GREATER VALLEJO RECREATION DISTRICT

395 Amador Street, Vallejo, CA 94590-6320 • 707-648-4600 • FAX 707-648-4616

In compliance with the Americans with Disabilities Act, if you are a disabled person and you need a disability-related modification or accommodation to participate in this meeting, please contact the District Office at 707-648-4604 or fax 707-648-4616. Requests must be made as soon as possible and at least three (3) full business days before the start of the meeting.

Facilities and Development Committee
Directors: Bowen and Salvadori

Agenda
Monday, October 18, 2021
3:00 p.m.
Administration Office – Board Room
395 Amador Street

1. **Presentation – Wilson Avenue Property (Tim Hiemstra)**
2. **Presentation – Field Usage/Condition (Vallejo United Soccer)**
3. **395-401 Amador Street Construction Update**
4. **Hanns Park Disc Golf Course**
5. **Franklin Middle School Lease**
6. **Vallejo Community Center Upgrades**
7. **Cunningham Pool Upgrades**

Next Meeting: Nov 15, 2021

Mission Statement:
Building community and enhancing quality of life through people, parks, and programs.
Website: www.gvrd.org

San Francisco Bay Trail

Look for the Bay Trail sign when out on the trail (photo: Bay Trail Project)

The San Francisco Bay Trail is a visionary plan for a 500-mile walking and bicycling path that will one day allow continuous travel around San Francisco Bay. Over 340 miles of trail are complete in the form of multi-use pathways, levee-top trails, bike lanes, and sidewalks. Eventually, the Bay Trail will link the shoreline of nine counties, passing through 47 cities and crossing seven toll bridges. New segments are constantly opening.

The Bay Trail provides scenic recreation for hikers, joggers, bicyclists, skaters, and wheelchair riders. It also offers a setting for wildlife viewing and environmental education which increases public respect, stewardship, and appreciation for the bay.

The Bay Trail has important transportation benefits: it provides a commute alternative for bicyclists and connects to numerous public transportation facilities, including ferry terminals, light-rail lines, bus stops, Caltrain, Amtrak, and BART.

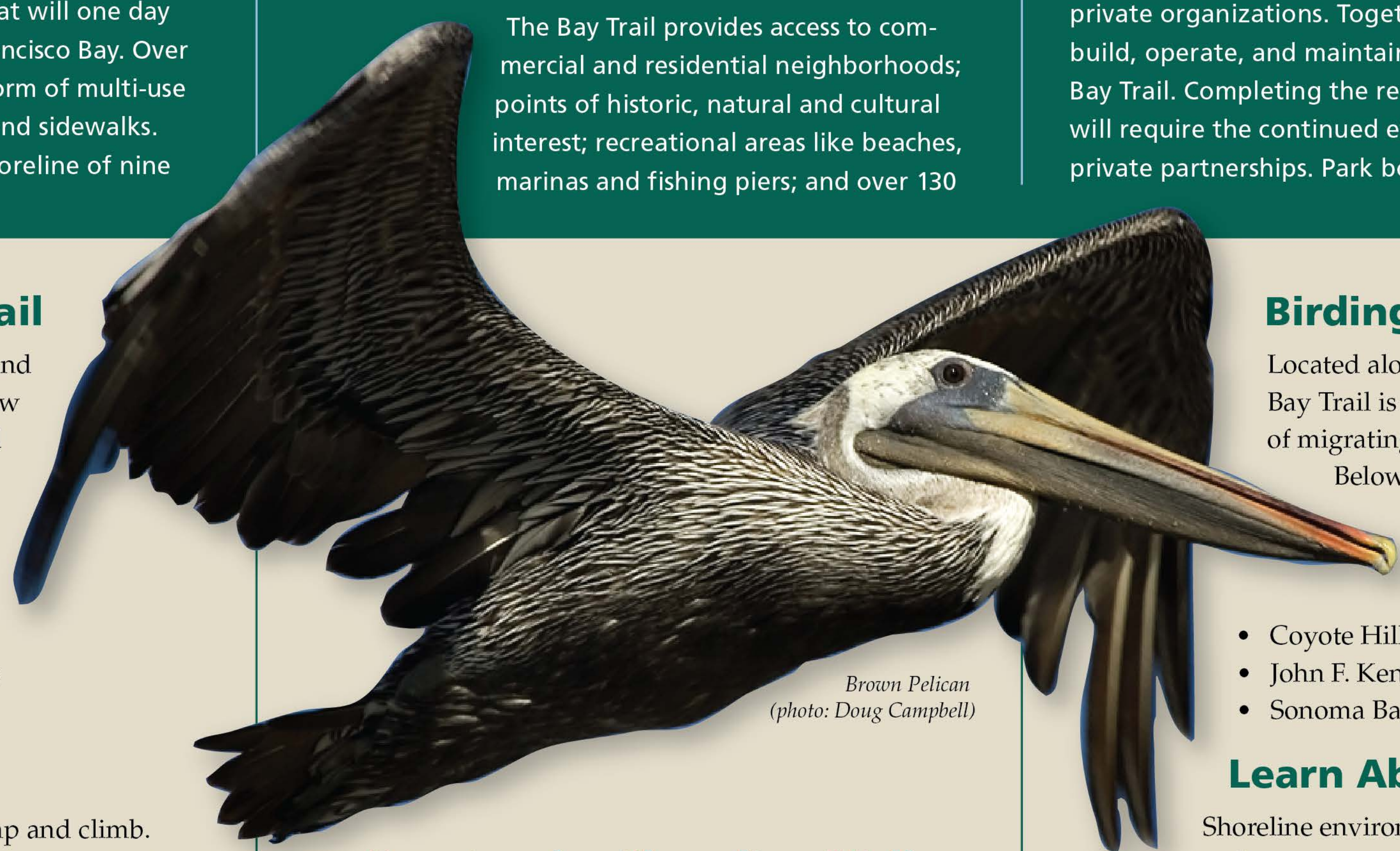
The Bay Trail provides access to commercial and residential neighborhoods; points of historic, natural and cultural interest; recreational areas like beaches, marinas and fishing piers; and over 130 parks totaling more than 57,000 acres of open space.

Implementation of the Bay Trail is coordinated by the San Francisco Bay Trail Project, a nonprofit organization created by the Association of Bay Area Governments and housed at its offices. To carry out its mission, the Bay Trail Project provides grants for trail planning and construction, ensures consistency with the adopted Bay Trail Plan, provides technical assistance, enlists public participation in trail-related activities, and publicizes the Bay Trail and its benefits to the region.

Making the Bay Trail a reality involves a collaborative effort between a wide-variety of public agencies and private organizations. Together, these organizations build, operate, and maintain the various parts of the Bay Trail. Completing the remaining miles of Bay Trail will require the continued efforts of these public and private partnerships. Park bonds, transportation funding measures, private funds, and many other sources all contribute to the Bay Trail vision.

Everyone can help complete the Bay Trail by supporting projects that will build new Bay Trail segments, voting for park bonds and transportation measures that help fund the Bay Trail, and donating to the Bay Trail Project.

Point Pinole Regional Shoreline, Richmond (photo: Josh Maddox)



Brown Pelican (photo: Doug Campbell)

Views from the Bay Trail

Take in views of gleaming bay waters and rediscover San Francisco Bay from a new perspective. Venture to these areas, and so many others, for rewarding vistas:

- Crissy Field, San Francisco
- Middle Harbor Shoreline Park, Oakland
- Cesar Chavez Park, Berkeley
- George Miller Trail, Carquinez Strait
- Tiburon Pathway

Great Places for Kids

Adventures await kids who love to jump and climb. Kids will love these creative playgrounds along the bay:

- Magic Mountain Playground, Coyote Point Recreation Area, San Mateo
- Ryder Park, San Mateo
- Marina Park, San Leandro
- Adventure Playground, Berkeley
- Pickleweed Park, San Rafael

Exceptional Bike Rides

Ride your bicycle for miles on trails that meander along the bay shoreline. These rides will take you to familiar sites and places off the beaten path:

- San Francisco Ferry Building to Sausalito via the Golden Gate Bridge, ferry back to San Francisco
- Coyote Point Park to Redwood Shores
- San Leandro Marina to Hayward Regional Shoreline
- Point Isabel Regional Shoreline to Lucretia Edwards Park, Richmond
- Mill Valley-Sausalito Pathway

Spectacular Shoreline Walks

There is much to discover along the bay edge. What better way to explore than by foot? Travel at your own pace and enjoy the sights and sounds of San Francisco Bay:

- San Francisco waterfront
- South San Francisco shoreline
- Martin Luther King Jr. Regional Shoreline, Oakland
- Emeryville Waterfront to Berkeley Marina
- Glen Cove Waterfront Park, Vallejo

Dog Parks

Before inviting Fido on your sojourn, be sure to respect trail regulations regarding dogs. Some parts of the Bay Trail are off-limits to dogs, and most sections require dogs to be on leash. These are some dog-friendly parks:

- Bayside Park, Burlingame
- Point Isabel Regional Shoreline, Richmond
- Bayfront Park, Mill Valley

Birding the Bay Trail

Located along the Pacific Flyway, the Bay Trail is the place to view an array of migrating and resident shorebirds. Below are a few noteworthy spots:

- Palo Alto Baylands
- Presidio of San Francisco
- Coyote Hills Regional Park, Fremont
- John F. Kennedy Memorial Park, Napa
- Sonoma Baylands, Sonoma County

Learn About the Bay

Shoreline environmental education centers and museums inspire people of all ages to learn about wetland ecology. Visit these centers, and others, along the Bay Trail:

- Redwood Shores Branch Library
- Lucy Evans Baylands Nature Interpretive Center, Palo Alto
- Alviso Environmental Education Center, San Jose
- Hayward Shoreline Interpretive Center, Hayward
- Crab Cove Visitor Center, Alameda

Shoreline Picnic and Barbecue Spots

Pack a lunch and head out to the trail. These parks are jewels of bayside open space for the whole family to enjoy:

- Crissy Field, San Francisco
- Robert Crown Memorial State Beach, Alameda
- Miller-Knox Regional Shoreline, Richmond
- Cesar Chavez Park, Berkeley
- China Camp State Park, San Rafael

Aquatic Park, San Francisco (photo: Wilfred J. Jones)

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Association of Bay Area Governments (ABAG)
www.baytrail.org
Produced by Lohnes+Wright, Oakland

SAN FRANCISCO Bay Trail

The San Francisco Bay Trail is a 500-mile shoreline walking and bicycling path that will one day encircle the entire bay. If you have ever cycled across the Golden Gate Bridge, skated around Bay Farm Island, or walked in the Palo Alto Baylands, you have experienced the Bay Trail. Made up of paved multi-use paths, dirt trails, bike lanes, and sidewalks, the trail passes through urbanized areas like downtown San Francisco as well as remote natural areas like the Don Edwards San Francisco Bay National Wildlife Refuge. With over 340 miles in place, the Bay Trail is a collaborative effort between a wide-variety of public agencies and private organizations. Together, these organizations build, operate, and maintain the various parts of the Bay Trail.

Love for the Bay Trail starts early at Cesar Chavez Park (photo: Bay Trail Project)

Views of the Bay (photo: Don Weden)

Golden Gate Promenade at Crissy Field (photo: Will Elder)

Moffett Bay Trail (photo: Ron Horii)

San Francisco Bay Trail

Association of Bay Area Governments

The Association of Bay Area Governments (ABAG) is the regional planning agency and council of governments for the nine counties and 101 cities and towns of the San Francisco Bay region, thus making it the perfect home for the San Francisco Bay Trail Project. ABAG's programs work to address regional economic, social, and environmental challenges.

Metropolitan Transportation Commission

The Metropolitan Transportation Commission (MTC) is the transportation planning, coordinating, and financing agency for the nine-county Bay Area. The Bay Trail serves as the backbone of MTC's Regional Bicycle Network, and MTC provides funding for the development of the San Francisco Bay Trail.

Coastal Conservancy

The State Coastal Conservancy is a principal funding source for the development of the Bay Trail. Established in 1976, the Conservancy is a State agency that uses entrepreneurial techniques to protect and improve coastal and Bay Area natural resources and to provide public access to the shore.

Sonoma Baylands (photo: Stephen Joseph)

Disclaimer: This map reflects conditions known to its makers at the time of its creation, and reasonable steps have been taken to ensure its accuracy. Changes to the built and natural

the San Francisco Bay Trail Project makes any guarantees about trail conditions or assumes any liability for any injury or damage arising out of, or in connection with, any use of

regional Bay Trail maps – a set of 25 cards containing detailed maps and descriptions of individual segments of the Bay Trail. These maps can be purchased at local stores or via the Bay

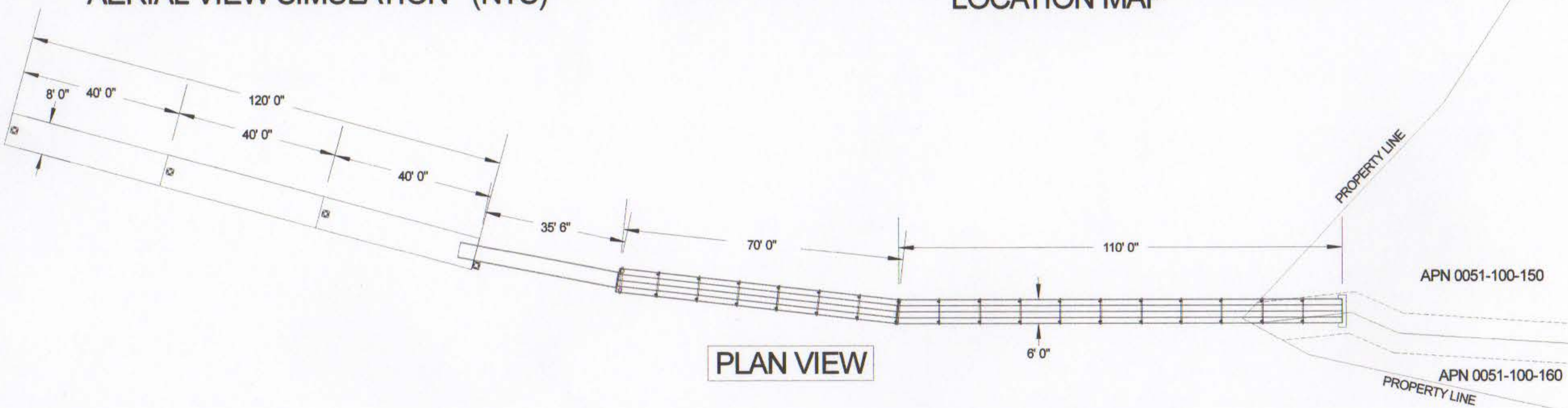
www.baytrail.org

Over 340 miles of adventure by foot or by wheel

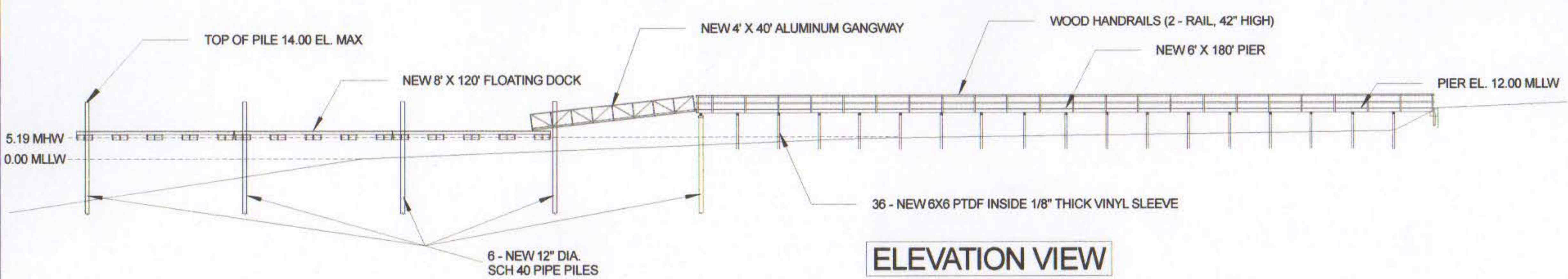

SAN FRANCISCO
Bay Trail
San Francisco Bay Trail
Paved
Dirt/Gravel
On Street
Planned
Other Regional Trails
Existing
Planned
0
2.5
5 miles
San Francisco
Bay Trail
Over 340 miles of adventure by foot or by wheel
San Pablo Bay
San Francisco Bay
Pacific Ocean
SONOMA
NAPA
PETALUMA
NOVATO
VALLEJO
BENICIA
FAIRFIELD
SUISUN CITY
MARTINEZ
RICHMOND
BERKELEY
OAKLAND
ALAMEDA
SAN FRANCISCO
DALY CITY
SAN MATEO
HAYWARD
FREMONT
SAN JOSE
SAUSALITO
TIBURON
SAN RAFAEL

AERIAL VIEW SIMULATION (NTS)

LOCATION MAP



PLAN VIEW



ELEVATION VIEW

Drawing Index

P1 - Title page, location map, simulated aerial photo, plan view, profile view.

P2 - Dock and Pie details and job site photos.

Project Description

1. Remove the old existing pier and piles and replace, in the same location, a new 6'x180' residential pier (1080 sf), one 8'x120' floating dock, one 4'x40 al gangway
2. Drive 6 - 12" HDG steel piles
3. Drive 36 - 6"X6" PTDF post enclosed with 1/8" thick vinyl sleeve extrusion.
4. Pier and Dock constructed with cedar wood frame and ThruFlow decking (50% + light transmittance).

Project Name

SCOT SHOEMAKER
APN 0051-100-150 & 160
913 & 915 Wilson Ave.
Vallejo, Ca. 94590
phone# 415-332-8529
e-mail - shoeatlaw@aol.com

Contractor & Agent

MID-CAL CONSTRUCTION INC.
2716 E. Miner Ave., Suite S
Stockton, Ca. 95205
phone # 209-832-4400
fax # 209-955-8022

Date Approved	Approved By	
Sheet No. P1 of 2	Date 02/02/13	Drawn by Rick Pelchat
Scale 1" = 30'	Job #	

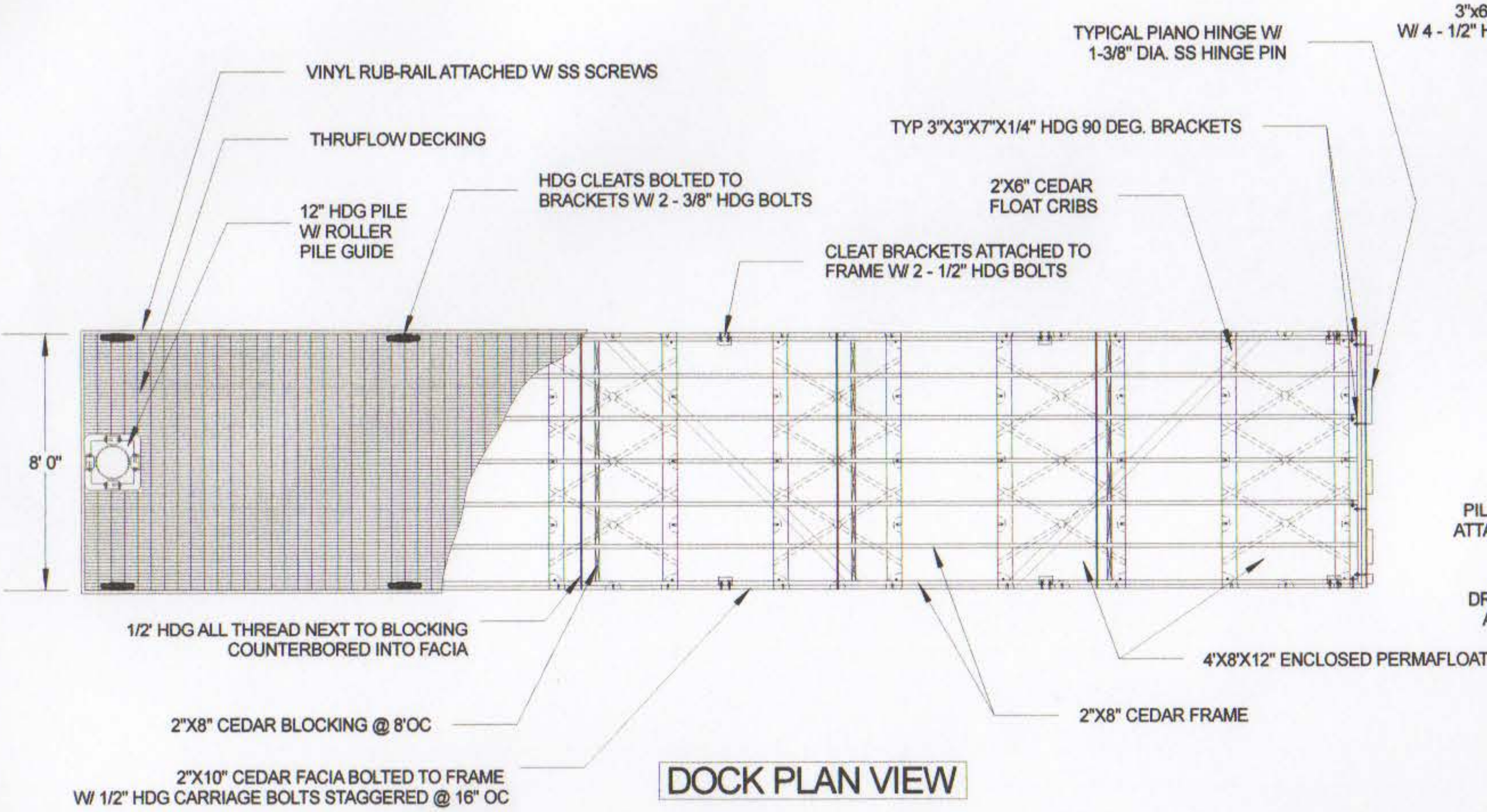
AERIAL PHOTO SHOWING THE EXISTING PIER AND DOCK LOCATION IN 2004.

LOOKING WEST FROM THE SHORE, WHERE THE PIER BEGINS

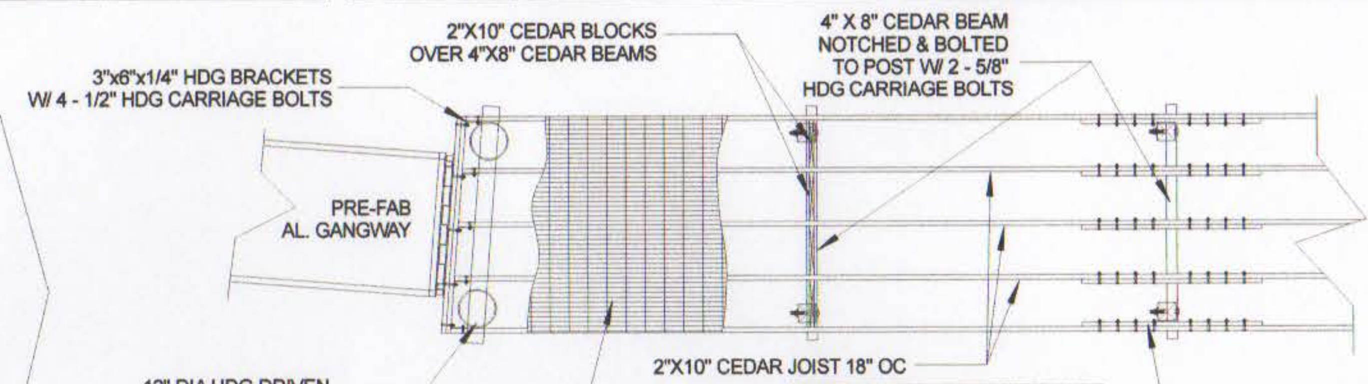
LOOKING NORTH FROM WHERE THE PIER BEGINS

LOOKING SOUTH FROM WHERE THE PIER BEGINS

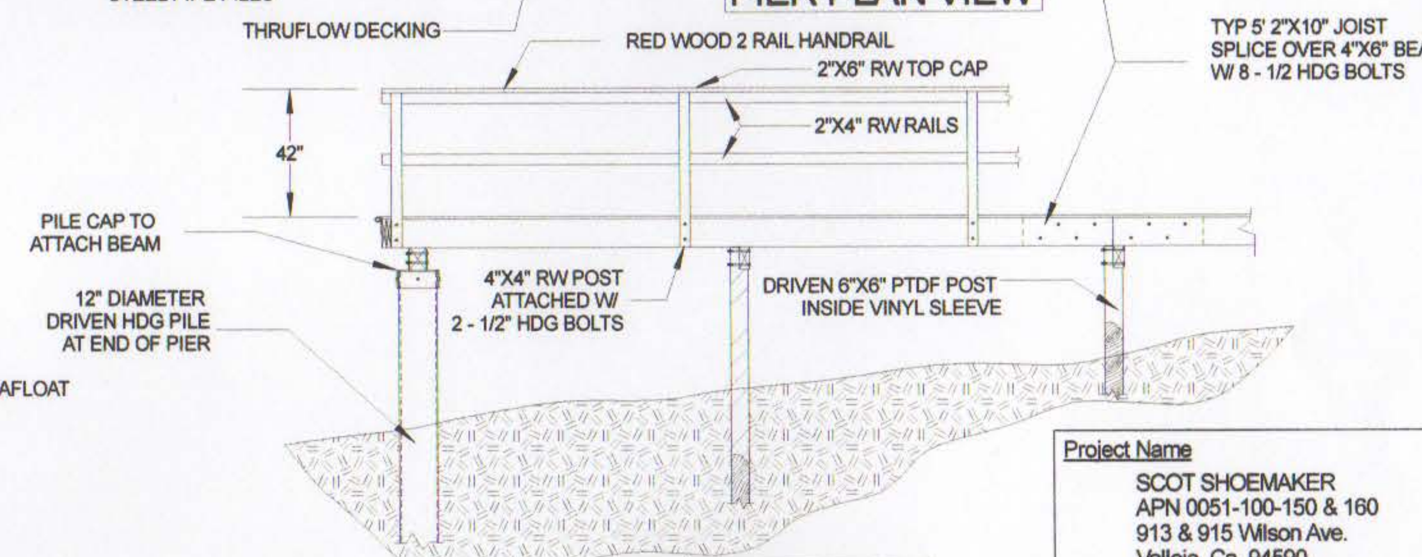
LOOKING EAST FROM WHERE THE PIER BEGINS



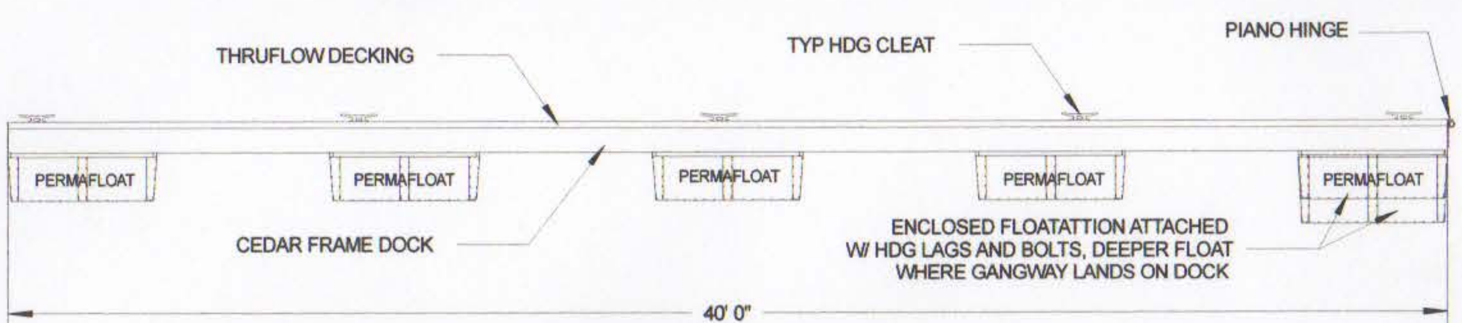
DOCK PLAN VIEW



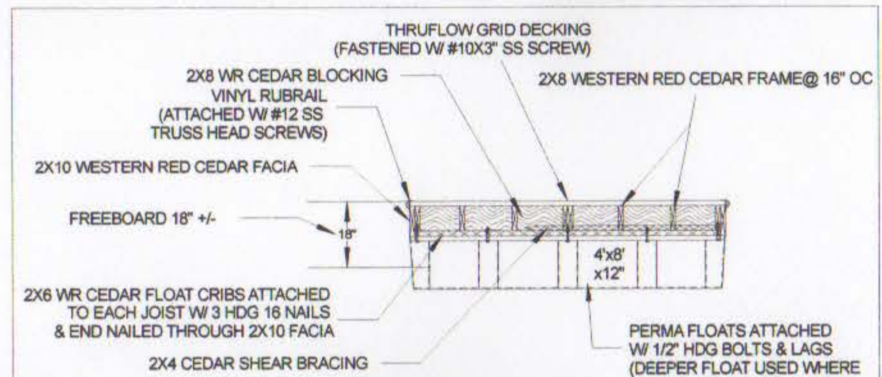
PIER PLAN VIEW



PIER ELEVATION



DOCK ELEVATION



DOCK SECTION nts

Project Name

SCOT SHOEMAKER
APN 0051-100-150 & 160
913 & 915 Wilson Ave.
Vallejo, Ca. 94590
phone# 415-332-8529
e-mail - shoeatlaw@aol.com

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Stockton, Ca. 95205
phone # 209-832-4400
fax # 209-955-8022

Date Approved	Approved By	
Sheet No. P2 of 2	Date 02/02/14	Drawn by Rick Pelchat
Scale 3/16" = 1'	Job #	

BASIS OF BEARINGS

THE BASIS OF BEARINGS FOR THIS MAP TAKEN FROM 28 SURVEYS 31-38 (CCS ZONE II, NAD 83)

(THE BEARINGS SHOWN ON 30 PM 56 ROTATED 1° 13' 13" COUNTERCLOCKWISE.)

BENCHMARK

CITY CONTROL MONUMENT # 9650 AT THE INETERSECTION OF SACRAMENTO STREET AND COGHLAN STREET.
BENCHMARK ELEV= 35.33' (NAVD88)

NOTES

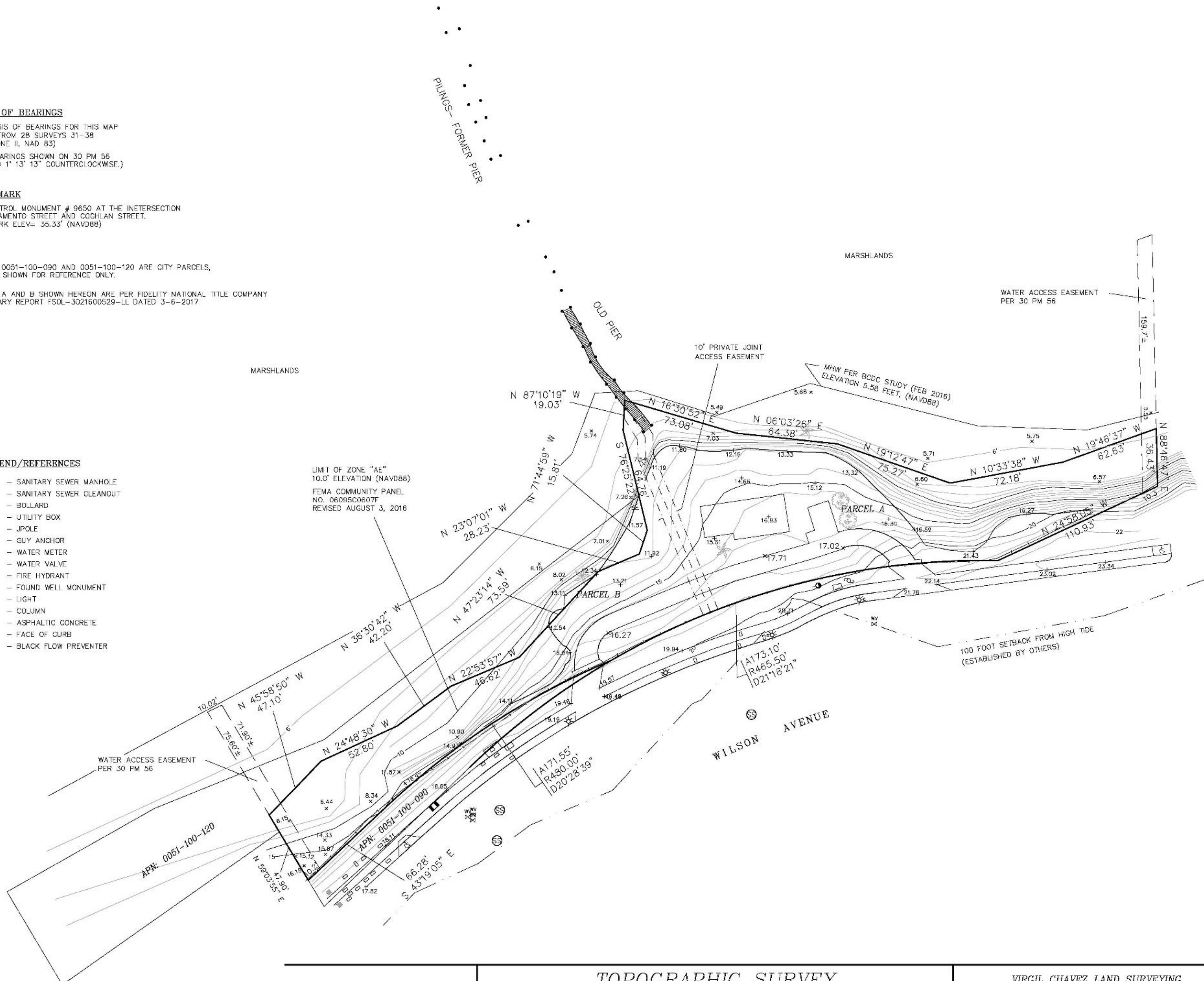
PARCELS 0051-100-090 AND 0051-100-120 ARE CITY PARCELS, AND ARE SHOWN FOR REFERENCE ONLY.

PARCELS A AND B SHOWN HEREON ARE PER FIDELITY NATIONAL TITLE COMPANY PRELIMINARY REPORT FSOL-3021600529-LL DATED 3-6-2017

LEGEND/REFERENCES

- SANITARY SEWER MANHOLE
- SANITARY SEWER CLEANOUT
- BOLLARD
- UTILITY BOX
- JPOLE
- GUY ANCHOR
- WATER METER
- WATER VALVE
- FIRE HYDRANT
- FOUND WELL MONUMENT
- LIGHT
- COLUMN

A/C - ASPHALTIC CONCRETE
F/C - FACE OF CURB
BFP - BLACK FLOW PREVENTER



TOPOGRAPHIC SURVEY

913 AND 915 WILSON AVENUE
LOCATED IN THE CITY OF VALLEJO
COUNTY OF SOLANO, STATE OF CALIFORNIA

VIRGIL CHAVEZ LAND SURVEYING
721 TUOLUMNE STREET, VALLEJO, CALIF. 94590
PHONE: (707) 553-2476

DATE:	MARCH, 2020		
DRAWN BY:	RRS		
SCALE:	1"= 30'		
CHECKED:	VC		
PROJ. MGR:	VC	DC FILE:	392600
PROJ. NO. 3926-00		CAD FILE:	392600

SHEET NO. 1 OF 1 SHEETS

GEOTECHNICAL INVESTIGATION
on
PROPOSED CUSTOM RESIDENCES
913 and 915 Wilson Avenue
Vallejo, California
for
H&B DEVELOPERS

By

KC ENGINEERING COMPANY

Project No. VV1558-04

8 September 2004

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

8798 Airport Road
Redding, California 96002
(530) 222-0832, fax 222-1611

GEOTECHNICAL ENGINEERING CONSULTANTS

KC ENGINEERING COMPANY
A SUBSIDIARY OF MATERIALS TESTING, INC.

Project No. VV1558-04
8 September 2004

H&B Developers
c/o Mr. Robert Karn
Robert A. Karn & Associates
707 Beck Avenue
Fairfield, California 94533

Subject: Proposed Custom Residences
913 and 915 Wilson Avenue
Vallejo, California
GEOTECHNICAL INVESTIGATION

Dear Mr. Karn:

In accordance with your authorization, **KC ENGINEERING COMPANY** has investigated the geotechnical conditions of the surface and subsurface soils at the subject site of the proposed new custom residences to be constructed at 913 and 915 Wilson Avenue in Vallejo, California.

The accompanying report presents our conclusions and recommendations based on our investigation. Our findings indicate that the proposed residences are feasible for construction, from a geotechnical standpoint, 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.

Respectfully Submitted,
KC ENGINEERING COMPANY

Reviewed By;

David V. Cymanski, G.E.
Principal Engineer

Jerry S. Pascoe, G.E.
Senior Engineer

Copies: 6 to Client

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GEOTECHNICAL INVESTIGATION

Purpose and Scope

The purpose of the geotechnical investigation for the proposed new custom residences to be constructed at 913 and 915 Wilson Avenue in Vallejo, California, was to determine the surface and subsurface soil conditions at the subject site. Based on the results of the investigation, geotechnical criteria were established for the grading of the site, the design of foundations for the proposed residences, and the construction of other related facilities on the property.

In accordance with our proposal dated 5 August 2004, our investigation services included the following tasks:

a. A review of available geotechnical and geologic literature concerning the site and vicinity;
b. Site reconnaissance by the Soil Engineer to map the surface conditions;
c. Drilling of three exploratory borings and the sampling of the subsurface soils;
d. Laboratory testing of the samples obtained to determine their 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 site is located in the northwestern portion of the City of Vallejo on the west side of Wilson Avenue just south of Highway 37 as shown in Figure 1, Vicinity Map. The irregular-shaped site is comprised of two parcels of land, APN's 051-100-150 and 160, totaling 0.83 acres. The site is bounded on the east by Wilson Avenue and on the remaining sides by marshlands of the Napa River. The site was created by placing import fill over the marshland deposits. Topographically, the site is located approximately 3 feet lower in elevation than Wilson Avenue and continues to gently slope down towards the river. At the river's edge, a 6 to 7 feet high steep fill slope extends down to the natural marsh deposits. There is an abandoned one story residence on the central portion of the site that is understood to have been constructed in the early 1940's. The main house structure is founded on a perimeter continuous footing with interior isolated piers and a raised wood floor. An addition was constructed to the main house at the rear middle portion and is supported on timber piers extending below grade. A rectangular-shaped cesspool is located at the rear middle portion of the house, and is partially beneath the addition. A square-shaped rear middle portion of the house, and is partially beneath the addition. A square-shaped concrete pad is located just northeast of the northeast house corner. A wood deck is located behind the residence and extends beyond the edge of the fill slope. In addition, the remnants of an old wood pier extends from the middle of the site out to the river. With the exception of the area around the residence, the site is densely vegetated with trees, bushes, ivy and weeds. Along the eastern property line, southeast of the residence, we observed a "hole" in the ground that had previously been covered by wood planks. Over the wood planks was a layer of soil approximately 9 inches deep. As noted by surface deflection, the wood planks extend over an area approximately 10 ft. by 10 ft. The nature of this subsurface "structure" is unknown at this time.

The above description is based on a reconnaissance of the site by the Geotechnical Engineer, on a Topographic Map prepared by Robert A. Karn & Associates, Inc. dated 23 February 2002 and on the USGS Topographic Map of the Mare Island Quadrangle as obtained from the 3D TopoQuads program by DeLorme. The Topographic Map is the basis for our "Site Plan" included as Figure 2 in the Appendix.

Proposed Development

The proposed project will consist of demolishing the existing residence and associated improvements. A lot line adjustment will be performed between the two lots. A new one and/or two story, wood and/or steel frame residence will be constructed on each lot. Minimal grading (less than 2 feet of cut and/or fill) is expected to achieve the design grades.

Field Investigation

The field investigation was performed on 18 August 2004 and included a reconnaissance of the site and the drilling of three exploratory test borings at the approximate locations shown on Figure No. 2, "Site Plan" included in the Appendix.

The borings were drilled to a maximum depth of 31 feet below the existing ground surface. The drilling was performed with a Mobile B24 rig using power-driven, 4-inch diameter continuous flight solid augers. Visual classifications were made from the 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., split-tube sampler, containing thin brass liners, into the boring bottom. Disturbed samples were obtained by driving a 2-inch O.D., split-barrel sampler into the boring bottom in accordance with ASTM D1586. The sampler was driven into the in-situ soils 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 were adjusted to the standard penetration resistance (N-Value). When the sampler was withdrawn from the boring bottom, the brass liners containing the relatively undisturbed samples were removed, examined for identification purposes, labeled and sealed to preserve the natural or in-situ moisture content. The samples were then transported to our laboratory for testing. 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 "Logs of Test Borings" contained within the Appendix.

Laboratory Investigation

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. A summary of all laboratory test results is presented 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.

The strength parameters of the foundation soils were determined from unconfined compression tests (ASTM D2166) performed on selected relatively undisturbed soil samples. Standard field penetration resistance (N-Values) also assisted in the determination of strength and bearing capacity. The standard penetration resistance values are recorded on the respective "Logs of Test Borings".

One sieve analysis test (ASTM C136) was performed on a selected soil sample to assist in the identification and classification of the subsurface soils. The expansion potential and classification of the near surface soils were evaluated by means of two Atterberg Limits Tests (ASTM D4318).

Subsurface Conditions

Based on our field exploration and laboratory investigation, the subsurface soil profile was found to consist of 10 to 11.5 feet of fill overlying 3.5 to 8 feet of native soil further overlying bedrock. The fill was found to consist of olive brown to red and grey stiff to very stiff, silty and sandy clay with varying amounts of mudstone and sandstone fragments. The fill was found to be moderately expansive. In Boring 2, significant red brick debris was encountered within the fill between 4 and 10.5 feet. In addition, at a depth of 7 feet, concrete debris was observed. The native soils generally consist of dark grey/black wet and soft to firm clay that is locally known as Bay Mud.

Bedrock was encountered beneath the fill and native soils at depths ranging from 16 to 19 feet below the ground surface. The bedrock consists of red brown and grey mudstone with interbedded sandstone that was found to be weak, moderately to highly weathered and closely to intensely fractured.

Groundwater was in all three borings at depths ranging from 7.5 to 11 feet below the existing ground surface or at elevations ranging from approximately –3 to –3.5 feet mean sea level. The groundwater level will fluctuate with the tides.

A more thorough description and stratification of the soils encountered along with the results of the laboratory tests are presented on the "Logs of Test Borings" in the Appendix. The approximate location of the boring is shown on Figure 2, "Site Plan," in the Appendix.

Site Geology & Seismicity

According to the USGS Open File Report 86-17[1], the geologic deposits underlying the site consist of Holocene-aged intertidal deposits of soft mud and peat locally known as Bay Mud. Located across the street to the east are low lying hills that are underlain by Lower Cretaceous and Upper Jurassic Sedimentary Rocks locally known as the Panoche Formation. These deposits consist of undifferentiated marine sandstone, mudstone, and minor conglomerate. The materials encountered and observed on the site correlate with those mapped on the site and vicinity.

The site is not located within an Alquist-Priolo Special Studies Zone. There are no known active or inactive faults crossing the site as mapped and/or recognized by the State of California. However, the entire Bay Area is considered to be a seismically-active region. Earthquake related ground shaking should be expected during the design life of structures constructed on the site. Earthquake related ground shaking should be expected during the design life of the structures at the site. The California Geological Survey (formerly the CDMG) 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[2], the nearest active faults are the West Napa, Rodgers Creek, Hayward and Green Valley Faults located approximately 3.5 miles northeast, 4.1 miles west, 8.5 miles southwest, and 8.9 miles east of the site, respectively. Various other faults in the area may produce seismic shaking at the site. Using an attenuation relationship developed by Sadigh et al. (1997) and the EQFAULT program by

[1] Bortugno, Edward J., 1987, *Landslide Hazards in the Vallejo-Vallejo Area, Solano County, California*, Landslide Hazard Identification Map No. 8, California Division of Mines and Geology, Open File Report 86-17.
[2] Jennings, Charles W., 1994, *Fault Activity Map of California and Adjacent Areas*, California Division of Mines and Geology Geologic Map Data Series, Map No. 6.

Blake (1994), a maximum peak ground acceleration of 0.46g (deterministic) was calculated for the subject site. This acceleration was computed based on an earthquake moment magnitude of M7.0 on the Rodgers Creek Fault. However, based on the Interactive Probabilistic Seismic Hazard Map on the CGS website, the peak ground acceleration that has a 10% probability of exceedance in 50 years is 0.47g. The structures at the site should be designed in accordance with the 2001 California Building Code to withstand the anticipated ground accelerations.

Earthquake Design Criteria

The 2001 California Building Code (CBC) Chapter 16, Division IV Earthquake Design requires that structures be designed using certain earthquake design criteria. The criteria are based in part on the seismic zone, soil profile and the proximity of the site to active seismic sources (faults). During an earthquake event, structures located very close to active faults can be subjected to near source energy motions that may be damaging to structures, if the effects of these energy motions are not considered in the structural design. The CBC indicates that the types of seismic sources (active faults) that generate near source (N_a and N_v) factors greater than 1.0 are classified as Type A or Type B. In 1998, the International Conference of Building Officials (ICBO) published a map folio to be used in scaling distances to the Type A or Type B faults. According to this map folio, and the information from published maps and the EQFAULT program, the nearest fault is the Type B Fault is the West Napa while the nearest Type A is the Rodgers Creek Fault. Based on our review of published maps and the probabilistic ground motion parameters from the CGS website, the following 2001 California Building Code earthquake design criteria should be used by the Structural Engineer:

Soil Profile Type:	S_D
Seismic Zone:	4
Seismic Zone Factor:	0.40
Seismic Source Type:	A
Seismic Coefficients:	$C_a = 0.44N_a$, $C_v = 0.64N_v$
Near Source Factors:	$N_a = 1.14$; $N_v = 1.47$

Liquefaction Potential Evaluation

Soil liquefaction is a phenomenon in which loose and saturated cohesionless soils are subject to a temporary, but essentially total loss of shear strength, because of 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 5% fines passing the No. 200 sieve. According to Special Publication 117 by the Division of Mines and Geology, 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 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, the soil type, depth to bedrock, in-situ moisture contents, the groundwater level, and the location of the site to the nearest active fault and the predicted ground surface acceleration. Bedrock was encountered 14 to 19 feet below the ground surface. The soils overlying the bedrock are predominately clayey in nature with a soft to very stiff consistency. Interbedded granular soil layers were encountered within the Baymud deposits in Boring 3. The granular layer was found to have approximately 44% passing the No. 200 sieve. Based on the data obtained and in view of the above noted criteria, it is our opinion that the potential for liquefaction related hazards at the site is low.

Settlement Considerations

As previously discussed, the site is underlain by fill and Bay Mud deposits which extend to 14 to 18 feet below the existing ground surface. The fill has been in place for more than 60 years based on the age of the house. The fill was found to generally consist of silty and sandy clay with bedrock fragments and has a stiff to very stiff consistency. The native Bay Mud deposits are considered to be potentially compressible, especially where new loads are applied due to buildings or fill. Based on the anticipated loading conditions from a one and/or two story house and the anticipated minor grading, we estimate that the long term total settlement will be on the order of 2 inches with up to 1 inch of differential settlement across the residence foundations. Should the scope of the project change from that known at the time of this report, we must be notified to further evaluate the settlement potential and provide mitigation recommendations as required.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

General

From a geotechnical point of view, the proposed residences and associated improvements are feasible for construction on the subject site 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. Any work related to the grading and/or foundation operations performed without the full knowledge and under the direct observation of the Soil Engineer will render the recommendations of this report invalid.

Geotechnical Considerations

The primary geotechnical concerns for the site are the near-surface, moderately expansive clays, the presence of old fill, potentially compressible Baymud deposits, and buried debris. The near surface soils are prone to heave and shrink movements with changes in moisture content and, consequently, must be carefully considered in the design of grading, foundations, drainage, and landscaping. The recommendations provided in the following sections will minimize the effects of expansive soil movement.

The subsurface fills and Baymud deposits are anticipated to undergo some consolidation due to the proposed loading conditions. We expect total settlements of up to 2 inches and differential settlements of up to 1 inch. Based on these anticipated settlements and the potential variability of the fill soils and debris content, it is our opinion that shallow spread footing foundations are not appropriate for structure support. Therefore, two options are recommended for the residences to be constructed on the site. The first is to support the residences on a pier and grade beam foundation system that extends into the underlying bedrock. Based on our field investigation, piers on the order of 25 feet deep could be expected. The second option is to support the residences on a structural mat-slab foundation that is designed for the above settlements. Regardless of the option chosen, it is recommended to rework the upper 4 feet of the site within the structural building areas to provide relatively uniform near surface conditions for slab and structure support.

At the time of this report, development plans were not available indicating the extent of the planned grading and the location of the residences. Once these plans are provided for our review, supplemental recommendations will be provided as necessary for the actual proposed conditions.

Demolition

Prior to any grading or construction on the site, demolition of the existing structures at the site should be completed. Demolition should include the complete removal of all surface and subsurface structures. Where any of the following are encountered: concrete, underground tanks, storm drain systems, foundations, asphalt, debris and trash, these should also be removed, with the exception of items specified by the owner for salvage. In addition, all underground structures must be located on the grading plans so that proper removal may be carried out. It is vital that ***KC ENGINEERING CO.***, intermittently observe the demolition operations and be notified in ample time to ensure that subsurface structures are not covered.

Excavations made by the removal of any structure should be left open by the demolition contractor for backfill in accordance with the requirements for engineered fill. The removal of any underground structures should be done under the observation of the Soil Engineer to assure adequacy of the removal and that subsoils are left in proper condition for placement of engineered fills. Any soil exposed by the demolition operations, which are deemed soft or unsuitable by the Soil Engineer, shall be excavated as uncompacted fill soil and be removed as required by the Soil Engineer during grading. The demolition operation should be approved by the Soil Engineer prior to commencing grading operations. Any resulting excavations should be properly backfilled with engineered fill under the observation of the Soil Engineer. Should the location of any localized excavation be found to underlie any structure, backfill should be compacted to a minimum relative compaction of 95% or the excavation widened to extend 5 feet beyond the footprint of the structure and backfilled to the specifications for engineered fill as recommended in the "grading" section herein.

Grading

Grading activities during the rainy season will be hampered by excessive moisture. Grading activities may be performed during the rainy season, however, achieving proper compaction may be difficult due to excessive moisture; and delays may occur. Grading performed during the dry months will minimize the occurrence of the above problems.

The surface of the site in areas to be graded should be stripped to remove all existing vegetation and/or other deleterious materials. It is estimated that stripping depths of 1 to 2 inches may be necessary, however, the actual depth of stripping will be determined in the field by the Soil Engineer. Any material that is deemed to be topsoil and requiring stripping may not be used as engineered fill but may be stockpiled and used later for landscaping purposes. Removal of trees and bushes should include the rootball and associated root systems. Depressions resulting from removal of trees and bushes should be cleaned of loose soils and roots, and properly backfilled in accordance with the recommendations of this report.

Where any loose or soft soils are encountered, they must be excavated to undisturbed native ground. 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 investigation, except any excessive organic contaminated materials, would be suitable for use as engineered fill when placed and compacted at the recommended moisture content.

Following site stripping of vegetation and the demolition operations, the upper 4 feet of the site should be reworked within the structural areas. A structural area is considered the footprint of the residence, driveways and surrounding flatwork plus 5 feet beyond laterally. The lower 1 foot of the structural fill may be processed in place. Therefore, the site should be excavated to a depth of 3 feet below existing grade. The bottom of the excavation should be scarified to a minimum depth of 12 inches and compacted to a minimum degree of relative compaction of 90% at a moisture content at least 3% above optimum as determined by ASTM D1557 Laboratory Test Procedure. If any additional loose or soft soils are encountered, they must be excavated to competent ground as observed and approved by the Soils Engineer. The upper 3 feet may then be replaced as engineered fill in lifts not exceeding 8 inches in loose thickness and compacted as noted above.

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.

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 in accordance with Laboratory Test Procedure ASTM D2922 and D3017, respectively. 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, has not been met, the particular layer, or portion thereof, shall be reworked until the compaction requirements have been met.

Surface Drainage

A very important factor affecting the performance of structures and pavement areas is the proper design, implementation, and maintenance of surface drainage. Ponded water will cause swelling and/or loss of soil strength and may also seep under structures or pavement areas. 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. The site drainage should be designed by the Project Civil Engineer. To minimize the potential for the above problems, 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 or seeping beneath the structures and/or pavement areas. Where concrete is not provided adjacent to the structure, the finished grades should be compacted and sloped at a minimum 3% gradient away from the exterior foundation and be directed to yard swales or area drains that discharge to the street or other approved drainage facility. All hardscapes must also slope away from the structures.

b) Enclosed or trapped planter areas adjacent to the structure foundation 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 foundation and into area inlets. Drainage area inlets should be piped to a suitable discharge facility.

c) The construction of continuous roof gutters is recommended. The downspouts should be connected to a closed pipe system to carry storm water away from the structures and graded areas. In doing this, the possibility of soil saturation adjacent to the foundation and engineered fills is reduced. Downspout water may be allowed to discharge directly onto hardscape surfaces provided positive drainage is maintained.

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 structure.

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.

Foundations

Based on the results of the field and laboratory testing program, the site's near surface foundation soils are considered to be moderately expansive. In addition, the site is underlain by potentially compressible fill and Bay Mud deposits. Therefore, the proposed residential structures should be supported on a post-tensioned slab-on-grade or a friction pier and grade beam foundation system. Recommendations for both foundation types are presented below.

Post-Tensioned Slabs-On-Grade

The post-tensioned slab should be a minimum 10 inches in thickness and designed using the following criteria which is based on the Post-Tensioning Institute's *Design and Construction of Post-Tensioned Slabs-On-Ground*. The design should be performed by the expansion potential and the compressible soils method with the final design based on the controlling method.

Edge Moisture Variation Distance:		
e_m (Edge Lift)	=	4 feet
e_m (Center Lift)	=	4 feet
Differential Movement:		
y_m (Edge Lift)	=	1.0 inches
y_m (Center Lift)	=	1.5 inches
Differential Settlement:	=	1.0 inch (compressible soils)

The following recommendations should be incorporated into the design and construction for the above structural mat foundation systems:

a) An allowable bearing capacity of 1,000 p.s.f. should be utilized and may be increased by one-third to resist short-term wind and seismic loading.

b) To resist lateral loading, a coefficient of friction of 0.25 may be used.

c) All areas to receive slabs should be wetted to seal any dessication cracks prior to placing the underslab components. This work should be performed under the observation of the Soil Engineer and approved prior to concrete placement.

d) The reinforcement and/or cables shall be placed in the center of the slab unless otherwise designated by the Structural Engineer.

e) A heavy duty vapor retarder (10-mil minimum) membrane, that meets ASTM E1745, should be placed between the moisture conditioned subgrade and the slab to minimize moisture condensation under the floor covering and upward vapor transmission. It is recommended that the vapor retarder be adequately lapped and taped in accordance with ASTM E1643.

f) The slabs should be thickened a minimum of 12 inches wide at the edges to extend below pad grade at least 2 inches to create frictional resistance for lateral loading. If it is desired to construct the foundation at pad grade, instead of trenching 2 inches at the perimeter, the slab may be constructed as follows; slab over vapor retarder over 2 inches of a granular base material over the moisture conditioned pad. The base material as recommended in ACI 302.1R should be an unwashed size No. 10 material per ASTM D 448. This material should meet a gradation of 100% passing 3/8", 85 to 100% passing No. 4, 10 to 30% passing No. 100, and 0 to 5% passing No. 200.

g) Garage slabs and front porch slabs should be designed as part of the mat foundation system as recommended above.

h) The foundation plans, specifications, and calculations should be provided to us for review prior to construction to ensure conformance with the above recommendations.

The following items of consideration are presented with respect to the above recommendations and with respect to placement of moisture sensitive floor coverings on concrete.

a) Placement of concrete directly on a vapor retarder can result in the delay of the initial set of concrete. The concrete contractor should be notified to allow for proper finishing and curing of the concrete.

b) Water vapor migrating to the surface of the concrete can adversely affect floor covering adhesives. Provisions should be provided in the concrete mix design to minimize moisture emissions. This could include the selection of a water-cement ratio which inhibits water permeation (0.50 max) or the addition of suitable admixtures to limit water transmission.

Pier and Grade Beam:

The drilled piers should have a minimum diameter of 18 inches and extend a minimum depth of 6 feet into the underlying bedrock. It is noted that the final pier depth should be determined in the field by the Soil Engineer based on actual field conditions. Therefore, the Soil Engineer must be present during the foundation excavation operations.

The piers should be designed on the basis of skin friction acting between the soil and that portion of the pier that extends below a depth of 10 feet below finished grade. For the soil/bedrock at the site, an allowable skin friction value of 500 p.s.f. can be used for combined dead and live loads.

This value can be increased by one-third for total loads which include wind or seismic forces. Reinforced concrete grade beams should be used to support the perimeter walls and, if desired, certain bearing walls of the building structures. Reinforcing steel should be provided as necessary for structural support and continuity of pier and grade beam. Piers supporting grade beams should be reinforced with a minimum of four No. 4 reinforcing bars for the full depth of the piers and interior girder piers should be reinforced with a minimum of two No. 4 bars. The grade beams should be reinforced with a minimum of four No. 4 bars, two located near the top and two near the bottom of the grade beams. Spacing of the piers should be determined, as required, by the load distribution but minimum spacing should not be less than three pier diameters, center to center. It is noted that the above recommendations are minimums only. The actual design of the foundation must be performed by a qualified Structural Engineer in accordance with current standard of practice and for the anticipated loading conditions.

To resist lateral loads, the passive resistance of the soil can be used. The soil passive pressures can be assumed to act against the lateral projected area of the pier described by the vertical dimension of twice the pier diameter. It is recommended that a passive pressure equivalent of that of a fluid weighing 300 p.c.f. be used below the upper 5 feet.

Even though the piers will be designed to develop their capacity through friction, their bottoms should be cleaned and/or tamped prior to placing reinforcing steel and pouring concrete. Also, it is important that care be exercised to ensure that any concrete spills during the concrete pour must be removed, and no "mushrooming" effects are allowed to remain around the top of the pier or bottom of the grade beam.

It is noted that the piers will extend below sea level. The pier excavations are expected to remain open for the short duration required to place the reinforcing steel and concrete. However, it is noted that some minor sloughing may occur, especially where debris is encountered. Where significant debris is encountered, casing may be required. It is suggested that the piers drilled on a given day be poured on the same day to minimize caving. The concrete will need to be placed by the tremmie method in order to displace the water and any mud. The foundation contractor must be made aware of these conditions.

Non-Structural Slab-on-Grade Construction

Garage slabs and exterior concrete flatwork, including driveways, patios and walkways, placed on the expansive soils may experience some cracking due to moisture variations within the underlying soils and the resulting shrink/swell phenomenon. Interior slabs on grade are not recommended in living areas where a pier and grade beam foundation is used. To reduce the potential cracking of the slabs-on-grade, the following recommendations are made:

a. All areas to receive slabs should be thoroughly wetted to seal any dessication cracks prior to placing of concrete. This work should be done under the observation of the Soil Engineer.

b. Four inches of angular gravel or clean crushed rock material should be placed between the finished subgrade and slabs, excluding a 6 inch width at slab edges to serve as a capillary break between the subsoil and the slab. The concrete slabs should be thickened at the edges and rest on grade.

c. All slabs should be a minimum of 5 inches thick and reinforced with a minimum of No. 4 bars at 16 inches on center each way. The reinforcement shall be placed in the center of the slab unless otherwise designated by the design engineer.

d. Also, exterior slabs should be provided with tool joints or crack control strips to control expansion and contraction of the concrete. The joints should extend along the middle of the slab in both directions.

General Construction Requirements

Utility trenches extending underneath all traffic areas must be backfilled with native or approved import material and compacted to relative compaction of 90% to within 6 inches of the subgrade. The upper 8 inches should be compacted to 92% 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 Vallejo, 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.

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

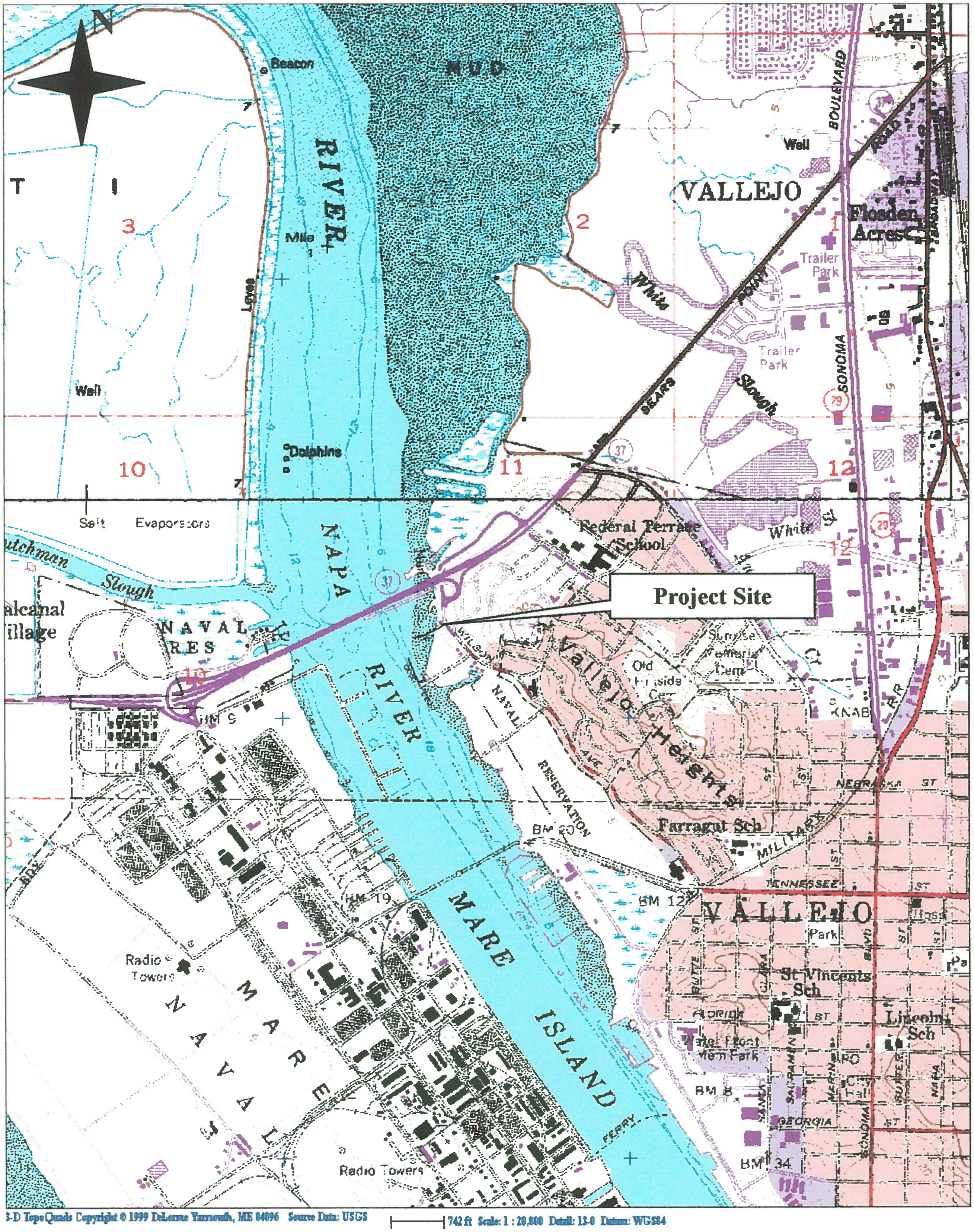
<ins>Vicinity Map</ins>

<ins>Site Plan</ins>

<ins>Log of Test Boring</ins>

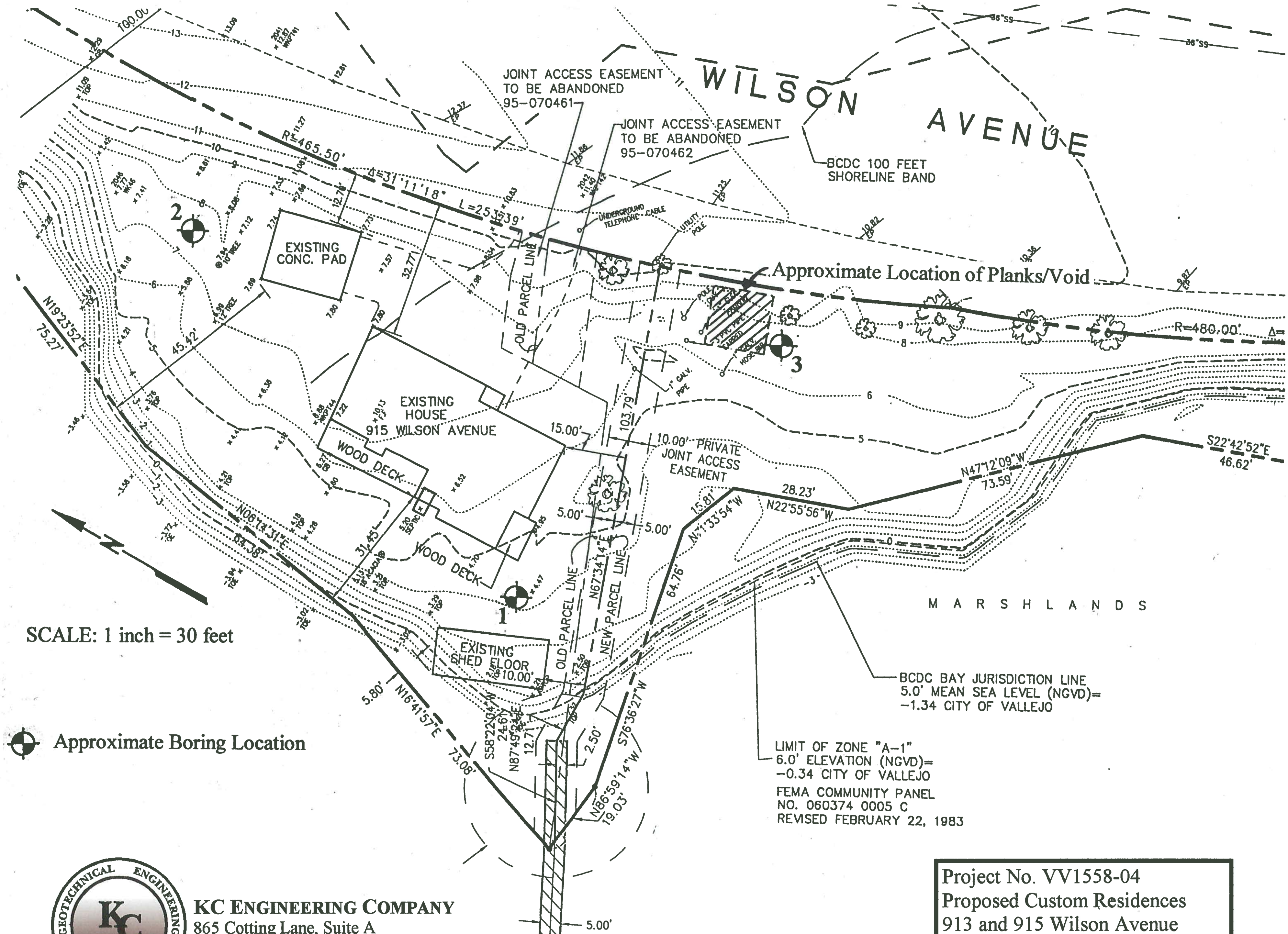
<ins>Boring Log Legend</ins>

<ins>Laboratory Test Results</ins>



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

Project No. VV1558-04
Proposed Custom Residences
913 and 915 Wilson Avenue
Vallejo, California
FIGURE NO. 1 - VICINITY MAP



SCALE: 1 inch = 30 feet

Approximate Boring Location

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

Project No. VV1558-04
Proposed Custom Residences
913 and 915 Wilson Avenue
Vallejo, California
FIGURE NO. 2 – SITE PLAN

LOG OF TEST BORING
BORING NO.: 1

PROJECT: Proposed Custom Residences
CLIENT: H&B Developers
LOCATION: 913 and 915 Wilson Ave, Vallejo
DRILLER: Ram Geotechnical Drilling, Inc.
DRILL RIG: Mobile B24
DEPTH TO WATER: INITIAL ▽ : 7.5 ft.

PROJECT NO.: VV1558-04
DATE: 18 August 2004
ELEVATION: 4.4 ft
LOGGED BY: JSP
BORING DIAMETER: 4 inch
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				Olive Brown silty CLAY with sandstone fragments; dry, stiff to very stiff (FILL)	CL				
	1-1					15	-	-	LL=33% PI=17
5	1-2					22	112.4	13.3	UCC=8124 psf
				Grey and Red mottled silty CLAY with mudstone and sandstone fragments; damp, very stiff, grades to firm w/ depth (FILL) ▽	CL				
10	1-3					8	107.1	20.7	UCC=787 psf
	SPT 1-4			Dark Grey/Black CLAY; wet, soft (BAYMUD-Native)	CH	4	-	-	
15				Mottled Red and Grey silty CLAY with caliche pockets; wet, very stiff (Completely weathered mudstone)	CL				
	1-5					26	103.8	25.9	
				Red and Grey MUDSTONE with interbedded Sandstone; moderately weathered, weak, closely fractured					
20	1-6					62	-	-	
25									

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

LOG OF TEST BORING
BORING NO.: 1

PROJECT: Proposed Custom Residences
CLIENT: H&B Developers
LOCATION: 913 and 915 Wilson Ave, Vallejo
DRILLER: Ram Geotechnical Drilling, Inc.
DRILL RIG: Mobile B24
DEPTH TO WATER: INITIAL ▽ : 7.5 ft.

PROJECT NO.: VV1558-04
DATE: 18 August 2004
ELEVATION: 4.4 ft
LOGGED BY: JSP
BORING DIAMETER: 4 inch
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)
30	1-7					100+	-	-	
				Boring terminated at 31 feet					
35									
40									
45									
50									

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

KC ENGINEERING CO.

Figure 3

LOG OF TEST BORING
BORING NO.: 2

PROJECT: Proposed Custom Residences
CLIENT: H&B Developers
LOCATION: 913 and 915 Wilson Ave, Vallejo
DRILLER: Ram Geotechnical Drilling, Inc.
DRILL RIG: Mobile B24
DEPTH TO WATER: INITIAL ▽ : 11 ft.

PROJECT NO.: VV1558-04
DATE: 18 August 2004
ELEVATION: 7.5 ft
LOGGED BY: JSP
BORING DIAMETER: 4 inch
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 with mudstone and sandstone fragments; dry, stiff (FILL)	CL				
	2-1			Red brick debris		12	-	-	
5				----concrete, possibly a slab?					
10	2-2			Dark Grey CLAY; wet, firm (BAYMUD-Native) ▽	CH	6	-	-	
15	2-3			Brown and Grey MUDSTONE with interbedded sandstone; intensely fractured, weak, highly weathered		100+	-	-	
20	2-4					100+	-	-	
				Boring terminated at 21 feet					
25									

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

KC ENGINEERING CO.

Figure 4

LOG OF TEST BORING
BORING NO.: 3

PROJECT: Proposed Custom Residences
CLIENT: H&B Developers
LOCATION: 913 and 915 Wilson Ave, Vallejo
DRILLER: Ram Geotechnical Drilling, Inc.
DRILL RIG: Mobile B24
DEPTH TO WATER: INITIAL ▽ : 10.5 ft.

PROJECT NO.: VV1558-04
DATE: 18 August 2004
ELEVATION: 7 ft
LOGGED BY: JSP
BORING DIAMETER: 4 inch
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				Olive Brown sandy CLAY with sandstone fragments; dry, loose to stiff (FILL)	CL				
5	3-1					18	113.1	6.0	LL=38% PI=20
				----Grades without bedrock fragments, very stiff					
	3-2					21	112.6	12.1	
10				Dark Grey CLAY with occasional thin sand lenses; wet, stiff (BAYMUD-Native) ▽	CH				
15	3-3					9	103.6	23.7	%Gravel=7.0 %Sand=49.4 %<200=43.6
20				Red Brown MUDSTONE with interbedded sandstone; weak, intensely fractured, moderately weathered					
	3-4					100+	-	-	
25				Boring terminated at 24 feet					

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

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than No. 200 Sieve	GRAVELS 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
		Gravel with fines (>12% fines)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures (PI<4 & below "A" line)
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures (PI>7 & above "A" line)
	SANDS More than half of 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
		Sands with fines (>12% fines)	SM	Silty sands, poorly graded sand-silt mixtures (PI<5 & below "A" line)
			SC	Clayey sands, poorly graded sand-clay mixtures (PI>7 & below "A" line)
FINE GRAINED SOILS More than half of material is smaller than No. 200 Sieve	SILTS AND CLAYS Liquid Limit is less than 50%		ML	Inorganic silts and very fine sands, silty or clayey fine sands, clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and clays of low plasticity
	SILTS AND CLAYS Liquid Limit is more than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of medium to high plasticity
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

GEOTECHNICAL ENGINEERING CONSULTANTS KC

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

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
Φ=Friction Angle
C=Cohesion
UCC=Unconfined Compression
R value=Resistance Value
Consol=Consolidation Test

SOIL GRAIN SIZE

U.S. STANDARD SIEVE OPENINGS

CLAY	SILT	SAND			GRAVEL		COBBLES	BOULDERS
		FINE	MEDIUM	COARSE	FINE	COARSE		
Sieve:		#200	#40	#10	#4	¾"	3"	12"
mm:	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 fractrured	5/8 to 1
Crushed	<5/8

MTI
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

CLIENT: H & B Developers
c/o Robert A. Karn Associates
707 Beck Avenue
Fairfield, CA 94533

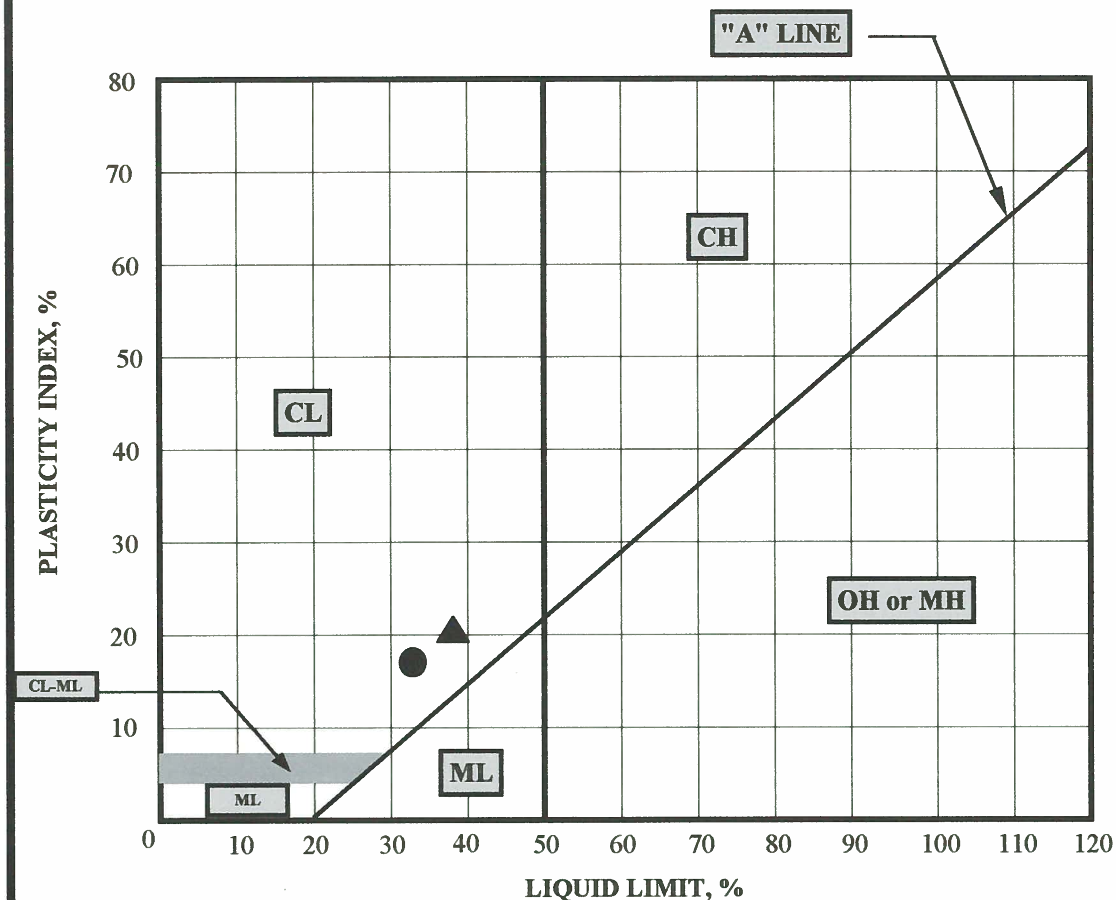
CLIENT NO: VV1558-001
REPORT NO: 0300-002
DATE: 09/02/04

SUBJECT: 913/915 Wilson Avenue
Vallejo, California

SUBMITTED BY: KC Engineering

DENSITY OF IN PLACE SOIL BY THE DRIVE TUBE METHOD (ASTM D2937)
LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX OF SOILS (ASTM D4318)
DATA SHEET

Sample #	Description	Dry Density pcf.	Moisture Content %	Liquid Limit %	Plastic Limit %	Plastic Index %
1-1 @ 3.0’	Brown Sandy Clay (Visual)	---	---	33	16	17
1-2 @ 6.0’	Olive Brown Sandy Clay (Visual)	112.4	13.3	---	---	---
1-3 @ 11.0’	Olive Brown Gravelly Clay (Visual)	107.1	20.7	---	---	---
1-5 @ 16.0’	Olive Brown Clay (Visual)	103.8	25.9	---	---	---
3-1 @ 4.0’	Light Brown Gravelly Clay (Visual)	113.1	6.0	38	18	20
3-2 @ 9.0’	Brown Clay (Visual)	112.6	12.1	---	---	---
3-3 @ 14.0’	Olive Brown Clayey Sand with Gravel (Visual)	103.6	23.7	---	---	---



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	PLASTIC LIMIT, PL, %	LIQUID LIMIT, LL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	1-1	3 feet	–	16	33	17	N/A	CL
▲	3-1	4 feet	6.0	18	38	20	-0.60	CL

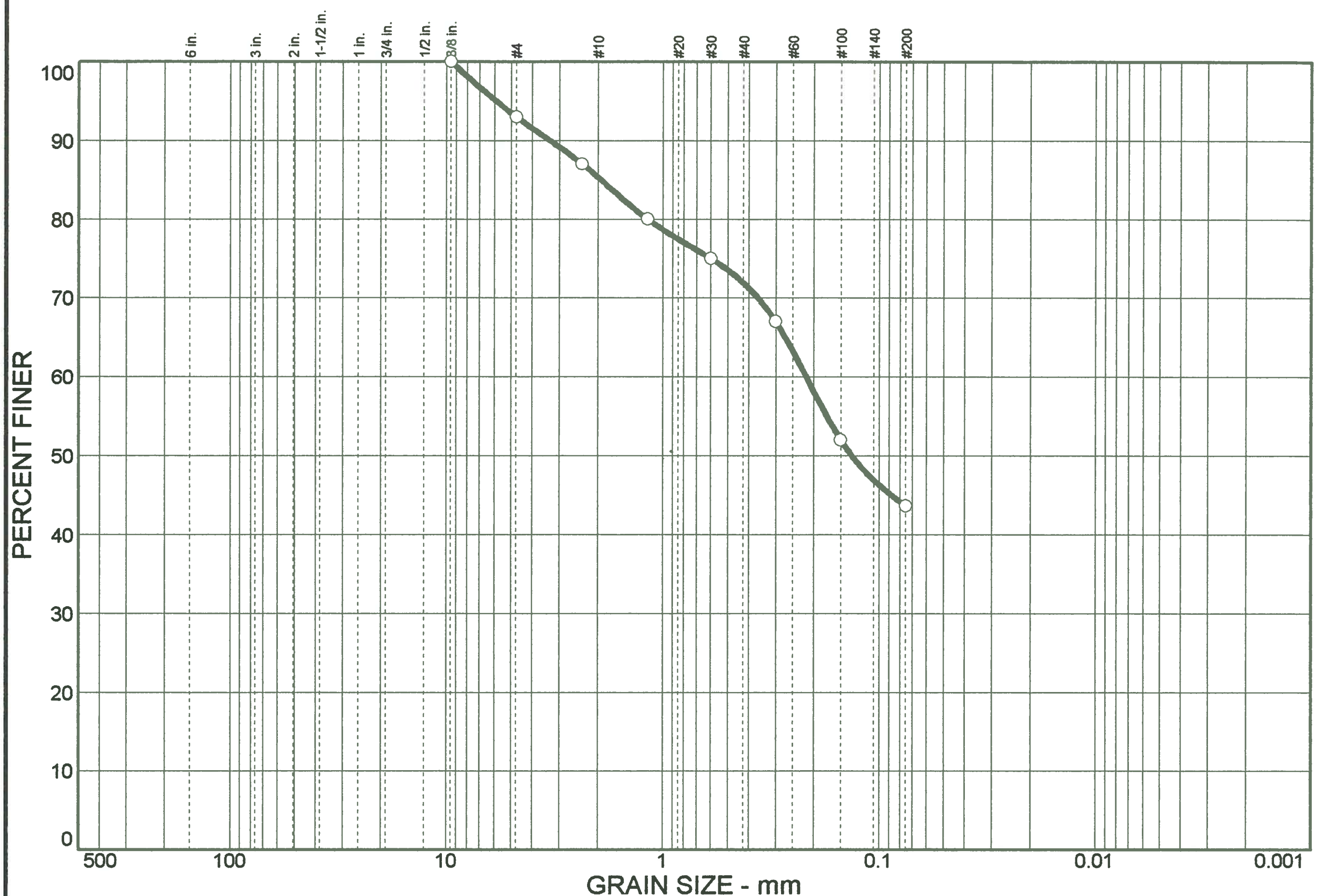
KC ENGINEERING CO.

PLASTICITY CHART AND DATA

Proposed Custom Residences
913 and 915 Wilson Avenue, Vallejo

PROJECT	DATE	FIGURE
VV1558-04	9/2/2004	

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	7.0	49.4	43.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	93.0		
#8	87.0		
#16	80.0		
#30	75.0		
#50	67.0		
#100	52.0		
#200	43.6		

* (no specification provided)

Soil Description

Olive Brown Clayey Sand with Gravel (Visual)

Atterberg Limits

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

Coefficients

D_{85}= 1.94 D_{60}= 0.217 D_{50}= 0.133
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

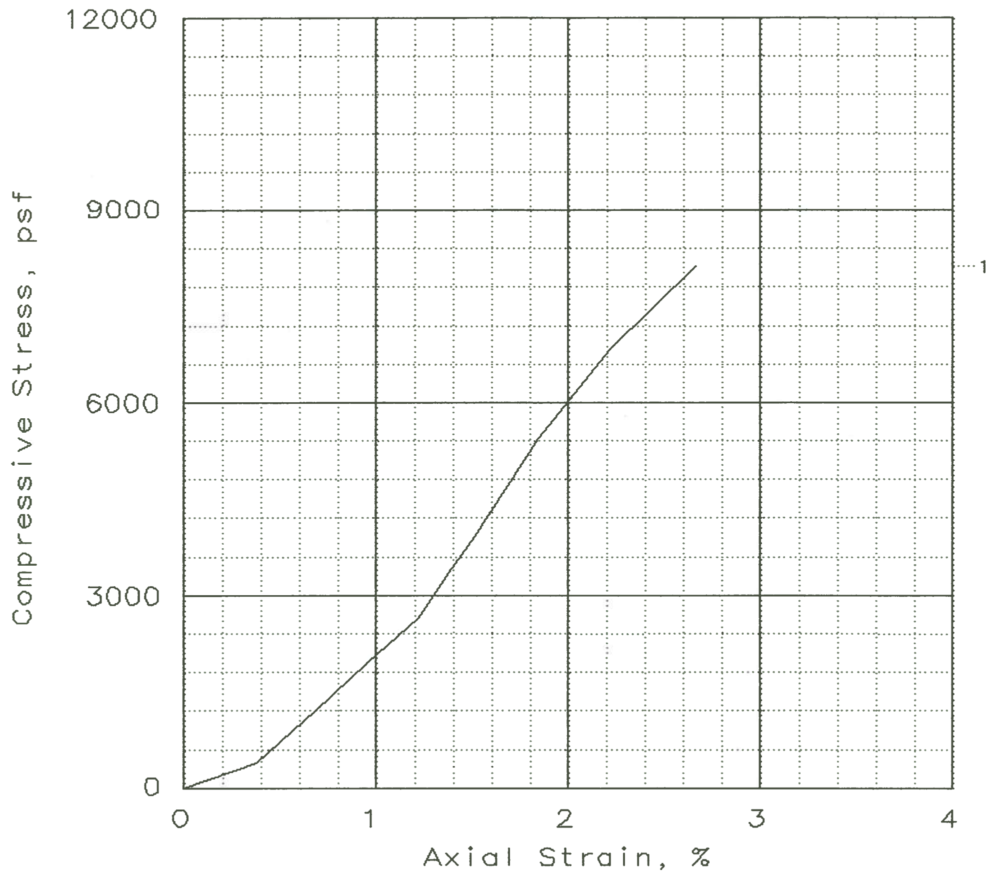
Remarks

Sample No.: 3-3 **Source of Sample:** 913/915 Wilson Avenue **Date:** 09/02/04
Location: **Elev./Depth:** 14'

Materials Testing, Inc.

Client: H & B Developers
Project: 913/915 Wilson Avenue - Vallejo
Project No: VV1558-001 **Plate** 0400-001

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	8124			
Undrained shear strength, psf	4062			
Failure strain, %	2.7			
Strain rate, %/min				
Water content, % (cuttings after test)	13.3			
Wet density, pcf	127.4			
Dry density, pcf	112.4			
Saturation, %	85.6			
Void ratio	0.3882			
Specimen diameter, in	2.410			
Specimen height, in	4.500			
Height/diameter ratio	1.87			

Description: Olive Brown Sandy Clay

			GS= 2.5	Type: Tube

Project No.: VV1558
Date: 9-2-04
Remarks:
Type of Failure

Exceeded Load Ring Capacity

Report No.: ______

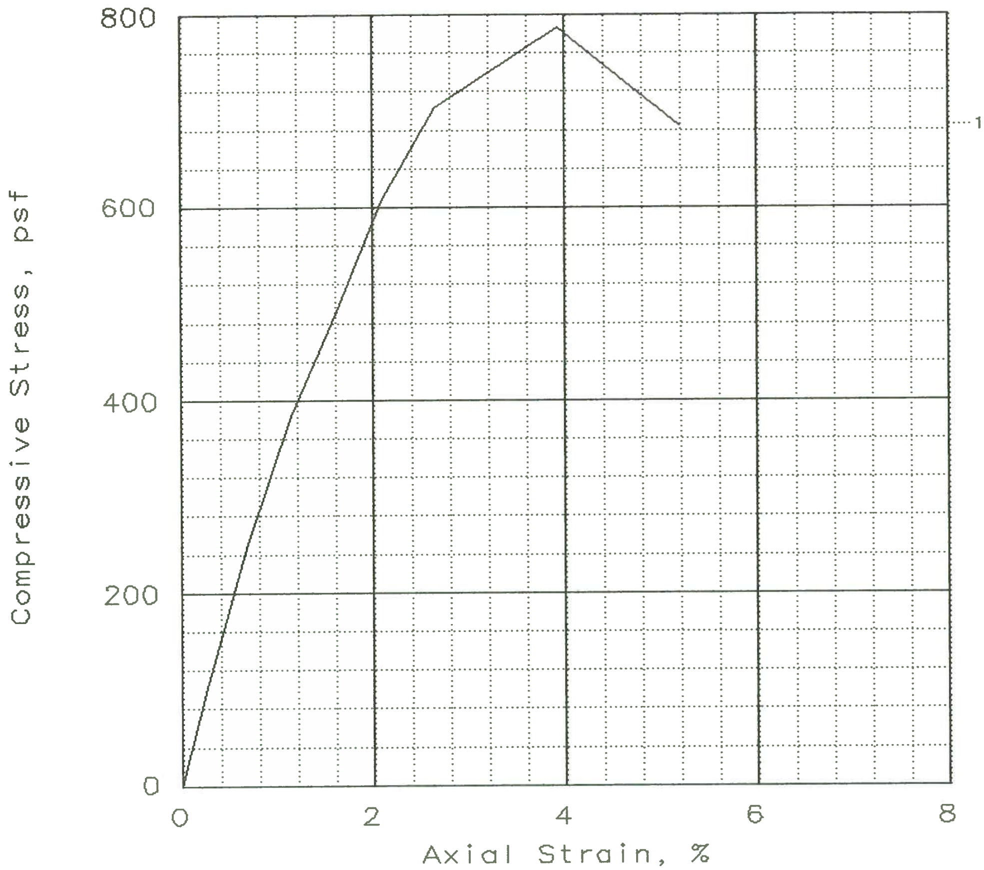
Client: H & B Developers

Project: 913/915 Wilson Avenue
Vallejo, Ca.
Location: 1-2@6'

UNCONFINED COMPRESSION TEST

MATERIALS TESTING, INC.

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	787			
Undrained shear strength, psf	394			
Failure strain, %	3.9			
Strain rate, %/min				
Water content, % (cuttings after test)	20.7			
Wet density, pcf	129.2			
Dry density, pcf	107.1			
Saturation, %	111.9			
Void ratio	0.4632			
Specimen diameter, in	2.410			
Specimen height, in	3.920			
Height/diameter ratio	1.63			

Description: Olive Brown Gravelly Clay

GS= 2.51 Type: Tube

Project No.: VV1558

Date: 9-2-04

Remarks:

Type of Failure

Cone & Split

Report No.: ____

Client: H & B Developers

Project: 913/915 Wilson Avenue
Vallejo, Ca.

Location: 1-3@11'

UNCONFINED COMPRESSION TEST

MATERIALS TESTING, INC.