2006, Quaternary Fault and Fold Database for the United States, accessed 9/22/16, from USGS web site: http//earthquake.usgs.gov/regional/qfaults/.
[7] American Society of Civil Engineer (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures, Standard 7-10.

to be 0.725g using the United States Geological Survey web-based seismic design tool with a site coefficient (F_{PGA}) of 1.0 for Site Class D.

Structures at the site should be designed to withstand the anticipated ground accelerations. Based on the USGS Seismic Design Maps[8] website and ASCE 7-10, the 2013 CBC earthquake design values are as follows.

Site Class: D
Mapped Acceleration Parameters: S_S = 1.882g; S_1 = 0.757g
Design Spectral Response Accelerations: S_{DS} = 1.254g; S_{D1} = 0.757g

The summary and detailed USGS design maps reports are included in Appendix.

Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on our review of geologic maps, no known active or inactive faults cross or project toward the subject site. No evidence of active faulting was visible on the site during our site reconnaissance. Therefore, it is our opinion that there is no potential for fault-related surface rupture at the subject site.

Landsliding

The cut and fill slopes observed on the site do not have any obvious signs of landsliding or slumping. It is our opinion the potential for seismically-induced landslide hazards is low.

Liquefaction

Soil liquefaction is a phenomenon in which loose and saturated cohesionless soils are subject to a temporary, but essentially total loss of shear strength, due to pore pressure build-up under the reversing cyclic shear stresses associated with earthquakes. Soils typically found most susceptible to liquefaction are saturated and loose, fine to medium grained sand having a uniform particle range and less than 35% fines passing the No. 200 sieve, and a corrected standard penetration blow count $(N_1)_{60}$ less than 30. According to Special Publication 117A by the California Geological Society, the assessment of hazards associated with potential liquefaction of soil deposits at a site must consider translational site instability (i.e. lateral spreading, etc.) and more localized hazards such as bearing failure and settlement. The acceptable factor of safety against liquefaction is recommended in SP117 to be 1.3 or greater.

[8] http://earthquake.usgs.gov/designmaps/us/application.php, accessed 9/22/16

The data used for evaluating liquefaction potential of the subsurface soils consisted of the in-situ Standard Penetration Resistance values $(N_1)_{60}$ values, the unit weights, gradations, in-situ moisture contents, the groundwater level, and the location of the site to the nearest active fault and the predicted ground surface acceleration The soil materials encountered on the site were found to be stiff to very stiff and predominately cohesive. Considering the stiff cohesive nature of the surficial soils and the relatively shallow bedrock, it is our opinion that liquefaction of the subsurface deposits are considered unlikely.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

General

From a geotechnical point of view, the proposed fill site and future commercial development are considered to be feasible for construction provided the recommendations presented in this report are incorporated into the project plans and specifications.

All grading and foundation plans for the development must be reviewed by the Soil Engineer prior to contract bidding or submittal to governmental agencies to ensure that the geotechnical recommendations contained herein are properly incorporated and utilized in design.

KC ENGINEERING CO. should be notified at least two working days prior to site clearing, grading, and/or foundation operations on the property. This will give the Soil Engineer ample time to discuss the problems that may be encountered in the field and coordinate the work with the contractor.

Field observation and testing during the grading and/or foundation operations must be provided by representatives of ***KC ENGINEERING CO.*** to enable them to form an opinion regarding the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements.

Geotechnical Considerations

The primary geotechnical concerns for the site are the presence of surficial undocumented fills, the highly to very highly expansive materials, and the potential for differential settlement from the variable thickness of fills under the proposed building footprints. Although compaction test records were not provided for our review, our field and lab data suggest that the fills were placed and relatively well-compacted. The upper 12 inches was found to be relatively dry due to the length of time since placement. In addition, the stockpiled soils on the upper pad were found to be generally clean of debris and will require over-excavating down to the previously compacted pad prior to use as engineered fill as described in the “Grading” section below.

The existing site soils and the proposed import soils from the Pinole WPCP are considered highly expansive and are prone to excessive heave and shrink movements with changes in moisture content and, consequently, must be carefully considered in the design of grading, foundations, drainage, and landscaping. Due to the expansive nature of these materials and expected variability in the proposed import material type, we recommend that the building pads and pavement areas be lime treated as recommended in the “Grading” and “Pavement” sections below.

Due to the variable fill thicknesses that will be present under the proposed buildings, we estimate that differential settlement of 1 to 2 inches may occur across the building footprints. To aid in dampening and minimizing potential differential settlement, we recommend that additional fills be compacted to a higher degree of relative compaction and that the upper 2 feet of the pads be lime modified.

Considering that all of the existing stockpiled materials and proposed import materials will be placed as compacted engineered fill with the surface of the pads lime treated, it is the opinion of **KC ENGINEERING COMPANY** that the proposed commercial structures may be supported on a deepened and inter-connected, well-reinforced spread footing foundation system with conventional interior slab-on-grade floors.

Grading, foundation design, and drainage recommendations are presented herein. At this time, final project or detailed grading plans have not been completed. When plans become available additional exploratory borings may be required. When project grading plans become available for our review, supplemental grading recommendations may be required.

Grading

Grading operations performed during the wet season will be hampered by excessive moisture. Grading activities may be performed during the wet season. However, achieving proper compaction may be difficult due to excessive moisture resulting in project delays to grade the site. Grading performed during the dry months will minimize the occurrence of the above problems.

The surface of the site should be stripped to remove all existing heavy brush and vegetation and/or other deleterious materials. The thick brush and dense vegetation should then be removed from the site. The remaining low native grasses may be disked and mixed into the proposed fills.

As discussed above, the existing stockpiled material on the upper pad, as well as the proposed import from the Pinole WPCP may be utilized as compacted engineered fill for the proposed commercial project. Prior to grading and additional fill placement, the stockpiled materials on the upper pad, as well as the storm water pond berms on the lower pad will need to be over-excavated to expose the original pad grade, followed by scarifying, moisture conditioning and compacting before additional fill is placed to design grades.

After stripping and removal of heavy vegetation, and over-excavation of the stockpiled materials and drainage berms, it is recommended the upper 12 inches of exposed surface (previously graded pad surface) be scarified, moisture conditioned and compacted to a minimum degree of relative compaction of 90% as determined by ASTM D1557 Laboratory Test Procedure. After processing the lower 12 inches, the site may be brought to the desired finished grades by placing engineered fill in lifts of 8 inches in un-compacted thickness and compacting to a minimum relative compaction of 92% at 3 or more percent over optimum in accordance with the aforementioned test procedure. All soils encountered during our investigation are suitable for use as engineered fill when placed and compacted at the recommended moisture content.

Any loose or soft soils encountered during original ground preparation must be over-excavated to competent materials prior to fill placement. Excavated soil materials may be used as engineered fill with the approval of the Soil Engineer provided they do not contain debris or excessive organics.

All fill material should be approved by the Soil Engineer. The material should be a soil or soil-rock mixture which is free from excessive organic matter or other deleterious substances. The fill material should not contain rocks or lumps over 6 inches in greatest dimension and not more than 15% larger than 2-½ inches. All soils encountered during our exploration, including the proposed import from the WPCP would be suitable for use as engineered fill when placed and compacted as recommended.

Prior to compaction, each layer should be spread evenly and should be thoroughly blade mixed during the spreading to obtain uniformity of material in each layer. The fill should be brought to a water content that will permit proper compaction by either (a) aerating the material if it is too wet, or (b) spraying the material with water if it is too dry. Compaction should be performed by footed rollers or other types of approved compaction equipment and methods. Compaction equipment should be of such design that they will be able to compact the fill to the specified density. Rolling of each layer should be continuous over its entire area and the equipment should make sufficient trips to ensure that the required density has been obtained. No ponding or jetting is permitted.

Should select import material be used to establish the upper 24 inches of the structural fill pad, the import material should be approved by the Soil Engineer before it is brought to the site. The select fill limits should extend 5 feet beyond the building footprint or to the edge of adjacent flatwork. Where select import soil is used within the upper 24 inches of the pad, it should meet the following requirements:

a. Have an R-Value of not less than 25;

b. Have a Plasticity Index not higher than 15;
c. Not more than 15% passing the No. 200 sieve;
d. No rocks larger than 3 inches in maximum size;

As an alternate to importing select low to non-expansive materials for the upper 24 inches of the building pads, the materials should be lime treated. The lime treatment limits should extend 5 feet beyond the building footprint or to the edge of adjacent concrete flatwork. The lime treatment may be performed in maximum 18 inch lifts. Therefore, the building pad should be graded to within 18 inches of the design pad grade, followed by lime treatment. The lime treatment should consist of a mixture of 5% lime by dry weight of high-calcium quicklime, for the 18-inch mixing depth. Considering the anticipated variability of the site materials and a soil unit weight of 115 p.c.f., the minimum spread rate for an 18 inch mixing depth would be 8.6 p.s.f., or alternatively 5.8 p.s.f. for a 12 inch mixing depth. A lower lime percentage and spread rate may be utilized if determined by lab testing to verify a Plasticity Index of 15 or less. The lime treated soils should be compacted to at least 95% relative compaction of the maximum wet density at moisture content at least 3% above optimum. The lime treatment must be performed by a qualified soil stabilization contractor in general conformance with Caltrans Standard Specification Section 24. The product specification and quality control test results must be provided to us by the contractor for review and acceptance prior to the treatment operations. The lime should be spread and mixed with equipment capable of providing relatively uniform conditions. The lime treated sections must be mixed at least twice prior to compaction which must be performed within 24 hours after final mixing. After compaction, it is important to moist cure the lime treated soils until placement of the subsequent slab subbase materials (i.e. do not let pad dry out and desiccate).

The standard test used to define maximum densities and optimum moisture content of all compaction work shall be the Laboratory Test procedure ASTM D1557 and field tests shall be expressed as a relative compaction in terms of the maximum dry density and optimum moisture content obtained in the laboratory by the foregoing standard procedure. Field density and moisture tests shall be made in each compacted layer by the Soil Engineer of Record in accordance with Laboratory Test Procedure ASTM D6938. When footed rollers are used for compaction, the density and moisture tests shall be taken in the compacted material below the surface disturbed by the roller. When these tests indicate that the compaction requirements on any layer of fill, or portion thereof, have not been met, the particular layer, or portion thereof, shall be reworked until the compaction requirements have been met.

Slopes

Considering the sites sloping contours and the anticipated placement of compacted fill, engineering guidelines should be followed when placing fill on sloping terrain. We anticipate that the northern development areas may have additional fill slope requirements. Prior to placement of fills and after stripping of vegetation for the future development, a toe of slope keyway must be constructed into competent soil materials prior to placement of engineered fill as required by the 2013 CBC Appendix J. A toe key excavation should be placed at the base of all such fills. This key should be a minimum of 12 feet in width, cut into competent non-yielding material a minimum of 2 vertical feet, and sloped into the hillside at a gradient of no less than 5%. A typical fill slope cross section has been included in the Appendix. A keyway is not required for fill slopes if the existing ground is 6H:1V or flatter.

A subdrain should be constructed in the toe of slope keyway as shown on the attached cross-section. The subdrain should be connected to a suitable discharge facility. The actual location and quantity of subsurface drains will be made in the field by the Soil Engineer and after review of the project grading plans.

Unsupported cut slopes should not be steeper than 2.5H:1V. Fill slopes should not be steeper than 2H:1V. Fill slopes must be compacted as the filling operation progresses by over-constructing the fill slopes and cutting back the looser surface soils to a firm and adequately compacted designed slope grade. Track-walking of slope surfaces does not provide adequate soil densities and is an unacceptable method of slope compaction.

Cut and fill slopes in soil may experience severe erosion when grading is halted during rainy weather. Before work is stopped, a positive gradient away from the slopes must be established to carry the surface runoff water away from the slopes to areas where erosion and sediment can be controlled. After the completion of the slope grading, erosion protection must be provided on all soil surfaces. Slope planting, preferably with deep-rooted native plants, must be completed on all exposed surfaces of cut and fill slopes. Graded slopes should not be left exposed through a winter season without the completion of erosion control measures and slope planting.

Surface Drainage

A very important factor affecting the performance of structures is the proper design, implementation, and maintenance of surface drainage, as well as maintaining uniform moisture conditions around the structures. Ponded water will cause swelling and/or loss of soil strength and may also seep under structures. Should surface water be allowed to seep under the structures, differential foundation movement resulting in structural damage and/or standing water under the slab will occur. This may cause dampness to the floor which may result in mildew, staining, and/or warping of floor coverings. To minimize the potential for the above problems, dampproofing and waterproofing should be provided as required by Section 1805 of the 2013 CBC. In addition, the following surface drainage measures are recommended and must be maintained by the property owner in perpetuity:

a) Liberal building pad slopes and surface drainage must be provided by the project Civil Engineer to remove all storm water from the pad and to prevent storm and/or irrigation water from ponding adjacent to the structure foundations. The finished pad grade around the structures should be compacted and sloped 5% away from the exterior foundations and as required in Section 1804.3 of the 2013 CBC.

b) Enclosed or trapped planter areas adjacent to the structure foundations should be avoided if possible. Where enclosed planter areas are constructed, these areas must be provided with adequate measures to drain surface water (irrigation and rainfall) away from the foundation. Positive surface gradients and/or controlled drainage area inlets should be provided. Care should be taken to adequately slope surface grades away from the structure foundations and into area inlets. Drainage area inlets should be piped to a suitable discharge facility.

c) Adequate measures for storm water discharge from the roof gutter downspouts must be provided by the project Civil Engineer and maintained by the property owners at all times, such that no water is allowed to pond next to the structure. Closed pipe discharge lines should be connected to downspouts and discharged into a suitable drainage facility. It is important not to allow concentrated discharge on the surface of any slope so as to prevent erosion.

d) Site drainage should be designed by the project Civil Engineer. Civil engineering, hydraulic engineering, and surveying expertise is necessary to design proper surface drainage to assure that the flow of water is directed away from the foundations.

e) Over-irrigation of plants is a common source of water migrating beneath a structure. Consequently, the amount of irrigation should not be any more than the amount necessary to support growth of the plants. Foliage requiring little irrigation (drip system) is recommended for the areas immediately adjacent to the structures.

f) Landscape mounds or concrete flatwork should not be constructed to block or obstruct the surface drainage paths. The Landscape Architect or other landscaper should be made aware of these landscaping recommendations and should implement them as designed. The surface drainage facilities should be constructed by the contractor as designed by the Civil Engineer.

Where bio-retention swales, basins or planter areas be planned by the Civil Engineer, we recommend that these areas be reviewed by the Soil engineer for comment and supplemental recommendations. Bottom subdrainage, positive slope gradients and/or water-proof liners may be required. In addition, we recommend that bio-swales and basins be a minimum of 10 feet away from building structures.

Foundations

Based on the results of the field exploration and laboratory testing, the site foundation soils are considered to be highly expansive. Provided the site is prepared in accordance with the "Grading" section above and the building pads consist of either select import or lime treated materials, the proposed structures may be supported on a well-reinforced inter-connected spread-footing foundation system.

A continuous spread footing should be placed around the perimeter of the structure and any interior wall or column footings should be continuously connected to the perimeter. Isolated footings should not be utilized unless connected with reinforced tie-beams. The continuous spread footings should extend to a minimum depth of 24 inches below lowest adjacent pad grade (i.e., trenching depth) and be a minimum of 18 inches wide. Tie-beams should be a minimum of 18 inches deep. At this depth, the recommended design bearing pressure for the continuous footings should not exceed 2,000 p.s.f. due to dead plus live loads. The above allowable pressures may be increased by 1/3 due to transient loads which include wind and seismic. All foundations must be adequately reinforced to provide structural continuity and resist the anticipated loads as determined by the project Structural Engineer. However, continuous footings and tie-beams are recommended to be reinforced with a minimum of four No. 6 bars, two at the top and two near the bottom of the footing. Additional reinforcement will be as required by the structural engineer and in accordance with structural building code requirements. Foundations designed in accordance with the above criteria are expected to experience a total settlement of less than 1 inch with less than 1/2 of an inch of differential settlement between columns.

To accommodate lateral building loads, the passive resistance of the foundation soil can be utilized. The passive soil pressures can be assumed to act against the front face of the footing below a depth of 1 foot below the ground surface. It is recommended that a passive pressure equivalent to that of a fluid weighing 250 p.c.f. be used. For design purposes, an allowable friction coefficient of 0.32 can be assumed at the base of the spread footings. These two modes of resistance should not be added unless the frictional component is reduced by 50 percent since the mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance.

Slab-on-Grade Construction

Interior and exterior concrete slabs/flatwork, including sidewalks, driveways and non-structural detached patios and general flatwork may experience some cracking due to finishing and curing methods as well as moisture variations within the underlying soils. To reduce the potential cracking of the slabs-on-grade, the following recommendations are made:

a) It is important to moist cure the lime treated soils on the building pad until placement of the subsequent materials. All areas to receive slabs should be thoroughly wetted to seal any desiccation cracks prior to placing concrete. This work should be done under the observation of the Soil Engineer.

b) Slabs should be underlain by a minimum of 4 inches of compacted angular Caltrans Class 2 Aggregate Base placed between the finished subgrade and the slabs to serve as slab support and a capillary break between the subsoil and the slab.

c) Interior and exterior slabs/flatwork should be a minimum of 5 inches thick and reinforced with a minimum of No. 4 rebar spaced 18 inches center to center, each way. Thicker slabs may be required in warehouse spaces depending on intended use. Additional concrete pavement recommendations are provided in the "Pavement Areas" section of this report. The actual slab thickness and reinforcement should be determined by the project structural engineer in accordance with the structural requirements and the anticipated loading conditions. The reinforcement shall be placed in the center of the slab unless otherwise designated by the design engineer.

d) A vapor retarder membrane should be installed between the prepared aggregate base of the building pad and the interior slabs to minimize moisture condensation under the floor coverings and/or upward vapor transmission. The vapor barrier membrane should be a minimum 15-mil extruded polyolefin plastic that complies with ASTM E1745 Class A and have a permeance of less than 0.01 perms per ASTM E96 or ASTM F1249. It is noted that polyethylene films (visqueen) do not meet these specifications. The vapor barrier must be adequately lapped and taped/sealed at penetrations and seems in accordance with ASTM E1643 and the manufacturer's specifications. The vapor retarder must be placed continuously across the slab area.

e) Exterior flatwork should be placed structurally independent of the foundations. A 30-pound felt strip, expansion joint material, or other positive separator should be provided around the edge of all floating slabs to prevent bonding to the foundation. As an added measure to minimize vertical deflections between the foundation and exterior slabs, rebar doweling should be provided. Doweling details should be provided by the Structural Engineer.

f) Slabs should be provided with crack control saw cut joints, tool joints or other methods to allow for expansion and contraction of the concrete. In general, contraction joints should be spaced no more than 20 times the slab thickness in each direction. The layout of the joints should be determined by the project Structural Engineer and/or Architect.

g) To minimize moisture infiltration under slabs and to add edge rigidity, we recommend that slabs be thickened at the edges to extend below the aggregate base layer to the soil subgrade for a minimum width of 6 inches.

h) We recommend that appropriate provisions be provided by the Structural Engineer and Contractor to minimize slab cracking, such as curing measures and/or admixtures to minimize concrete shrinkage and curling. American Concrete Institute methods and guidelines of curing, such as wet curing or membrane curing, are recommended to minimize drying shrinkage cracking.

Pavement Areas

The driveways and parking areas are anticipated to consist of either asphalt concrete (AC) or Portland cement concrete (PCC) surfaces. Recommendations for both pavement surfaces are presented below. We emphasize that the performance of the pavement is critically dependent upon adequate and uniform compaction of the subgrade soils, as well as engineered fill and utility trench backfill within the limits of pavements. Pavements will typically have poor performance and shorter life where water is allowed to migrate into the aggregate base and subgrade soils. The main source of water into a pavement section is landscape planters constructed within or adjacent to pavement areas. Where this is planned, it is recommended to extend the curbs into the soil subgrade at least 2 inches. The construction of all pavements should conform to the requirements set forth by the latest Standard Specifications of the Department of Transportation of the State of California (Caltrans) and the City of Pinole.

R-Value: A composite bulk sample was obtained of the near surface soils within the planned roadways that are relatively representative of the anticipated subgrade soils. The sample was tested in accordance with the California Test Method 301 to determine the R-Value for the site soils. An R-Value of 42 was determined for the sample obtained as shown in the Appendix. Due to the highly expansive nature of the on-site and import soils, we recommend that the upper 12 inches of the pavement subgrade areas be lime treated. For lime treated subgrades, we recommend a minimum R-Value of 25 be used for design.

Preparation of Subgrade: After underground utilities have been placed in the areas to receive pavement and removal of excess material has been completed, the upper 12 inches of the subgrade soil shall be lime treated with a 5% mixture and compacted to a minimum of 95%. The lime treatment and compaction criteria in the “Grading” section above should be followed. Prior to placement of aggregate baserock, it is recommended that the subgrade be proof rolled and observed for deflection by the Soils Engineer. Should deflection and/or pumping conditions be encountered, stabilization recommendations will be provided by the Geotechnical Engineer based on field conditions.

Aggregate Base: All aggregate base material placed subsequently should also be compacted to a minimum relative compaction of 95% based on the ASTM Test Procedure D1557. Aggregate base should meet the minimum requirements of Caltrans Class 2 per Section 26. The recommended aggregate base thicknesses for asphalt concrete pavements are noted in the table below. The minimum aggregate base thickness for PCC roadway pavements is 6 compacted inches.

Asphalt Concrete: Based on a lime treated subgrade with a minimum R-Value of 25 and traffic indices typical for commercial developments, the recommended pavement sections for asphalt concrete surfaces are summarized in the table below. The appropriate traffic index (TI) and any minimum pavement sections should be determined by the Civil Engineer in conformance with the City of Pinole Specifications.

Traffic Condition	**Traffic Index (TI)**	**Asphalt Concrete (inches)**	**Class II Aggregate Base[1] (inches)**
Parking Stalls	4.5	3.0	5.0
Drive Aisles	6.0	3.5	9.5
Heavy Truck Areas	8.0	4.5	13.0

NOTES:
(1) Minimum R-Value = 78 per Caltrans Section 26.
(2) All layers in compacted thickness to CalTrans Standard Specifications.

Portland Cement Concrete: Where PCC pavement areas are utilized, the concrete should be poured on the compacted aggregate base layer. The concrete section should be designed by the project Structural Engineer. We recommend a minimum of 7 inches thick PCC reinforced with a minimum of No. 4 rebar spaced at 16 inches on center, each way, underlain by 6 inches of compacted Class 2 aggregate base. Additional reinforcement may be required by the Structural Engineer.

Retaining Walls

Any retaining walls that are to be incorporated into the project should be designed to resist lateral pressures exerted from a media having an equivalent fluid weight as follows:

Gradient of Back Slope	Equivalent Fluid Weight (p.c.f.)			Coefficient of Friction
	Unrestrained Condition (Active)	Restrained Condition (At Rest)	Passive Resistance	
Horizontal	60	80	250	0.32
2:1	70	90	250	0.32

It should be noted that the effects of any surcharge or compaction loads behind the walls must be accounted for in the design of the walls. In addition, an earthquake load of $15H^2$ applied at 0.6H where H = wall height, from the bottom of the wall is applicable. Restrained conditions should be used where framing or other structural members rests on top or is connected to the top of walls.

The above criteria are based on fully drained conditions. In order to achieve fully-drained conditions, a drainage filter blanket should be placed behind the wall. The blanket should be a minimum of 12 inches thick and should extend the full height of the wall. If the excavated area behind the wall exceeds 12 inches, the entire excavated space behind the 12-inch blanket should consist of compacted engineered fill or blanket material. The gravel drainage blanket material may consist of either granular crushed rock or drain pipe fully encapsulated in geotextile filter fabric (Mirafi 140N or equivalent) or Class II permeable material that meets CalTrans Specification, Section 68. A 4-inch diameter SDR35 perforated drain pipe should be installed in the bottom of the drainage blanket and should be underlain by 4 inches of filter type material. Piping with a minimum gradient of 2% shall be provided to discharge water that collects behind the walls to an adequately controlled discharge system away from the structure foundations.

If mechanically stabilized earth, segmental retaining walls such as Keystone walls are utilized, the design and construction of these proposed flexible modular retaining wall systems should conform to the recommendations of the manufacturer and/or Keystone Retaining Wall Systems or the

National Concrete Masonry Association (NCMA). The following soil parameters would be applicable for design using on-site soil materials within the reinforced, retained and bearing zones: φ = 28 degrees, c = 0 p.s.f., γ = 115 p.c.f. The wall backfill within the reinforced zone may consist of the on-site soil materials provided it has a maximum Liquid Limit of 40 and a maximum Plasticity Index of 20. The wall embedment should conform to the recommendations by Keystone or NCMA.

General Construction Requirements

Utility trenches extending underneath all traffic areas must be backfilled with native or import soil materials and compacted to relative compaction of 92% to within 12 inches of the subgrade. The upper 12 inches should be compacted to 95% relative compaction in accordance with Laboratory Test Procedure ASTM D1557. Backfilling and compaction of these trenches must meet the requirements set forth by the City of Pinole, Department of Public Works.

Applicable safety standards require that trenches in excess of 5 feet must be properly shored or that the walls of the trench slope back to provide safety for installation of lines. If trench wall sloping is performed, the inclination should vary with the soil type and applicable OSHA Safety Standards. The soils at the site are considered to be Type B, except where groundwater is encountered Type C should be used.

With respect to state-of-the-art construction or local requirements, utility lines are generally bedded with granular materials. These materials can convey surface or subsurface water beneath the structures. It is, therefore, recommended that all utility trenches which possess the potential to transport water be sealed with a compacted impervious cohesive soil material or lean concrete where the trench enters/exits the building perimeter. This impervious seal should extend a minimum of 2 feet away from the building perimeter.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. It should be noted that it is the responsibility of the owner or his representative to notify ***KC ENGINEERING CO.***, in writing, a minimum of two working days before any clearing, grading, or foundation excavation operations can commence at the site.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings and from a reconnaissance of the site. Should any variations or undesirable conditions be encountered during the development of the site, ***KC ENGINEERING CO.***, will provide supplemental recommendations as dictated by the field conditions.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the Architect and Engineer for the project and incorporated into the plans and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

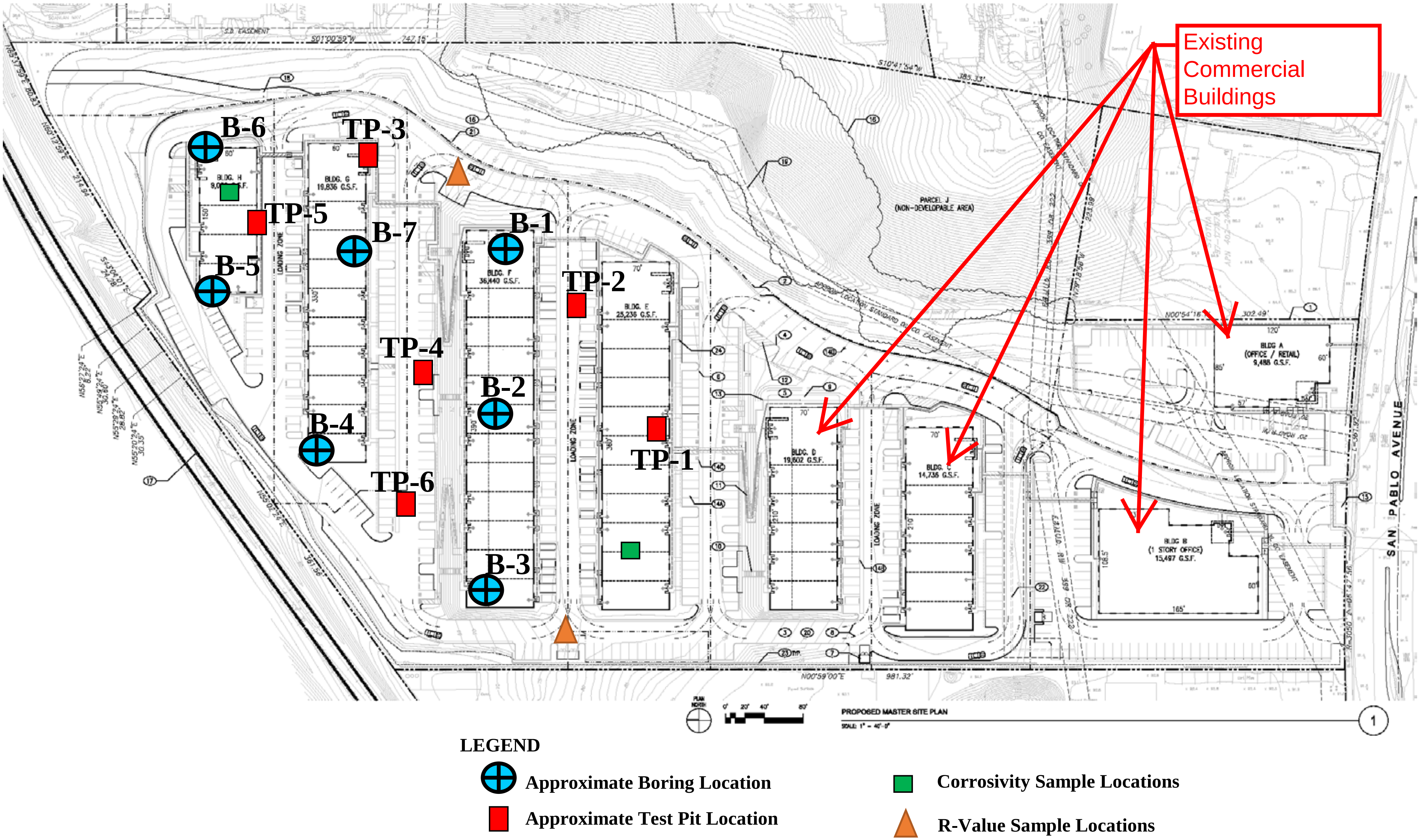
4. At the present date, the findings of this report are valid for the property investigated. With the passage of time, significant changes in the conditions of a property can occur due to natural processes or works of man on this or adjacent properties. In addition, legislation or the broadening of knowledge may result in changes in applicable standards. Changes outside of our control may render this report invalid, wholly or partially. Therefore, this report should not be considered valid after a period of two (2) years without our review, nor should it be used, or is it applicable, for any properties other than those investigated.

5. Not withstanding, all the foregoing applicable codes must be adhered to at all times.

APPENDIX

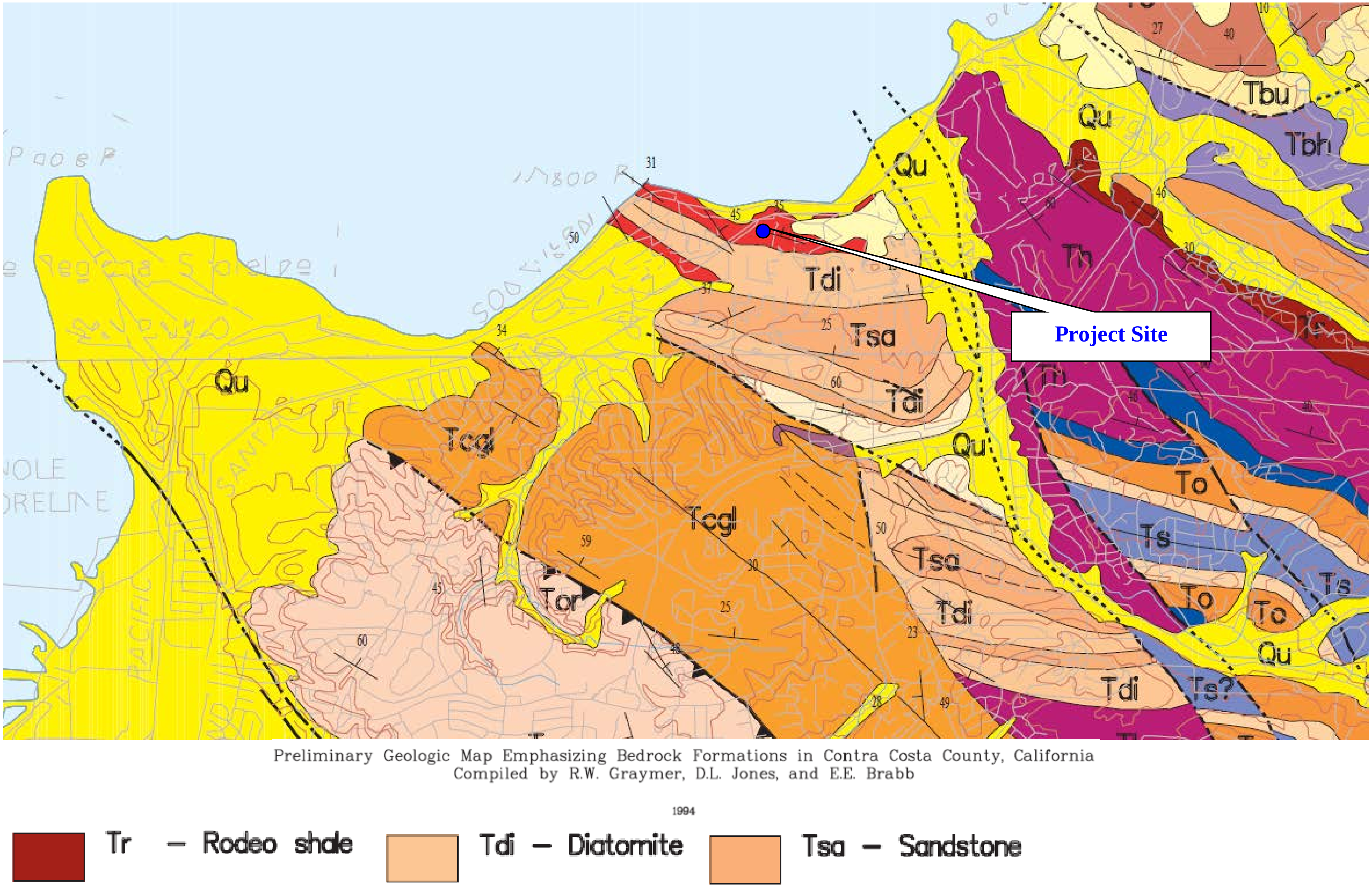
KC ENGINEERING CO.
865 Cotting Lane, Suite A
Vacaville, CA 95688
707.447.4025

Project No. VV3967
Proposed Fill Site & Future Commercial Development
836 San Pablo Ave., Pinole, CA
Figure 1 – AERIAL VICINITY MAP



KC ENGINEERING COMPANY
865 Cotting Lane, Suite A
Vacaville, CA 95688
707-447-4025

Project No. VV3967
Pinole Shores Commercial Project
836 San Pablo Ave., Pinole, CA
Figure 2 – SITE PLAN



Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California
Compiled by R.W. Graymer, D.L. Jones, and E.E. Brabb

1994

KC ENGINEERING COMPANY
865 Cotting Lane, Suite A
Vacaville, CA 95688
707-447-4025

Project No. VV3967
Proposed Fill Site & Future Commercial Development
836 San Pablo Ave., Pinole, CA
Figure 3 – GEOLOGIC MAP

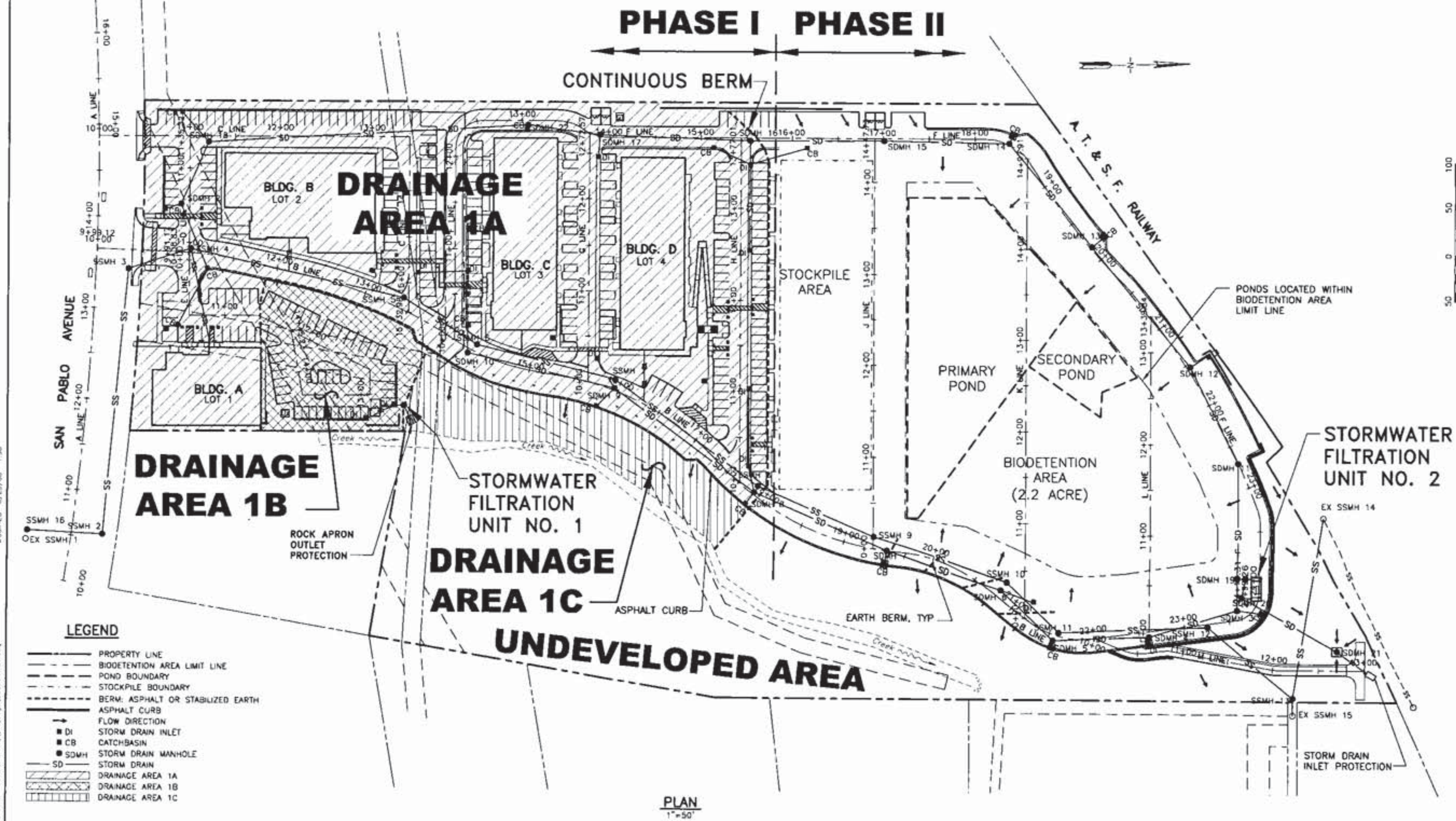
NOTES:

1. STORMWATER FILTRATION UNIT NO. 1 IS SIZED TO ACCOMMODATE A SITE ACREAGE OF 0.5 ACRES. THE STORMWATER MANHOLE UNIT IS A FILTRATION TYPE UNIT AND MEETING THE REQUIREMENTS OF THE REGIONAL WATER QUALITY CONTROL BOARD AND THE CITY OF PINOLE. STORMWATER FILTRATION UNIT NO. 1 IS A PRECAST 72" MANHOLE STORMFILTER MANUFACTURED BY CONTECH.

2. STORMWATER FILTRATION UNIT NO. 2 IS SIZED TO ACCOMMODATE A SITE ACREAGE OF 10 ACRES. THE STORMWATER UNIT IS A FILTRATION TYPE UNIT MEETING THE REQUIREMENTS OF THE REGIONAL WATER QUALITY CONTROL BOARD AND THE CITY OF PINOLE. STORMWATER FILTRATION UNIT NO. 2 IS A 10.5'X24' RECTANGULAR PRECAST STORMFILTER BY CONTECH.

3. AREA 1A IS THE PHASE 1 DEVELOPED AREA LESS AREA 1B AS SHOWN BELOW.

4. AREA 1C SLOPES DIRECTLY TO THE CREEK.



LEGEND

- PROPERTY LINE
- BIODETENTION AREA LIMIT LINE
- POND BOUNDARY
- STOCKPILE BOUNDARY
- BERM: ASPHALT OR STABILIZED EARTH
- ASPHALT CURB
- FLOW DIRECTION
- DI — STORM DRAIN INLET
- CB — CATCHBASIN
- SDMH — STORM DRAIN MANHOLE
- SD — STORM DRAIN
- DRAINAGE AREA 1A
- DRAINAGE AREA 1B
- DRAINAGE AREA 1C

PLAN
1"=50'

PANATTONI DEVELOPMENT COMPANY, LLC
PINOLE SHORES DEVELOPMENT

OPERATIONS AND MAINTENANCE PLAN

Dwg No: OMP-1

Designed By: RKL	Scale: 1"=50'
Drawn By: A/V, DDE	Project No: 5862-01
Checked By: RKL	Date: 09/07/07

AN WEST INC. Consulting Engineers
3095 Richmond Parkway
Suite 201
Richmond, California 94806

Rev.	Date	Description	By

LOG OF TEST BORING

BORING NO.: 1

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL ∇

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL: ▼: AFTER: HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Brown Sandy CLAY; dry, stiff. (FILL)	CL				
	1-1			Brown Sand CLAYSTONE, highly weathered, weak, closely fractured. (NATIVE)	Rx	53	82.5	33.9	LL=68% PI=37
5									
	1-2					50-6"	81.3	22.5	
				Boring Terminated @ 7.5'. No Groundwater Encountered.					
10									
15									
20									
25									

This information pertains only to this boring and is not necessarily indicative of the whole site.

LOG OF TEST BORING
BORING NO.: 2

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL: : AFTER: HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Brown Sandy CLAY; dry to moist, hard, mixed w/ rock fragments. (FILL)	CL				
	2-1					50-4"	82.8	32.6	
				Brown SANDSTONE, weathered, weak. (NATIVE)	Rx				
5									
	2-2					50-5"			
				Boring Terminated @ 8'. No Groundwater Encountered.					
10									
15									
20									
25									

This information pertains only to this boring and is not necessarily indicative of the whole site.

LOG OF TEST BORING

BORING NO.: 3

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL ▽ 15'

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL: ▼: AFTER: HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Dark Brown Sandy CLAY w/ Claystone Rock Fragments; moist, stiff. (FILL)	CH				
	3-1					15	91.2	26.7	LL=54% PI=31
5	3-2			As Above.		18	92.8	26.6	UCC=4345 psf Qp=4.0 tsf
10				Dark Brown CLAY w/ Shale Fragments, very moist, stiff. (FILL)	CH				
	3-3					13			Qp=3.0 tsp <200=55%
15				Black Gravelly CLAY; wet, stiff, gravels to 1/2". (NATIVE) ▽	CL/ CH				
20				Light Yellowish Brown CLAYSTONE/ SHALE, weak, highly weathered.	Rx				
	3-4					24	50	77.3	
25	3-5			As Above.		17			
				Boring Terminated @ 26.5'. Groundwater Encountered @ 15'.					

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

LOG OF TEST BORING

BORING NO.: 4

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL ∇ 24'

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL: ▼: AFTER: HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Brown & Black Sandy CLAY mixed w/ Claystone Fragments; dry to moist, very stiff. (FILL)	CL				
	4-1					26	68.3	47.4	
5				Dark Sandy CLAY w/ some Claystone inclusions; moist, stiff. (NATIVE)	CH				
	4-2					16	88.4	31.0	UCC=2507 psf Qp=2.5 tsf
10									
15	4-3			As Above.		16	83.4	32.7	
20									
				Brown CLAYSTONE/SHALE/DIATOMITE, highly weathered & fractured.	Rx				
25	4-4					18	70.8	48.1	
				Boring Terminated @ 26.5'. Groundwater Encountered @ 24'.					

This information pertains only to this boring and is not necessarily indicative of the whole site.

LOG OF TEST BORING

BORING NO.: 5

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL ▽ :

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL ▼ : AFTER: hrs.

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Mottled Brown Silty Sandy CLAY w/ Diatomite Fragments; moist, very stiff. (FILL)	CL				
	5-1					23	70.4	52.3	c=2,858 psf Ø=22º
5									
				Light Yellowish Brown CLAYSTONE/DIATOMITE; moist, highly weathered & fractured, weak.	Rx				
	5-2					30	64.3	56.9	
10									
15									
	5-3			As Above.		39			
				Boring Terminated @ 16.5'. No groundwater encountered.					
20									
25									

This information pertains only to this boring and is not necessarily indicative of the whole site.

LOG OF TEST BORING
BORING NO.: 6

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL: : AFTER: HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0	6-1			Dark Brown CLAY mixed w/ Claystone Fragments; moist, very stiff. (FILL)	CH	28	79.5	24.7	LL=62% PI=34 UCC=5008 psf
	6-1A								
5	6-2			Light Brown CLAYSTONE, highly weathered. (BOULDER FILL)	Rx	22			
				Dark Brown Sandy CLAY mixed w/ Claystone Fragments; very moist, firm. (FILL)	CL				
10	6-3					8			<200=47%
15									
	6-4			As Above. (FILL)		16	90.1	28.2	Qp=2.75 tsf <200=53%
20									
25				Brown CLAYSTONE, highly weathered, weak.	Rx				

This information pertains only to this boring and is not necessarily indicative of the whole site.

LOG OF TEST BORING
BORING NO.: 6

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL ▽

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL: ▼: AFTER: HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
	6-5					19			
30				Boring Terminated @ 29.5'. No Groundwater Encountered.					
35									
40									
45									
50									
55									

This information pertains only to this boring and is not necessarily indicative of the whole site.

LOG OF TEST BORING
BORING NO.: 7

PROJECT: Pinole Shores Commercial Project
CLIENT: Kiewit
LOCATION: 836 San Pablo, Pinole
DRILLER: Britton Exploration
DRILL RIG: CME 55
DEPTH TO WATER: INITIAL

PROJECT NO.: VV3967
DATE: 09/07/16
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 5"
FINAL: : AFTER: HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Brown Silty CLAY mixed w/ Claystone Fragments; dry to moist, very stiff. (FILL)	CH				
	7-1					22	72.5	45.1	
5				Dark Brown CLAY; moist, stiff. (NATIVE)	CL				
	7-2					50-5"			
10									
	7-3			As Above.		62			
15				Boring Terminated @ 15. No Groundwater Encountered.					
20									
25									

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

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GEOTECHNICAL ENGINEERING CONSULTANTS

KC ENGINEERING COMPANY
A SUBSIDIARY OF MATERIALS TESTING, INC.

TEST PIT LOG

Client: Kiewit Infrastructure West Co.
4650 Business Center Drive
Fairfield, CA 94534

Project No: VV3967
Date of Test Pits: 7 September 2016

Project: Pinole Shores Commercial Project
836 San Pablo Avenue
Pinole, CA

TEST PIT No.	DEPTH (feet)	USCS	DESCRIPTION
TP-1	0 – 4’	GM/MH	Light Brown Clayey SILT & Diatomite Fragments; moist, stiff. (FILL) (LL=89%, PI=34, EI=25)
	4 – 6.5’	CH	Dark Brown Silty CLAY; very moist, stiff. (NATIVE)
	6.5 – 8’	Rx	Light Yellowish Brown CLAYSTONE, highly weathered & fractured, weak, seepage @ 7.5’.
TP-2	0 – 8’	GM/MH	Light Brown Clayey SILT & Diatomite & Claystone Fragments; loose, dry to moist. (STOCKPILE) (LL=89%, PI=34, EI=25)
TP-3	0 – 8’	CL	Light Brown & Dark Brown Sandy CLAY w/ Mudstone Rock Fragments; moist, top 1’ loose then stiff. (FILL)
TP-4	0 – 2.5’	Rx	Yellowish Brown SILTSTONE, highly weathered, weak, fractured.

TP-5	0 – 3.5'	CL	Light to Dark Brown CLAY & MUDSTONE Rock Fragments; dry to moist, firm to stiff. (FILL)
	3.5 – 7'	Rx	Light Brown CLAYSTONE, highly weathered, weak. (NATIVE)
TP-6	0 – 7.5'	CL	Light Brown Sandy CLAY w/ Mudstone & Dolomite Rock Fragments; top 2' loose then stiff & moist. (FILL)
	7.5 – 9'	CH	Dark Brown Silty CLAY w/ some rock fragments; very moist, stiff. (NATIVE)

Note: Groundwater Encountered During the Field Exploration.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is retained on the No. 200 Sieve	GRAVEL More than half of coarse fraction is larger than No. 4 sieve	Clean gravels (<5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines (Cu>4 & 1<Cc<3)
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines (Cu < 4 and/or 1>Cc>3)
		Gravel with fines (>12% fines)	GM	Silty gravels and gravel-sand-silt mixtures (PI<4 or below "A" line)
			GC	Clayey gravels and gravel-sand-clay mixtures (PI>7 & on or above "A" line)
	SAND Half or more of the coarse fraction is smaller than No. 4 sieve	Clean sands (<5% fines)	SW	Well graded sands, gravelly sands, little or no fines (Cu>6 & 1<Cc<3)
			SP	Poorly graded sands, gravelly sands, little or no fines (Cu<6 and/or 1>Cc>3)
		Sand with fines (>12% fines)	SM	Silty sands and gravel-sand-silt mixtures (PI<4 or below "A" line)
			SC	Clayey sands and gravel-sand-clay mixtures (PI>7 & on or above "A" line)
FINE GRAINED SOILS Half or more of the material passes the No. 200 Sieve	SILTS AND CLAYS Liquid Limit is less than 50%		ML	Inorganic silts with gravel and sand having slight plasticity (PI<4 or below "A" line)
			CL	Inorganic clays of low to med. plasticity with gravel and sand (PI>7 & on or above "A" line)
			OL	Organic silts and clays of low plasticity
	SILTS AND CLAYS Liquid Limit is 50% or more		MH	Inorganic elastic silts (PI below "A" line)
			CH	Inorganic clays of high plasticity, fat clays (PI on or above "A" line)
			OH	Organic silts and clays of medium to high plasticity
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

MTI-KC ENGINEERING COMPANY
865 Cotting Lane, Ste A, Vacaville, CA 95688
8798 Airport Road, Redding, CA 96002

SAMPLER AND LAB TESTING LEGEND

- Auger
- Bulk Sample, taken from auger cuttings
- California Sampler
- Bulk/Grab Sample
- Pitcher
- Standard Penetration Test
- Shelby Tube
- No Recovery

LL=Liquid Limit (%)
PI=Plasticity Index
¦ =Friction Angle
C=Cohesion
UCC=Unconfined Compression
R value=Resistance Value
Consol=Consolidation Test

SOIL GRAIN SIZE

U.S. STANDARD SIEVE OPENINGS

		#200		#40	#10	#4	¾"	3"	12"
CLAY	SILT	SAND			GRAVEL		COBBLES	BOULDERS
		FINE	MEDIUM	COARSE	FINE	COARSE		
0.002	0.075	0.425	2.00	4.75	19.0	75	300	

SOIL GRAIN SIZE IN MILLIMETERS

RELATIVE DENSITY (Coarse-grained soils)

SANDS & GRAVELS	BLOWS/FOOT[1]
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	> 50

CONSISTENCY (Fine-grained soils)

SILTS & CLAYS	STRENGTH[2]	BLOWS/FOOT[1]
Very Soft	< 500	0 – 2
Soft	500 – 1,000	2 – 4
Firm	1,000 – 2,000	4 – 8
Stiff	2,000 – 4,000	8 – 15
Very Stiff	4,000 – 8,000	15 – 30
Hard	> 8,000	>30

1 – Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. split spoon sampler (ASTM D1586)
2 – Unconfined compressive strength in lb/ft^2 as determined by lab testing or approximated by the standard penetration test (ASTM D1586) or pocket penetrometer.

WEATHERING (Bedrock)

Fresh	No visible sign of decomposition or discoloration; rings under hammer impact
Slightly weathered	Slight discoloration inwards from open fractures; little or no effect on normal cementation; otherwise similar to Fresh
Moderately weathered	Discoloration throughout; weaker minerals decomposed; strength somewhat less than fresh rock but cores can not be broken by hand or scraped with knife; texture preserved; cementation little to not affected; fractures may contain filling
Highly weathered	Most minerals somewhat decomposed; specimens can be broken by hand with effort or shaved with knife; texture becoming indistinct but fabric preserved; faint fractures
Completely weathered	Minerals decomposed to soil but fabric and structure preserved; specimens can be easily crumbled or penetrated

STRENGTH (Bedrock)

Plastic	Very low strength
Friable	Crumbles easily by rubbing with fingers
Weak	An unfractured specimen will crumble under light hammer blows
Moderately strong	Specimen will withstand a few heavy hammer blows before breaking
Strong	Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments
Very strong	Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

BEDDING (Bedrock)	SPACING (inches)
Very thickly bedded	> 48
Thickly bedded	24 to 48
Thin bedded	2.5 to 24
Very thin bedded	5/8 to 2.5
Laminated	1/8 to 5/8
Thinly laminated	<1/8

FRACTURING (Bedrock)	SPACING (inches)
Very little fractured	> 48
Occasionally fractured	12 to 48
Moderately fractured	6 to 12
Closely fractured	1 to 6
Intensely fractured	5/8 to 1
Crushed	<5/8

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Redding, California 96002
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Vacaville, California 95688
(707) 447-4025, fax 447-4143

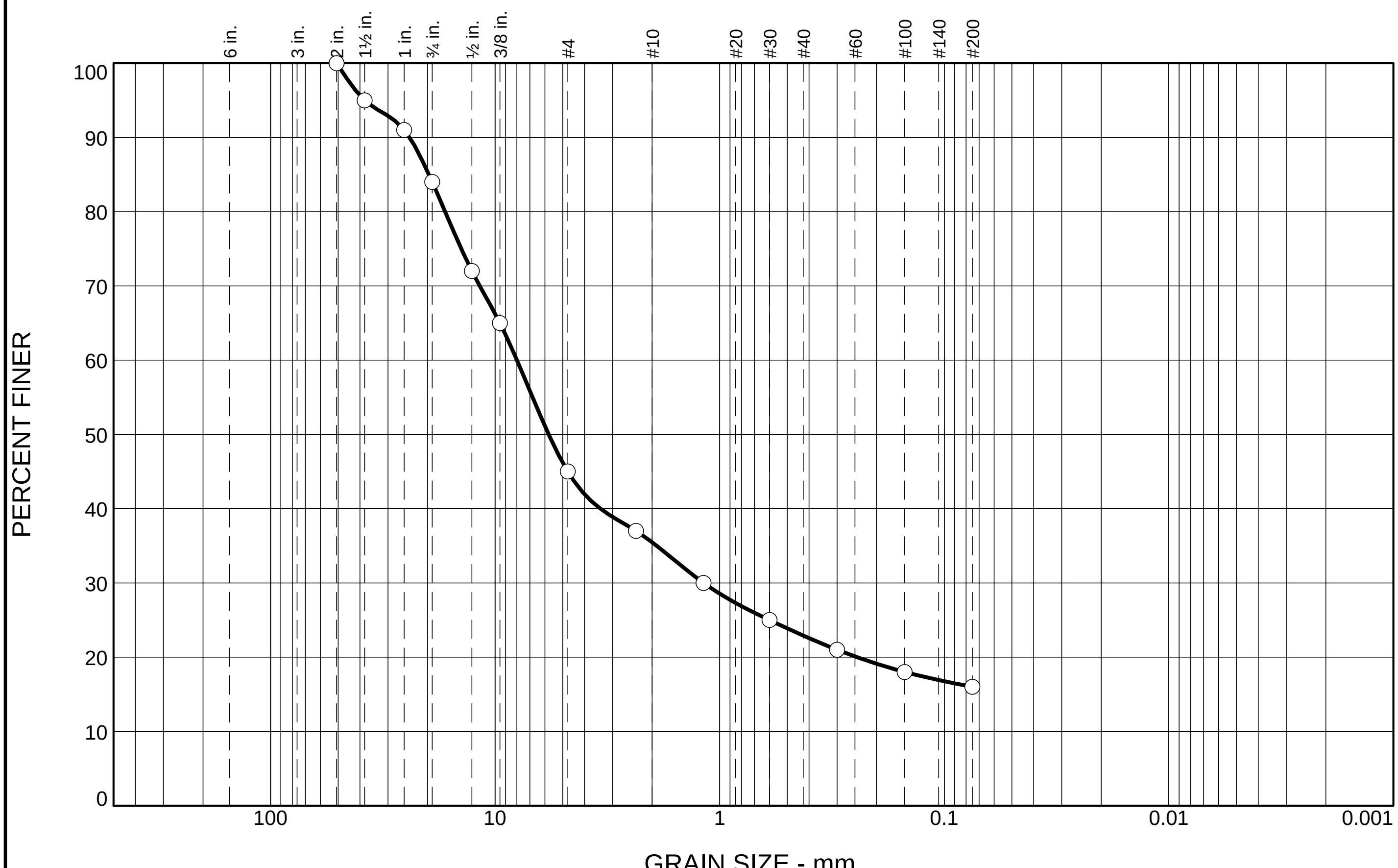
Client:	Kiewit Infrastructure West Inc. 4650 Business Center Drive Fairfield, CA 94534	**Client No.:** **Report No.:** **Date:**	VV3967-004 0300-001 09/21/16
Project:	836 San Pablo Avenue Pinole, California	**Submitted by:** **Submitted Date:**	KC Engineering 09/09/16

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937) and Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
Composite TP1 @ 0-4.0' & TP2 @ 0-7.0'	Light Brown Clayey Silt with Claystone Gravels (visual)	---	---	89	55	34
1-1 @ 3.0'	Brown Sandy Claystone (visual)	82.5	33.9	68	31	37
1-2 @ 7.0'	Brown Sandy Claystone (visual)	81.3	22.5	---	---	---
2-1 @ 2.5'	Yellowish Brown Sandy Clay (visual)	82.8	32.6	---	---	---
3-1 @ 2.0'	Brown Sandy Clay (visual)	91.2	26.7	54	23	31
3-2 @ 6.0'	Dark Brown Sandy Clay (visual)	92.8	26.6	---	---	---
3-4 @ 21.0'	Light Brown Claystone (visual)	50.0	77.3	---	----	---
4-1 @ 3.0'	Yellowish Brown Sandy Clay (visual)	68.3	47.4	---	---	---
4-2 @ 9.0'	Dark Brown Sandy Clay (visual)	88.4	31.0	---	---	---
4-3 @ 16.0'	Dark Brown Sandy Clay (visual)	83.4	32.7	---	---	---

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
4-4 @ 26.0’	Dark Brown Claystone / Diatomite (visual)	70.8	48.1	---	---	---
5-1 @ 2.5’	Brown Sandy Clay with Gravel (visual)	70.4	52.3	---	---	---
5-2 @ 9.0’	Light Brown Sandy Claystone / Diatomite (visual)	64.3	56.9	---	---	---
6-1 @ 1.0’	Light Brown Sandy Clay (visual)	79.5	24.7	---	---	---
6-1A @ 1.0 - 3.0’	Strong Brown Sandy Clay with Gravel (visual)	---	---	62	28	34
6-2 @ 6.0’	Light Brown Claystone (visual)	58.3	58.8	---	---	---
6-4 @ 19.0’	Dark Brown Sandy Clay with Gravel (visual)	90.1	28.2	---	---	---
7-1 @ 4.0’	Dark Brown Clay (visual)	72.5	45.1	---	---	---

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	16	39	10	12	7	16	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1-1/2"	95		
1"	91		
3/4"	84		
1/2"	72		
3/8"	65		
#4	45		
#8	37		
#16	30		
#30	25		
#50	21		
#100	18		
#200	16		

* (no specification provided)

Material Description

Light Brown Clayey Silt w/ Claystone Gravels (visual)

Atterberg Limits

PL= 55 LL= 89 PI= 34

Coefficients

D_{90}= 24.0463 D_{85}= 19.7121 D_{60}= 7.9978
D_{50}= 5.7765 D_{30}= 1.1800 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= GM / MH AASHTO= A-2-7(0)

Remarks

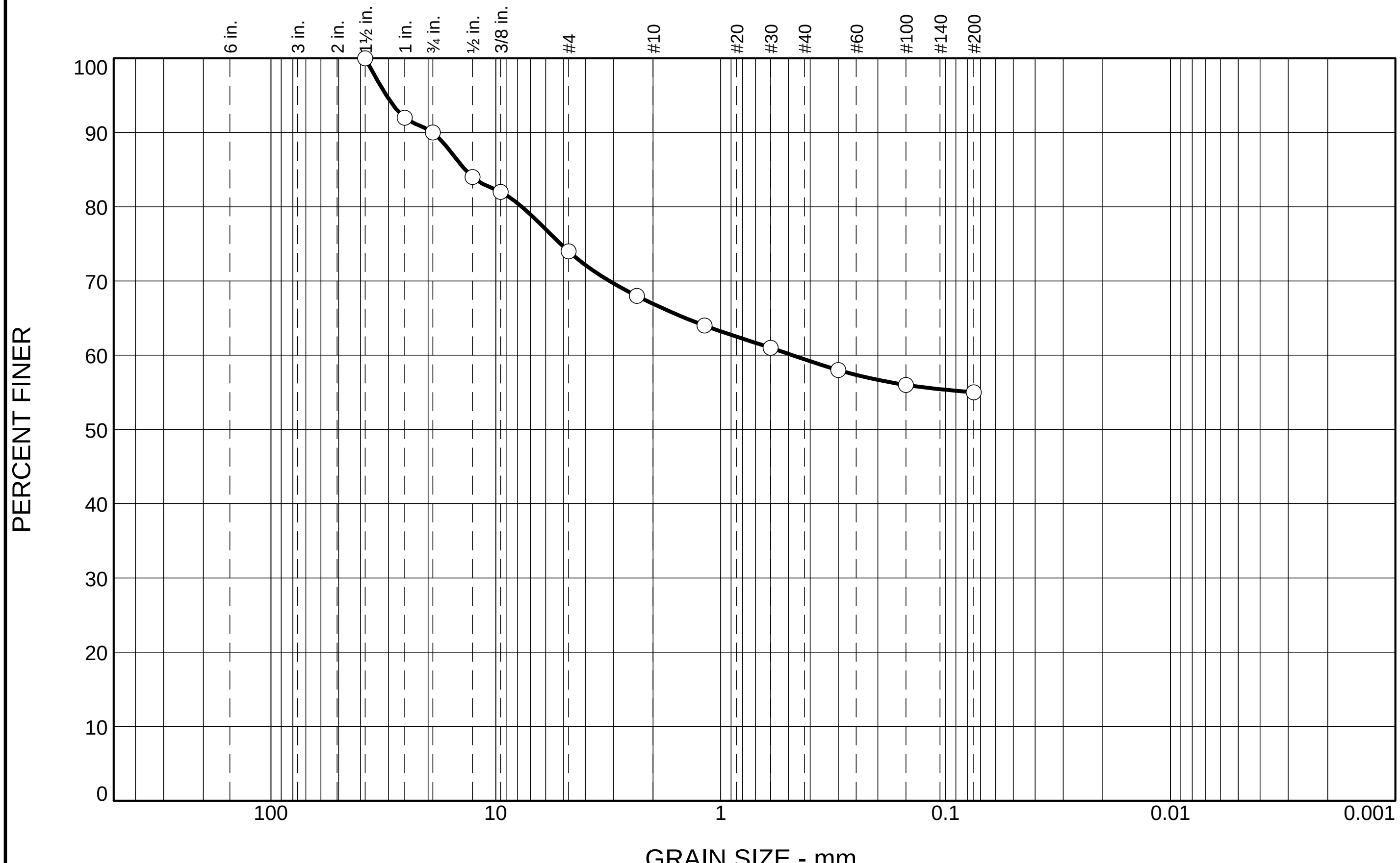
Sieve tested in accordance with ASTM D6913.
Atterberg Limits tested in accordance with ASTM D4318.

Location: TP1 & TP2 (Composite)
Sample Number: 1 **Depth:** 0.0' - 7.0' **Date:** 09/21/16

Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: 836 San Pablo Avenue
Pinole, CA
Project No: VV3967-004 **Figure** 0300-002

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	10	16	7	8	4	55	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2"	100		
1"	92		
3/4"	90		
1/2"	84		
3/8"	82		
#4	74		
#8	68		
#16	64		
#30	61		
#50	58		
#100	56		
#200	55		

* (no specification provided)

Material Description

Dark Brown Sandy Clay with Gravel (visual)

Atterberg Limits

PL= -- LL= --- PI= ---

Coefficients

D_{90}= 19.0500 D_{85}= 13.6981 D_{60}= 0.4797
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

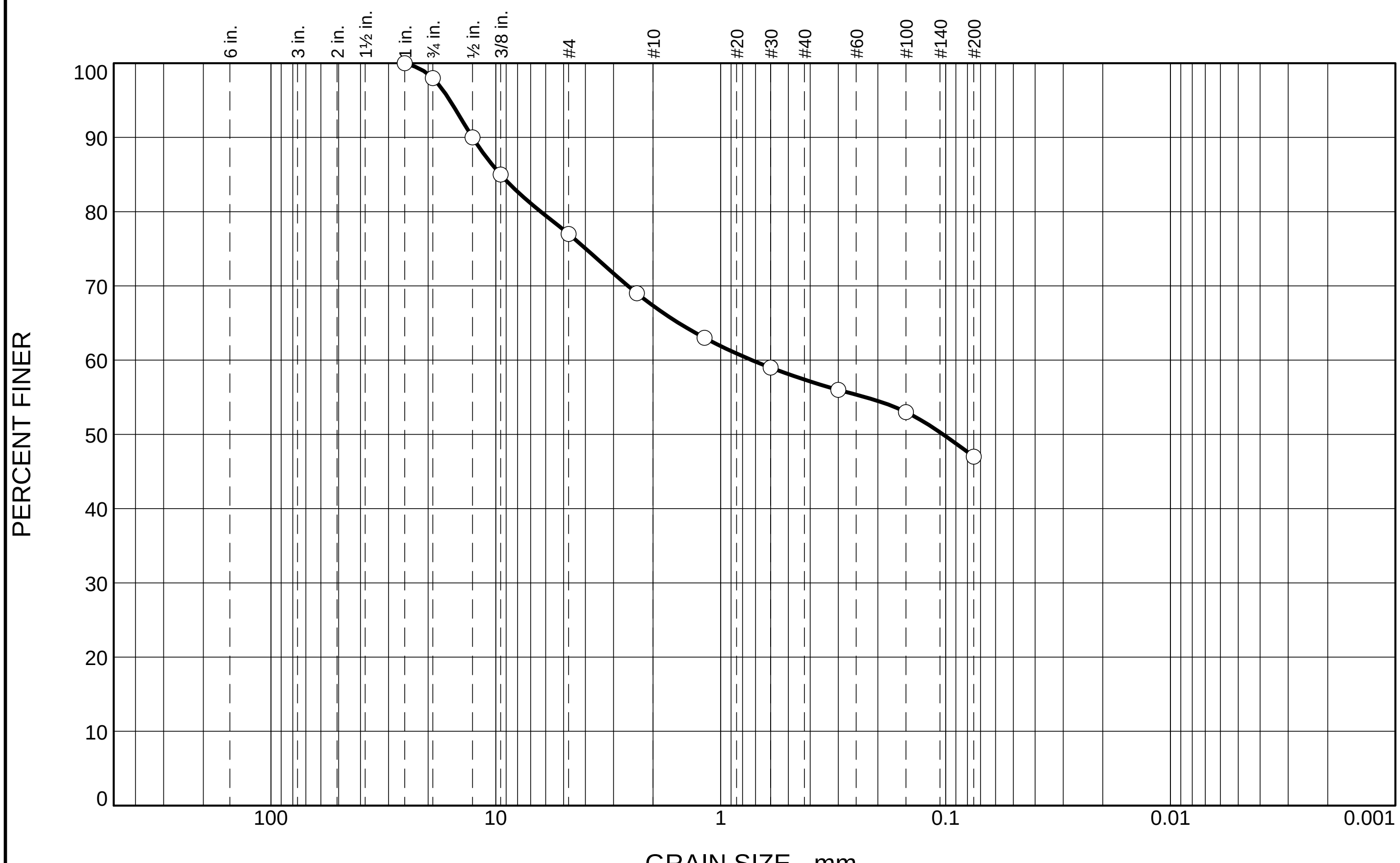
Material tested in accordance with ASTM D6913.

Location: 3-3
Sample Number: 9 **Depth:** 11.0' **Date:** 09/21/16

Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: 836 San Pablo Avenue
Pinole, CA
Project No: VV3967-004 **Figure** 0300-003

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	2	21	10	10	10	47	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100		
3/4"	98		
1/2"	90		
3/8"	85		
#4	77		
#8	69		
#16	63		
#30	59		
#50	56		
#100	53		
#200	47		

* (no specification provided)

Material Description

Dark Brown Clayey Sand with Gravel (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 12.7000 D_{85}= 9.5250 D_{60}= 0.7264
D_{50}= 0.1026 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

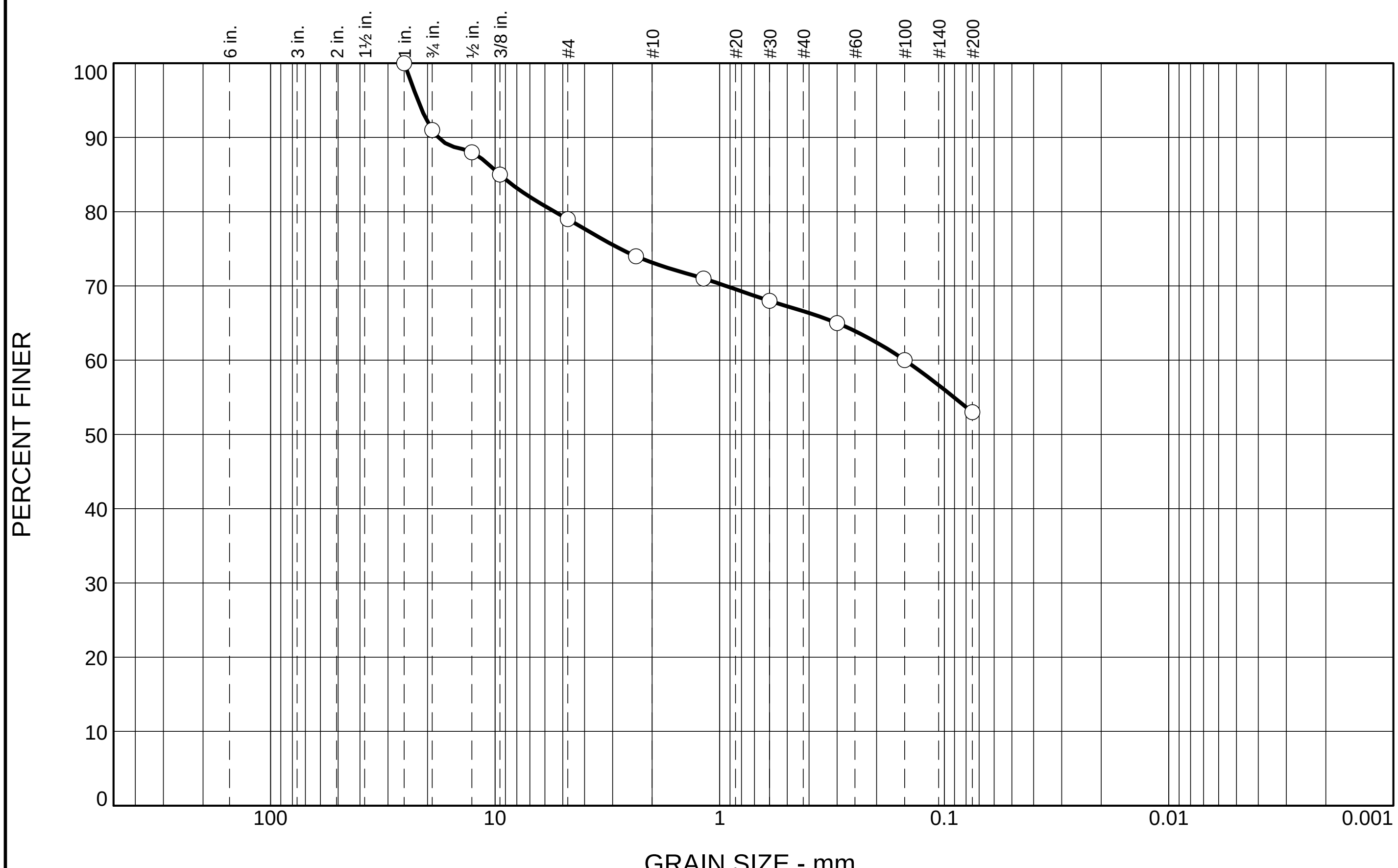
Material tested in accordance with ASTM D6913.

Location: 6-3
Sample Number: 20 **Depth:** 10.0'- 11.5' **Date:** 09/21/16

Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: 836 San Pablo Avenue
Pinole, CA
Project No: VV3967-004 **Figure** 0300-004

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	9	12	6	6	14	53	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100		
3/4"	91		
1/2"	88		
3/8"	85		
#4	79		
#8	74		
#16	71		
#30	68		
#50	65		
#100	60		
#200	53		

* (no specification provided)

Material Description

Dark Brown Sandy Clay with Gravel (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 17.9401 D_{85}= 9.5250 D_{60}= 0.1500
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

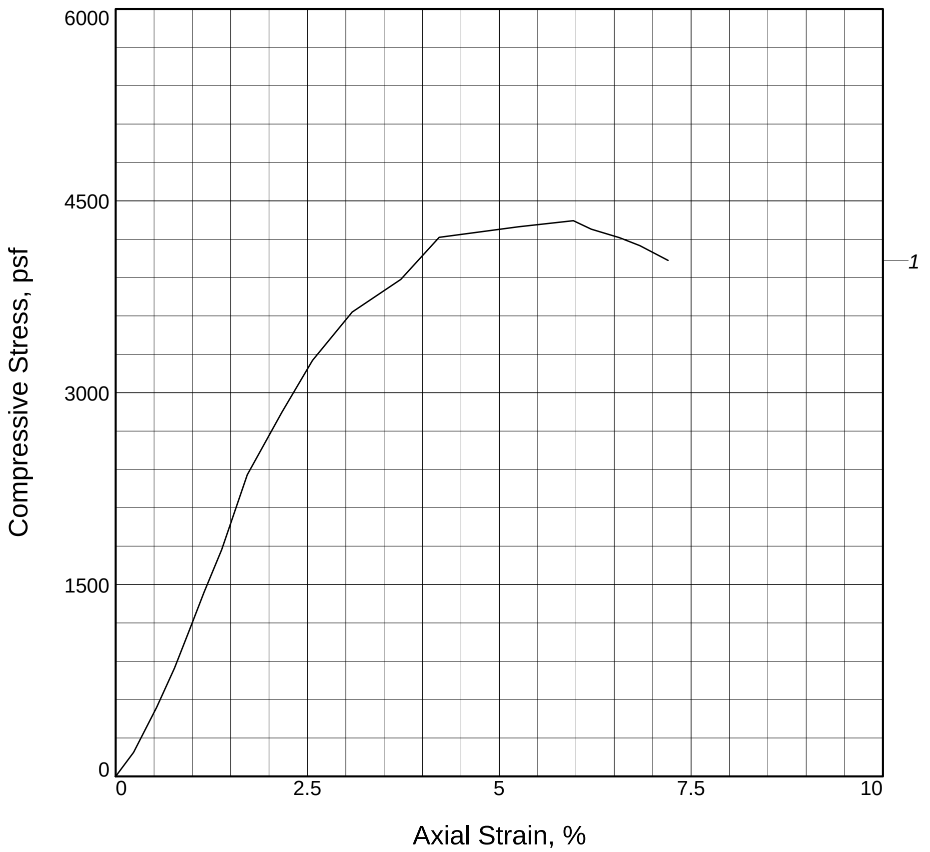
Material tested in accordance with ASTM D6913.

Location: 6-4
Sample Number: 21 **Depth:** 19.0' **Date:** 09/21/16

Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: 836 San Pablo Avenue
Pinole, CA
Project No: VV3967-004 **Figure** 0300-005

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	4345			
Undrained shear strength, psf	2172			
Failure strain, %	6.0			
Strain rate, in./min.	N/A			
Water content, %	26.6			
Wet density, pcf	117.4			
Dry density, pcf	92.8			
Saturation, %	92.7			
Void ratio	0.7427			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Dark Brown Sandy Clay (visual)

LL = ---	**PL =**	**PI =** ---	**GS=** 2.59	**Type:** tube

Project No.: VV3967-004

Date Sampled: 09/07/16

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure: Bulge / Columnar

Figure 0300-006

Client: Kiewit Infrastructure West Inc.

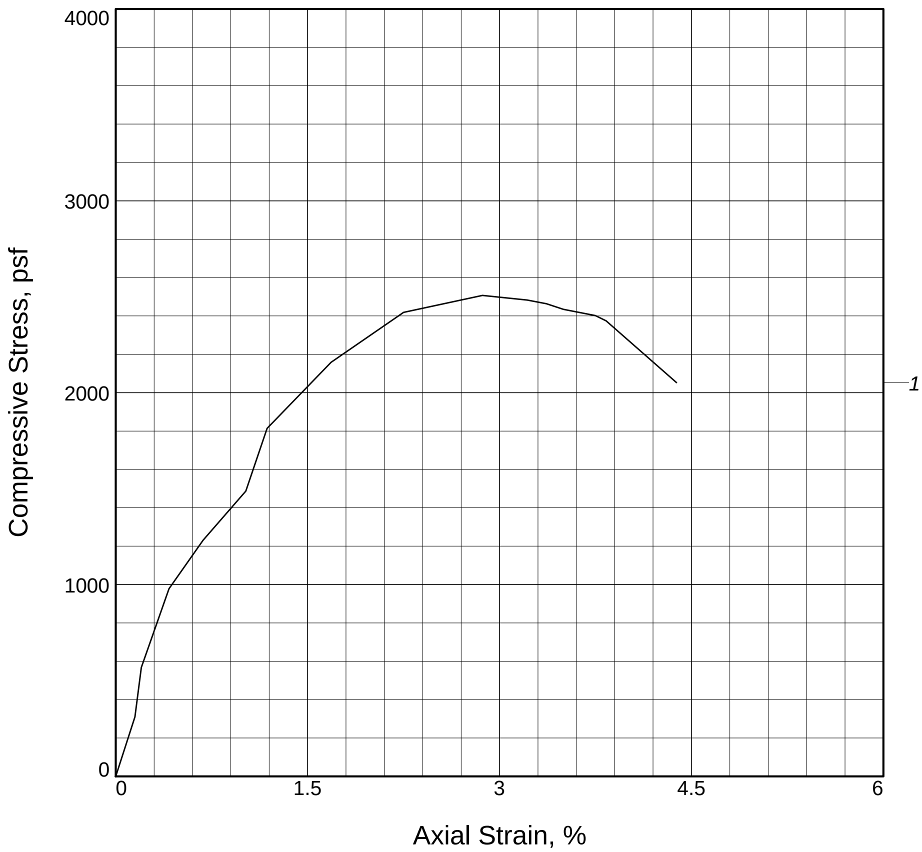
Project: 836 San Pablo Avenue
Pinole, CA

Location: 3-2

Sample Number: 8 **Depth:** 6.0'

Materials Testing, Inc.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	2507			
Undrained shear strength, psf	1254			
Failure strain, %	2.9			
Strain rate, in./min.	N/A			
Water content, %	31.0			
Wet density, pcf	115.8			
Dry density, pcf	88.4			
Saturation, %	101.2			
Void ratio	0.7660			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Dark Brown Sandy Clay (visual)

LL = ---	**PL =**	**PI =** ---	**GS=** 2.50	**Type:** tube

Project No.: VV3967-004
Date Sampled: 09/07/16

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure: Bulge / Shear

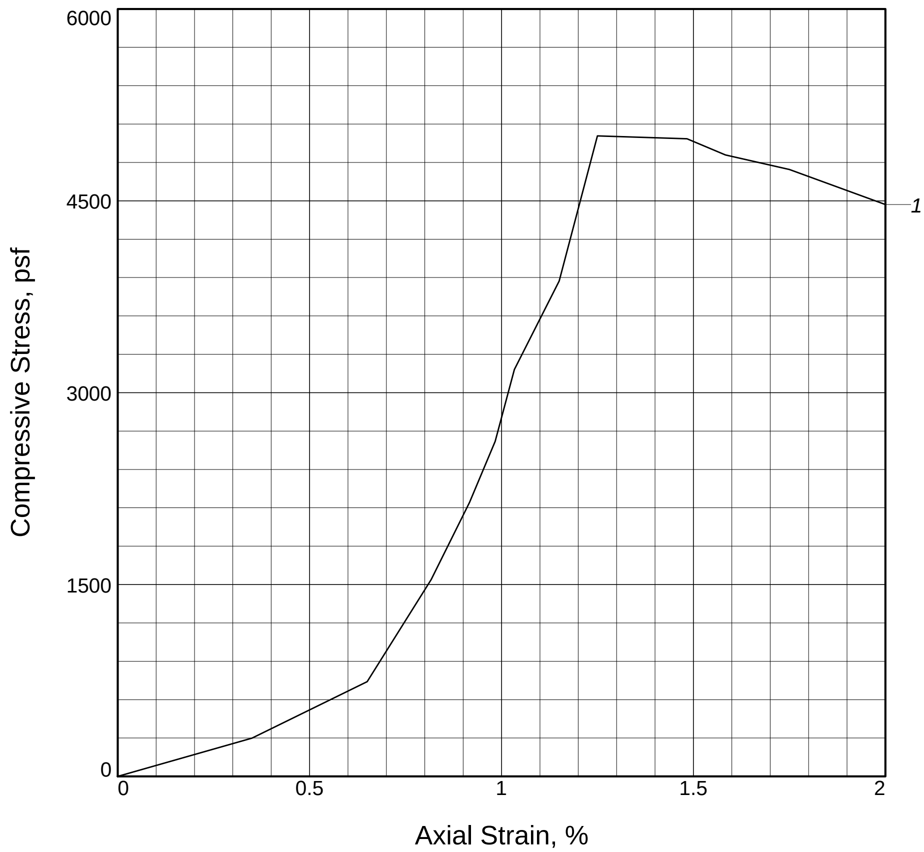
Figure 0300-007

Client: Kiewit Infrastructure West Inc.

Project: 836 San Pablo Avenue
Pinole, CA
Location: 4-2
Sample Number: 12 **Depth:** 9.0'

Materials Testing, Inc.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	5008			
Undrained shear strength, psf	2504			
Failure strain, %	1.3			
Strain rate, in./min.	N/A			
Water content, %	24.7			
Wet density, pcf	99.1			
Dry density, pcf	79.5			
Saturation, %	61.7			
Void ratio	1.0422			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Light Brown Sandy Clay (visual)

LL = ---	**PL =**	**PI =** ---	**GS=** 2.60	**Type:** tube

Project No.: VV3967-004

Date Sampled: 09/07/16

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure: Cone & Shear

Figure 0300-008

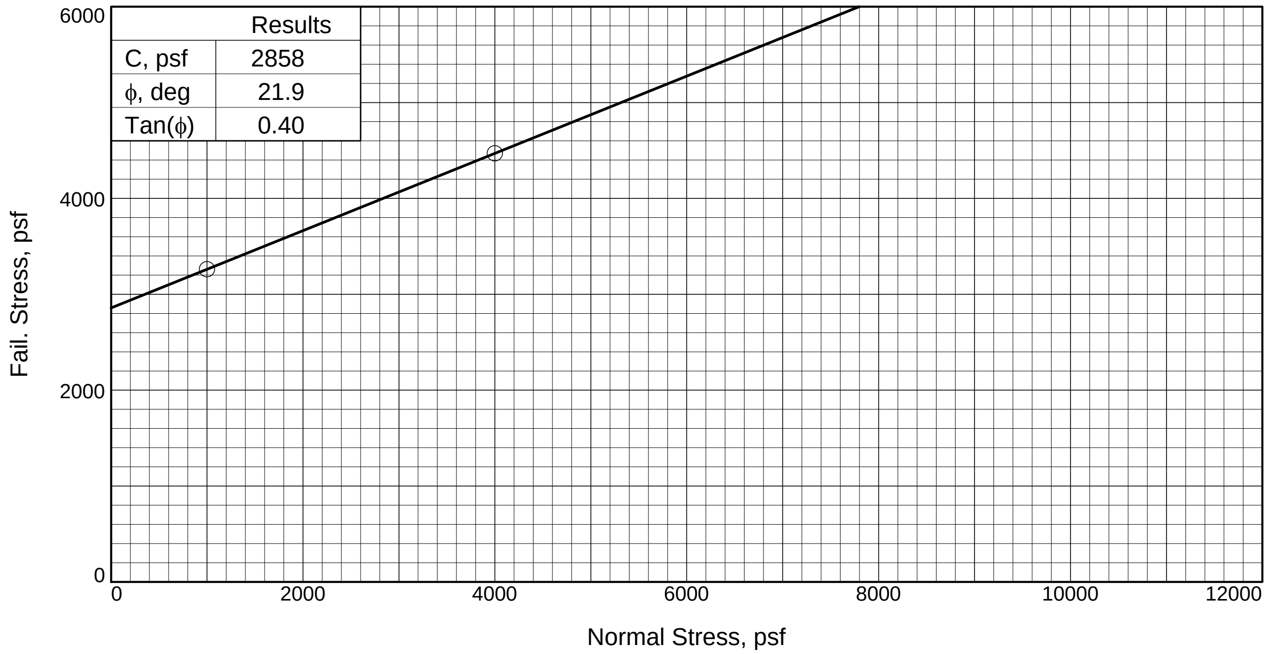
Client: Kiewit Infrastructure West Inc.

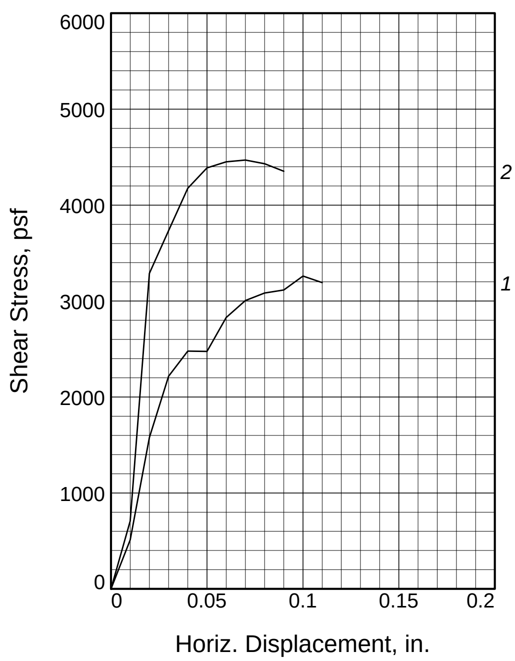
Project: 836 San Pablo Avenue
Pinole, CA

Location: 6-1

Sample Number: 17 **Depth:** 1.0'

Materials Testing, Inc.





Sample No.		1	2
Initial	Water Content, %	52.3	52.3
	Dry Density, pcf	66.9	73.9
	Saturation, %	96.6	115.3
	Void Ratio	1.3815	1.1570
	Diameter, in.	2.41	2.41
	Height, in.	1.00	1.00
At Test	Water Content, %	62.8	52.8
	Dry Density, pcf	67.2	74.5
	Saturation, %	116.8	118.2
	Void Ratio	1.3725	1.1400
	Diameter, in.	2.41	2.41
	Height, in.	1.00	0.99
Normal Stress, psf		1000	4000
Fail. Stress, psf		3261	4470
Displacement, in.		0.10	0.07
Ult. Stress, psf			
Displacement, in.			
Strain rate, in./min.		0.00	0.00

Sample Type: tube

Description: Brown Sandy Clay with Gravel (visual)

Specific Gravity= 2.553

Remarks: Material tested in accordance with ASTM D3080

Figure 0300-009

Client: Kiewit Infrastructure West Inc.

Project: 836 San Pablo Avenue
Pinole, CA

Location: 5-1

Sample Number: 15 **Depth:** 2.5'

Proj. No.: VV3967-004 **Date Sampled:** 09/07/16

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(707) 447-4025, fax 447-4143

Client	Kiewit Infrastructure West Inc. 4650 Business Center Drive Fairfield, CA 94534	**Client No.** **Report No.** **Date:**	VV3967-004 0300-010 09/21/16
Project:	836 San Pablo Avenue Pinole, California	**Submitted By:** **Submitted Date:**	KC Engineering 09/09/16

EXPANSION INDEX
(ASTM D4829)

Sample #:	Composite - TP1 @ 0.0 - 4.0’ & TP2 @ 0.0 - 7.0’
Soil Description:	Light Brown Clayey Silt with Claystone Gravels
Initial Moisture Content (%):	39.9
Moisture Content after Test (%):	69.4
Initial Dry Density (lb/ft^3):	54.6
After Test Wet Density (lb/ft^3):	92.5
Degree of Saturation (%):	51.9
Expansion Index:	25

Table 1 Classification of Potential Expansion of Soils Using EI (ASTM D4829-11)

Expansion Index, EI	Potential Expansion
0 - 20	Very Low
21 – 50	Low
51 – 90	Medium
91 – 130	High
>130	Very High

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(707) 447-4025, fax 447-4143

Client:	Kiewit Infrastructure West Inc. 4650 Business Center Drive Fairfield, CA 94534	**Client No:** **Report No:** **Date:**	VV3967-004 0300-012 09/21/16
Project:	836 San Pablo Avenue Pinole, California	**Submitted By:** **Source:**	KC Engineering

"R" VALUE TEST REPORT
(CTM 301)

Sample:	2
Description:	Gray Clayey SILT with Claystone & Diatomite Gravels
Location:	Composite of Subgrade @ 0.0 - 3.0'

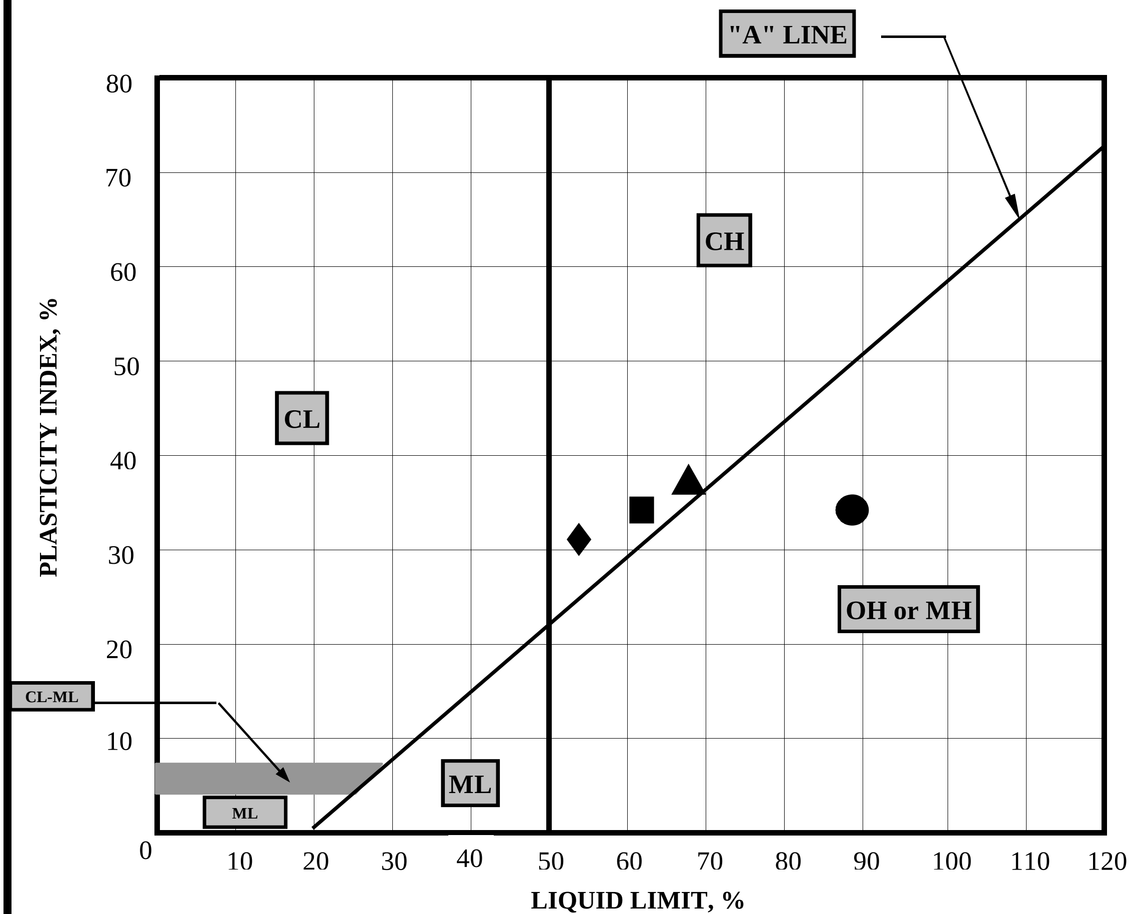
SIEVE ANALYSIS

Sieve Size	2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4
"As Received" (Percent Pass)	100	98	94	87	77	72	59
"As Used" (Percent Pass)				100	89	83	68

RESISTANCE VALUE

Specimen Number	Dry Unit Weight, PCF	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure Dial Reading & PSF		R-Value
1	81.0	34.5	457	55	238	60
2	83.8	35.6	343	18	78	44
3	79.0	39.4	247	13	93	40

R-Value @ 300 PSI Exudation Pressure = 42



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL, %	PLASTIC LIMIT, PL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	TP1&TP2	0.0'-7.0'	N/A	89	55	34	N/A	GM / MH
▲	1-1	3.0'	N/A	68	31	37	N/A	CH
◆	3-1	2.0'	N/A	54	23	31	N/A	CH
■	6-1A	1.0'-3.0'	N/A	62	28	34	N/A	CH

MATERIALS TESTING INCORPORATED
M T I

Materials Testing, Inc.

PLASTICITY CHART AND DATA

836 San Pablo Avenue
Pinole, California

Client No:	Date:	Report No:
VV3967-004	1/21/2016	0300-013

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/16/2016
Date Submitted 09/12/2016

To: David Cymanski
K.C. Engineering
865 Cotting Lane Suite A
Vacaville, CA 95688

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : VV3967 836 SAN PABLO Site ID : PAD @ 0-5 FT.
Thank you for your business.

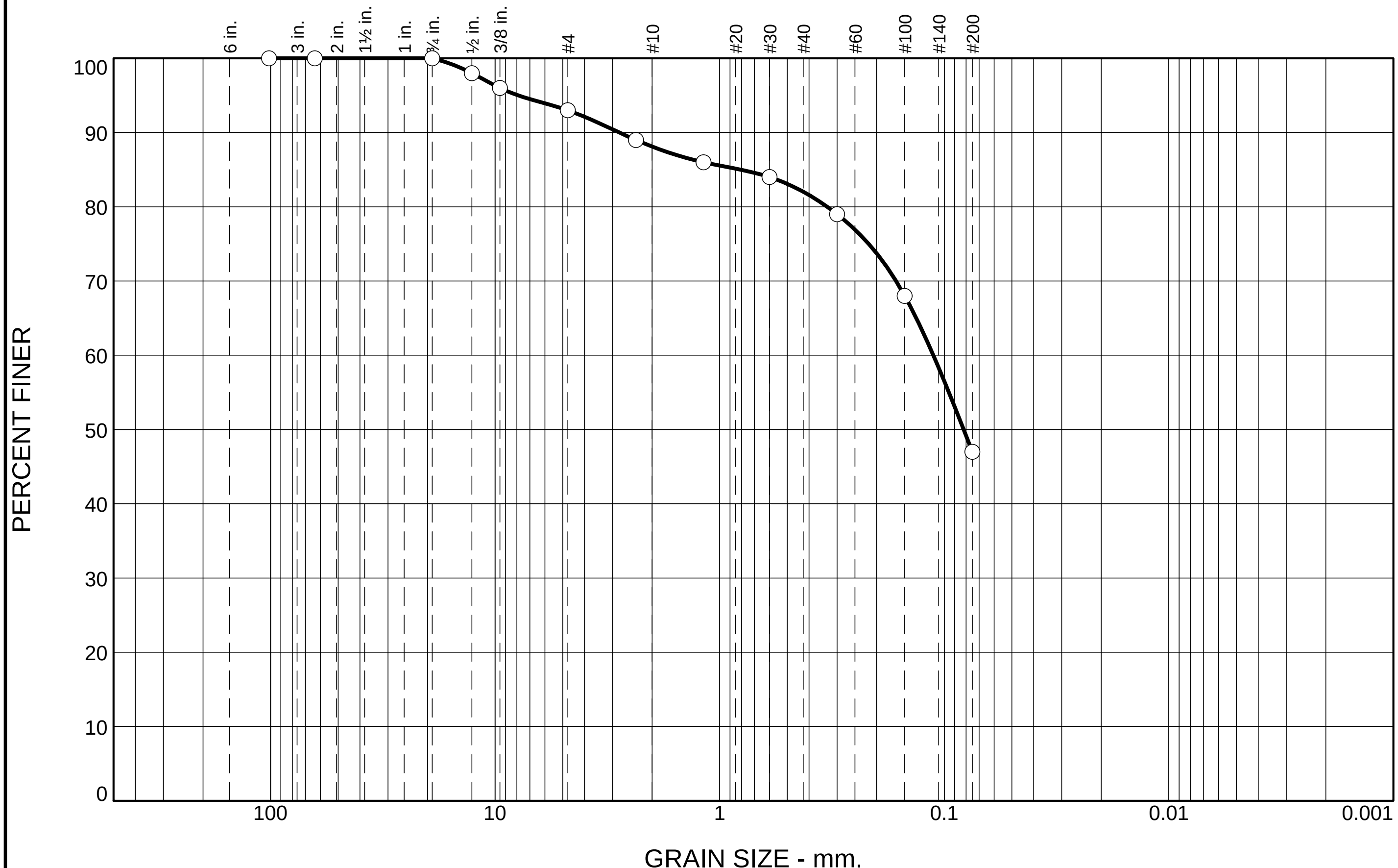
* For future reference to this analysis please use SUN # 72800-151980.

EVALUATION FOR SOIL CORROSION

Soil pH	5.11		
Minimum Resistivity	0.96	ohm-cm (x1000)	
Chloride	56.0 ppm	00.00560	%
Sulfate	186.0 ppm	00.01860	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	7	5	6	35	47	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
4"	100		
2-1/2"	100		
3/4"	100		
1/2"	98		
3/8"	96		
#4	93		
#8	89		
#16	86		
#30	84		
#50	79		
#100	68		
#200	47		

* Fill and Backfill Specifications (City of Pinole)

Material Description

Grayish Brown Clayey Sand

Atterberg Limits

PL= 18 LL= *50 (<40) PI= *32 (<12)

Coefficients

D_{90}= 2.7963 D_{85}= 0.8071 D_{60}= 0.1119
D_{50}= 0.0820 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= ---

Remarks

Sieve tested in accordance with CTM 202.
Material meets grading requirements for Fill and Backfill specifications but does not meet Plasticity Index requirements.

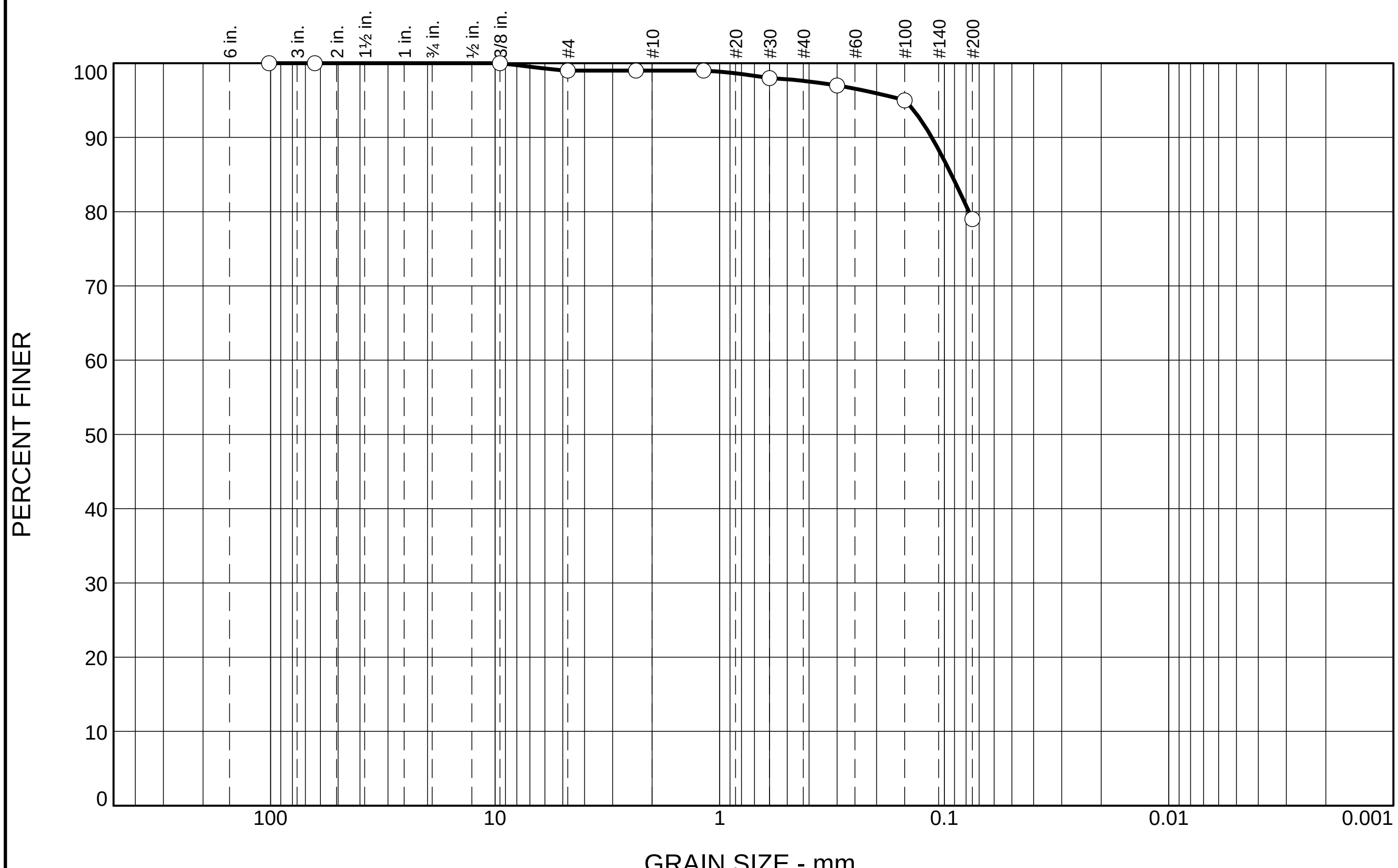
Location: Bulk TP-1
Sample Number: 1 **Depth:** 0-6' **Date:** 09/13/16

Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: Pinole/Hercules Water Pollution Control Plant Upgrades
Pinole, CA
Project No: VV3967-003 **Figure** 0400-001

Import Materials

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	1	0	1	19	79	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
4"	100		
2-1/2"	100		
3/8"	100		
#4	99		
#8	99		
#16	99		
#30	98		
#50	97		
#100	95		
#200	79		

* Fill and Backfill Specifications (City of Pinole)

Material Description

Grayish Brown Clay with Sand

Atterberg Limits

PL= 19 LL= *51 (<40) PI= *32 (<12)

Coefficients

D_{90}= 0.1136 D_{85}= 0.0930 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= ---

Remarks

Sieve tested in accordance with CTM 202.
Material meets grading requirements for Fill and Backfill specifications but does not meet Plasticity Index requirements.

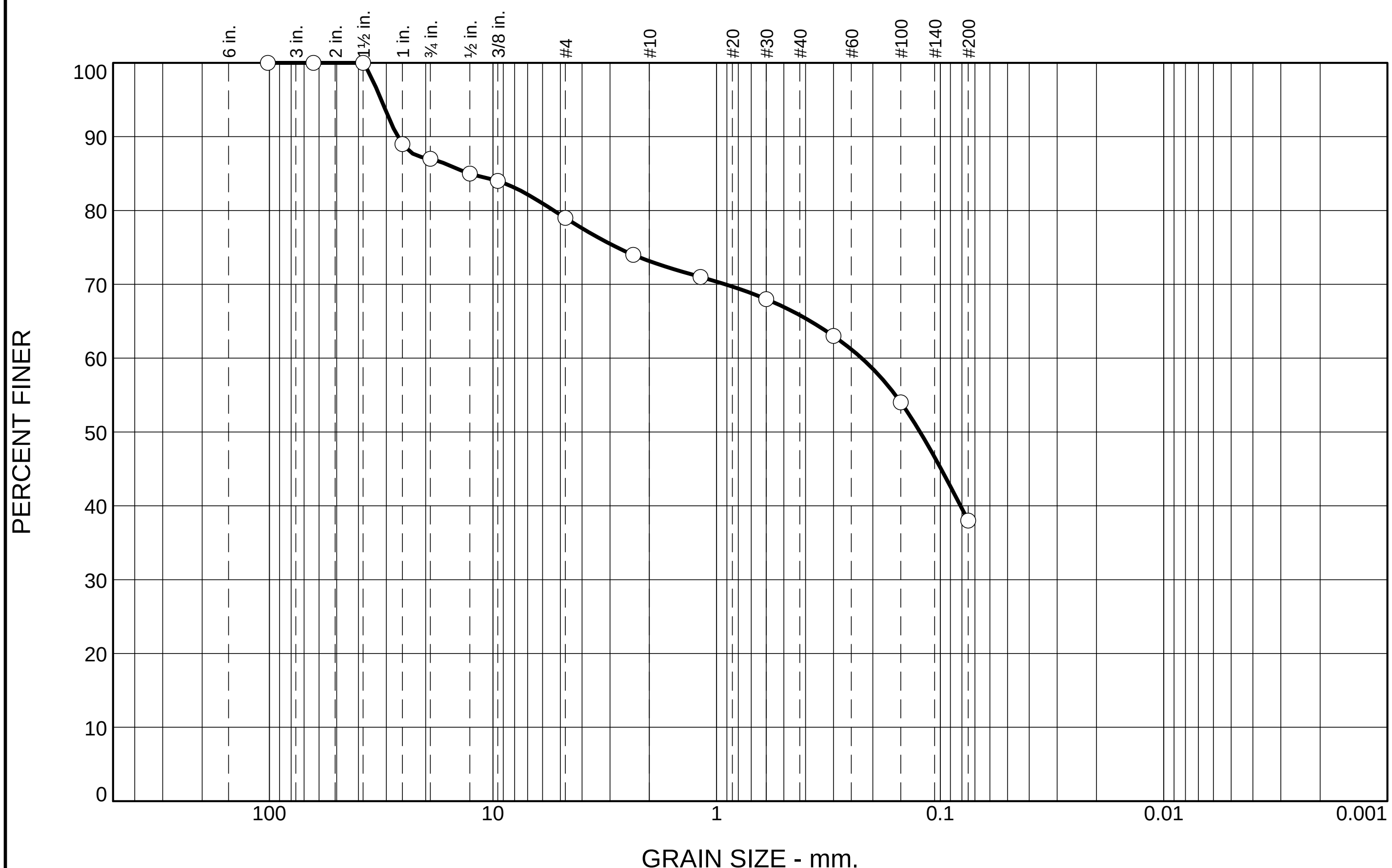
Location: Bulk TP-1
Sample Number: 2 **Depth:** 11-14' **Date:** 09/13/16

Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: Pinole/Hercules Water Pollution Control Plant Upgrades
Pinole, CA
Project No: VV3967-003 **Figure** 0400-002

Import Materials

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	13	8	6	7	28	38	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
4"	100		
2-1/2"	100		
1-1/2"	100		
1"	89		
3/4"	87		
1/2"	85		
3/8"	84		
#4	79		
#8	74		
#16	71		
#30	68		
#50	63		
#100	54		
#200	38		

* Fill and Backfill Specifications (City of Pinole)

Material Description

Grayish Brown Clayey Sand with Gravel

Atterberg Limits

PL= 18 LL= *46 (<40) PI= *28 (<12)

Coefficients

D_{90}= 26.6348 D_{85}= 12.7000 D_{60}= 0.2247
D_{50}= 0.1231 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sieve tested in accordance with CTM 202.
Material meets grading requirements for Fill and Backfill specifications but does not meet Plasticity Index requirements.

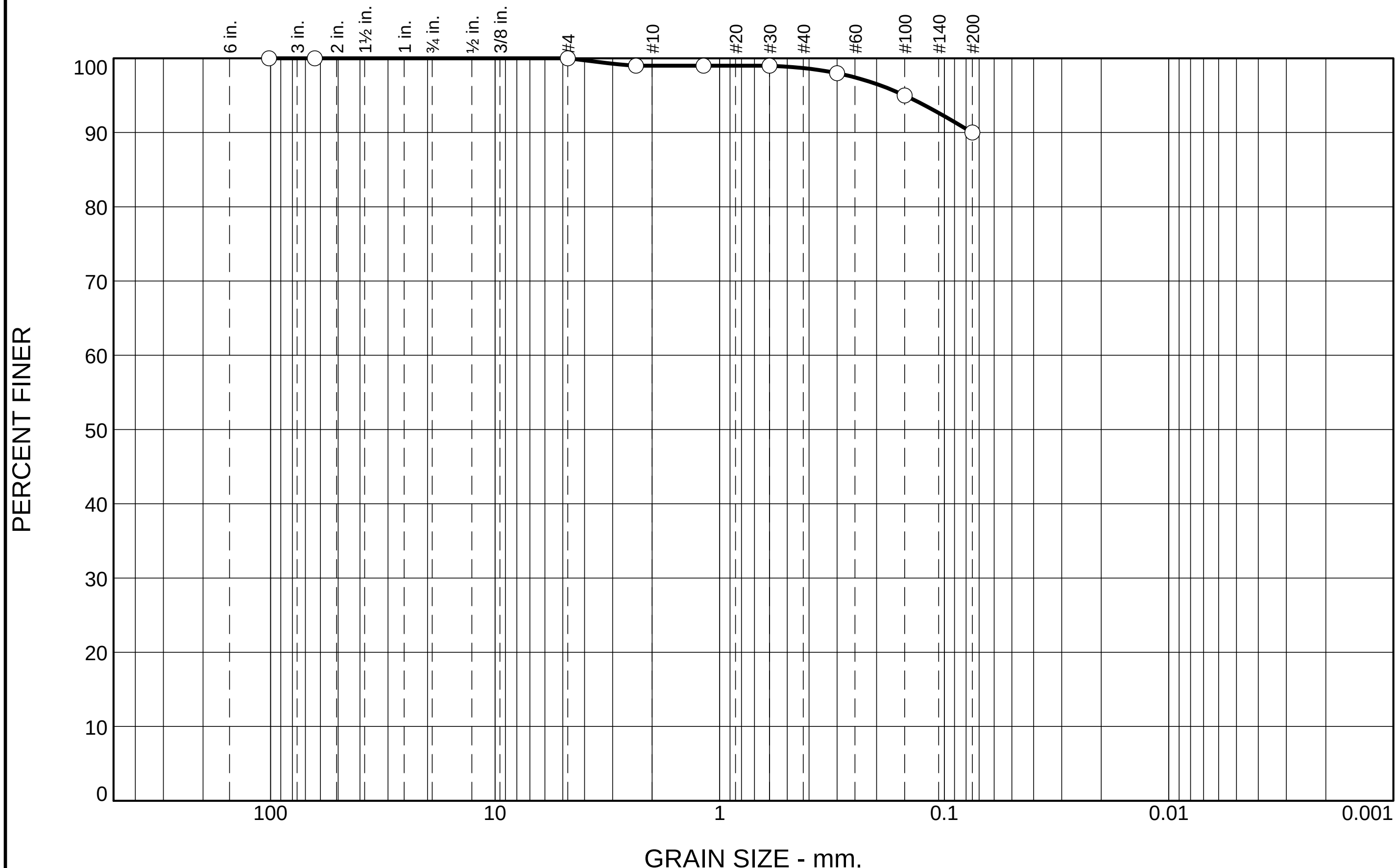
Location: Bulk TP-2
Sample Number: 3 **Depth:** 2-8' **Date:** 09/13/16

M T I Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: Pinole/Hercules Water Pollution Control Plant Upgrades
Pinole, CA
Project No: VV3967-003 **Figure** 0400-003

Import Materials

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	1	0	9	90	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
4"	100		
2-1/2"	100		
#4	100		
#8	99		
#16	99		
#30	99		
#50	98		
#100	95		
#200	90		

* Fill and Backfill Specifications (City of Pinole)

Material Description

Olive Gray Silty Clay

Atterberg Limits

PL= 24 LL= *82 (<40) PI= *58 (<12)

Coefficients

D_{90}= 0.0750 D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= ---

Remarks

Sieve tested in accordance with CTM 202.
Material meets grading requirements for Fill and Backfill specifications but does not meet Plasticity Index requirements.

Location: Bulk TP-2
Sample Number: 4 **Depth:** 12-22' **Date:** 09/13/16

Materials Testing, Inc.

Client: Kiewit Infrastructure West Inc.
Project: Pinole/Hercules Water Pollution Control Plant Upgrades
Pinole, CA
Project No: VV3967-003 **Figure** 0400-004

Import Materials

M T I

Materials Testing, Inc.

8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611

865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Import Materials

Client Kiewit Infrastructure West Inc.
4650 Business Center Drive
Fairfield, CA 94534

Client No. VV3967-003
Report No. 0300-005
Date: 09/13/16

Project: Pinole/Hercules Water Pollution Control Plant Upgrades
Pinole, CA

Submitted By: KC Engineering
Submitted Date: 09/06/16

EXPANSION INDEX
(ASTM D4829)

Sample #:	1
Location:	TP-1 @ 0-6'
Soil Description:	Grayish Brown Clayey Sand
Initial Moisture Content (%):	12.5
Moisture Content after Test (%):	28
Initial Dry Density (lb/ft^3):	94.5
After Test Wet Density (lb/ft^3):	121.2
Degree of Saturation (%):	48.2
Expansion Index:	**27* (20 max)**

Sample #:	2
Location:	TP-1 @ 11-14'
Soil Description:	Grayish Brown Clay with Sand
Initial Moisture Content (%):	12.7
Moisture Content after Test (%):	34
Initial Dry Density (lb/ft^3):	92.7
After Test Wet Density (lb/ft^3):	124.5
Degree of Saturation (%):	48.4
Expansion Index:	**57* (20 max)**

Note: *Does not meet project specifications for Fill and Backfill.

Sample #:	3
Location:	TP-2 @ 2-8’
Soil Description:	Grayish Brown Clay Sand with Gravel
Initial Moisture Content (%):	14.2
Moisture Content after Test (%):	29.8
Initial Dry Density (lb/ft^3):	95.9
After Test Wet Density (lb/ft^3):	124.5
Degree of Saturation (%):	51.0
Expansion Index:	**25* (20 max)**

Sample #:	4
Location:	TP-2 @ 12-22’
Soil Description:	Olive Gray Silty Clay
Initial Moisture Content (%):	18.5
Moisture Content after Test (%):	45
Initial Dry Density (lb/ft^3):	83.7
After Test Wet Density (lb/ft^3):	121.6
Degree of Saturation (%):	49.6
Expansion Index:	**143 (*20 max)**

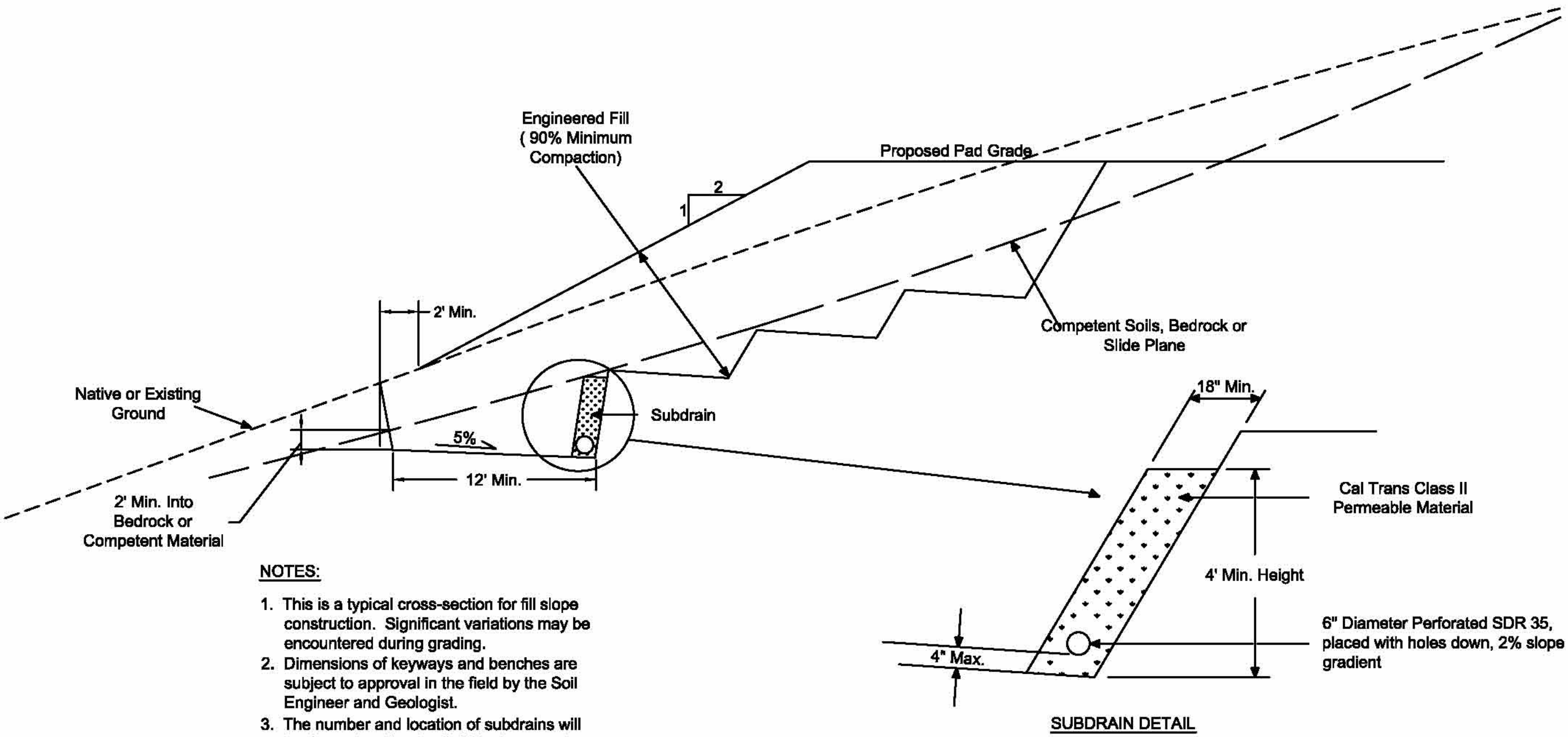
Table 1 Classification of Potential Expansion of Soils Using EI (ASTM D4829-11)

Expansion Index, EI	Potential Expansion
0 - 20	Very Low
21 – 50	Low
51 – 90	Medium
91 – 130	High
>130	Very High

Note: *Does not meet project specifications for Fill and Backfill.

TYPICAL FILL SLOPE & SUBDRAIN DETAILS

(Not to Scale)



NOTES:

1. This is a typical cross-section for fill slope construction. Significant variations may be encountered during grading.
2. Dimensions of keyways and benches are subject to approval in the field by the Soil Engineer and Geologist.
3. The number and location of subdrains will be determined by the Soil Engineer and Geologist in the field.

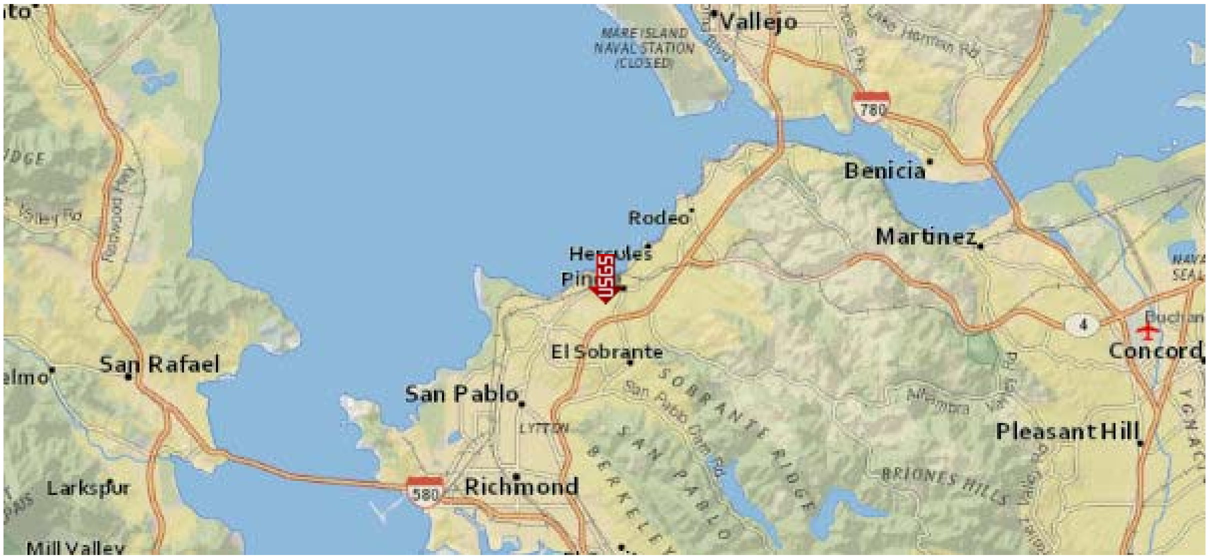
KC ENGINEERING COMPANY
865 Cotting Lane, Suite A
Vacaville, CA 95688

Proposed Hillside Fill Slope
TYPICAL FILL SLOPE, KEYWAY, BENCHING & SUBDRAIN DETAILS

USGS Design Maps Summary Report

User–Specified Input

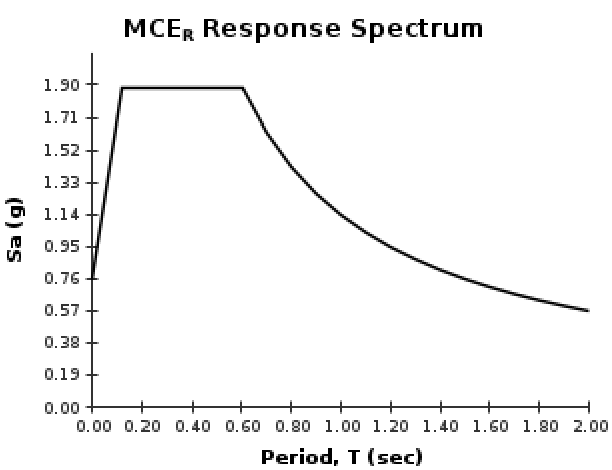
Report Title	836 San Pablo Ave., Pinole Thu September 22, 2016 21:15:40 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	38.0077°N, 122.3073°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

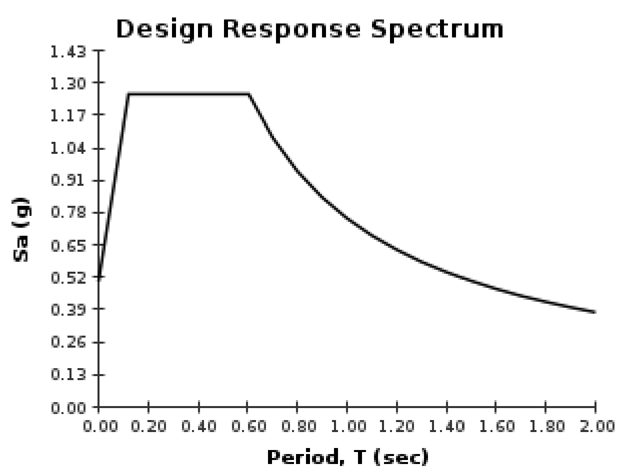


USGS–Provided Output

S_S =	1.882 g	S_{MS} =	1.882 g	S_{DS} =	1.254 g
S_1 =	0.757 g	S_{M1} =	1.136 g	S_{D1} =	0.757 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Design Maps Detailed Report

ASCE 7-10 Standard (38.0077°N, 122.3073°W)

Site Class D – “Stiff Soil”, Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.882 g
From Figure 22-2 [2]	S_1 = 0.757 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.882 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.757 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.882 = 1.882$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.757 = 1.136$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

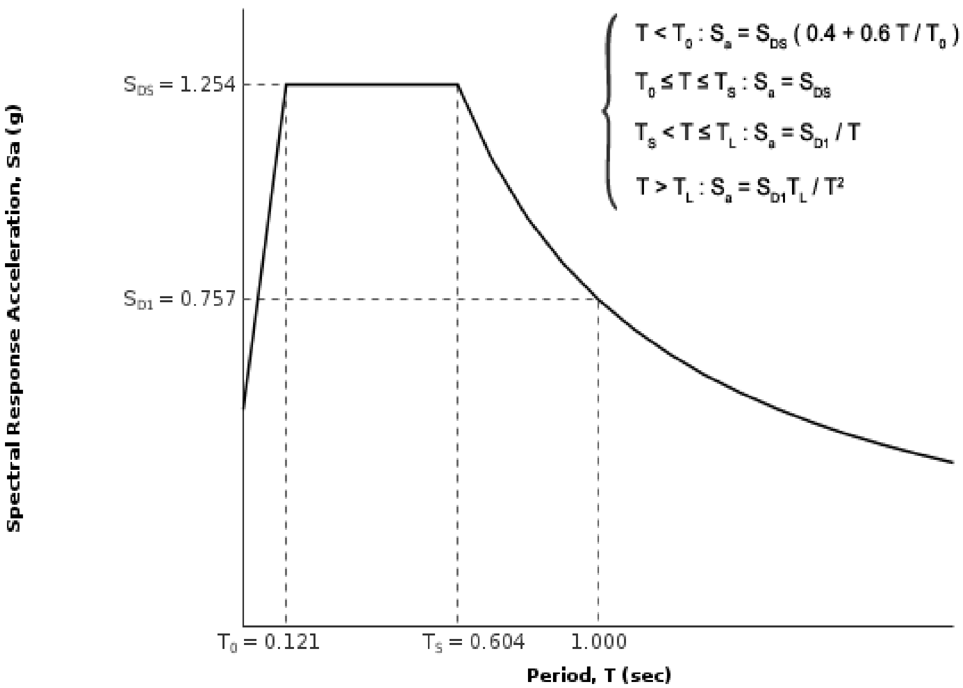
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.882 = 1.254$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.136 = 0.757$ g

Section 11.4.5 — Design Response Spectrum

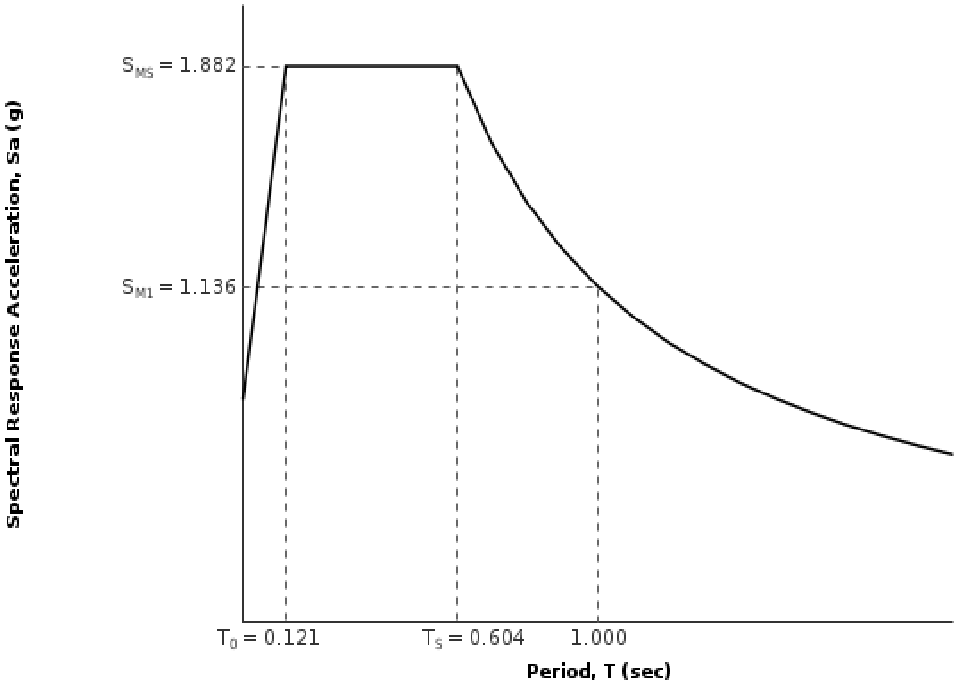
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design
Categories D through F

From Figure 22-7 [4]	PGA = 0.725
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.725 = 0.725$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.725 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 1.017
From Figure 22-18 [6]	C_{R1} = 0.998

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.254 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.757 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is E for buildings in Risk Categories I, II, and III, and F for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf