BUCKLEY ENGINEERING ASSOCIATES
Geotechnical Engineering and Geology

1770 Golden Hills Drive
Milpitas, CA 95035
Phone: 408/942-6952
Fax : 408/942-6952

September 19, 2005
Job #02505.1

RECEIVED
SEP 27 2005
COUNTY OF ALAMEDA
BUILDING INSPECTION
DIVISION

Mr. Wayne Ting
42329 Osgood Road, Suite A
Fremont, CA 94539-5061

RE: GEOLOGIC REPORT UPDATE
Proposed Residential Subdivision
2492 D Street
Hayward, California

Dear Mr. Ting:

As requested, we are providing this report update for the referenced project. Previously, we have prepared a geologic investigation report report, dated 8-21-02.

We observed the site conditions on September 16, 2005. Except for lush vegetation growing along the westward linear trending drainage, the conditions were essentially unchanged from the time of the original investigation. Therefore, the conclusions of that report remain valid for the present project.

We understand that the surficial fill and debris on the southern portion of the site will be addressed during grading. The project geotechnical engineer should review the plans and observe the geotechnical aspects of construction.

If you have any questions concerning this letter, please call our office.

Very truly yours,

BUCKLEY ENGINEERING ASSOCIATES

D. W. Buckley

David W. Buckley
Certified Engineering Geologist 1110

DAVID W. BUCKLEY
No. EG 1110
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

Distribution: 2 to Mr. Wayne Ting

Addendum Attachment E-5/p.1

BUCKLEY ENGINEERING ASSOCIATES
Geotechnical Engineering and Geology

3452 Lisbon Drive
San Jose, CA 95132
Phone: 408/942-6952
Fax : 408/942-6952

August 21, 2002
Job #02505.1

Mr. Wayne Ting
Wayne Ting & Associates
44360 South Grimmer Blvd.
Fremont, CA 94538

RE: GEOLOGIC INVESTIGATION
Proposed Subdivision
2492 D Street
Hayward, California

Dear Mr. Ting:

INTRODUCTION

As requested, we are providing this geologic investigation letter report for the referenced project. This investigation was conducted to evaluate the potential risks associated with geologic hazards in this area of Alameda County in order to aid in the planning and design of the proposed residential project. Cleary Consultants provided a geotechnical investigation for the site in 1989.

The site is located on the north side of D Street about 300 feet west of Stratton Court as shown on Plate 1, Vicinity Map. An existing unimproved narrow road extending about 600 feet northward off of D Street (Site Plan, Plate 3) accesses the site.

The scope of services undertaken to arrive at the findings and conclusions in this report included:

- Review of geologic maps and reports covering the site area.
- Geologic interpretation of stereo aerial photography.
- Site reconnaissance of the site and surrounding area.
- Geologic logging of 7 test pits excavated by backhoe.
- Analysis and preparation of this report.

FINDINGS

Moderate to steep, smoothly rounded slopes underlain at shallow depths by competent bedrock characterize the site. Intervening areas are underlain by thick colluvial-filled, ephemeral drainages. The site is covered with eucalyptus trees, grasses and brush.

The site is approximately 300 feet above sea level in the Castro Valley-Hayward foothills, about 1.4 miles northeast of the active Hayward Fault (Geologic Map, Plate 2). The site is far enough away not to be located in the Special Studies Zone associated with the Hayward Fault (CDMG, 1982). Parallel traces of the East and West Chabot Faults lie concealed under alluvium about ½ and 1-mile southwest of the site, respectively.

According to Dibblee (1980) the site is underlain by the Panoche Formation consisting of siltstone and sandstone with occasional shale and claystone interbeds. Northeast of the Chabot Faults, Dibblee shows that beds strike northwesterly and dip steeply to the northeast. We did not observe any rock outcrops on the site. However, bedrock was exposed at the bottom portions of all of the test pits.

The site lies in a seismically active region dominated by faults of the San Andreas Fault System. The trace of the active San Andreas Fault is located about 20 miles northeast of the site. The active Calaveras Fault lies about 7 miles northeast of the site. Major historic earthquakes produced by the San Andreas Fault System have produced strong to violent ground shaking at the site. The most recent of the strong earthquakes on the northern segment of the nearby Hayward Fault is thought to have occurred in 1868. This earthquake ruptured the ground surface along the main trace of the Hayward Fault southwest of the site.

In the aerial photographs, we did not observe any evidence of landsliding on the slopes at the site. In the 1939 photos the slopes appeared smooth, like they do today. The only difference in 1939 was that the site was covered with orchards.

In the test pits we encountered a variable thickness (from 1 to 7 feet) of generally low plasticity, silty clay underlain by sandstone bedrock (See the logs contained in Plates 4 - 6). Plate 4 also contains general descriptions of the materials encountered. The sandstone was fine-grained, massive, blocky and fractured. In Test Pit TP-2 we measured bedding trending North 40 degrees west and dipping almost vertically. Joints or perhaps bedding trending North 20 degrees west and dipping vertically were measured in Test Pit TP-7. Localized ground water seepage occurred at a depth of about 6 feet in Test Pit TP-1.

CONCLUSIONS

On the basis of our study, we conclude that there are no geologic hazards that would prohibit the proposed residential development. No faults have been mapped through the building sites, and no evidence of faulting through the building areas was found during this evaluation. Consequently, the risk of fault rupture affecting the project is low.

Geologists agree that the seismic shaking hazard is high in many areas in California, especially within about 30 miles of the San Andreas Fault System, which includes the San Andreas, Hayward and Calaveras Faults (State of California, 1996). Consequently, on the basis of the historic record, it is reasonable to assume that the site will be subject to violent ground shaking within the lifetime of the proposed project. Building damage due to shaking can be reduced provided the project is designed according to the seismic provisions of the 1997 Uniform Building Code and lessons learned from recent large earthquakes.

Because of the shallow depth to bedrock, the risk of liquefaction is very low. The use of engineered fills and retaining walls can mitigate possible seismic lateral spreading on the steeper slopes to be developed.

On the basis of our site reconnaissance and the materials encountered in the test pits, we believe that the site is underlain by relatively stable bedrock. In our opinion, provided drainage and ground water seepage is controlled, either static or earthquake-induced landsliding at the site is of low probability. This hazard can be further mitigated through the use of grading, engineered retaining walls and prudent foundation design.

Adverse bedding (bedding parallel to slopes) was not encountered during our investigation. Therefore, we do not expect adverse bedding conditions to be a factor during grading for this project.

Although backhoe refusal was encountered in a few of the test pits on the northern part of the site, we expect that the specified cuts can be achieved by heavy conventional excavating equipment.

REFERENCES CITED

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United States Geologic Survey, Menlo Park, Aerial Photography: BUT-282-31, 32; 1:20,000; 7-26-39.

LIMITATIONS

This letter report has been prepared in accordance with generally accepted engineering geologic principles and practices and is in accordance with the standards of practice set by the geologic consultants in the area. This acknowledgment is in lieu of all warranties, either expressed or implied.

We trust that this report provides the necessary information. If you have any questions, please call.

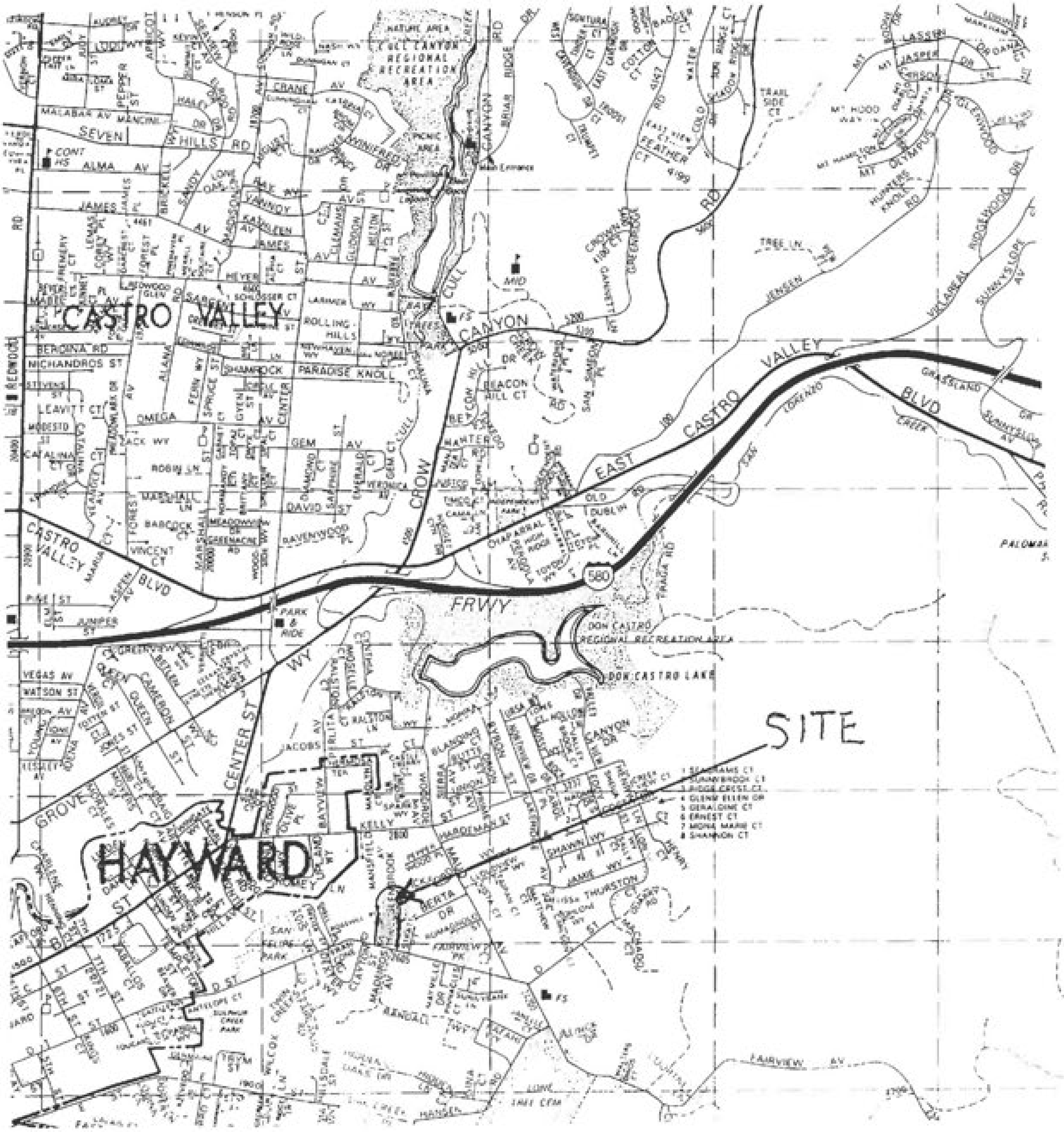
Very truly yours,

BUCKLEY ENGINEERING ASSOCIATES

David W. Buckley
Certified Engineering Geologist 1110

REGISTERED GEOLOGIST
DAVID W. BUCKLEY
No. EG 1110
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Distribution: 3 to Wayne Ting & Associates



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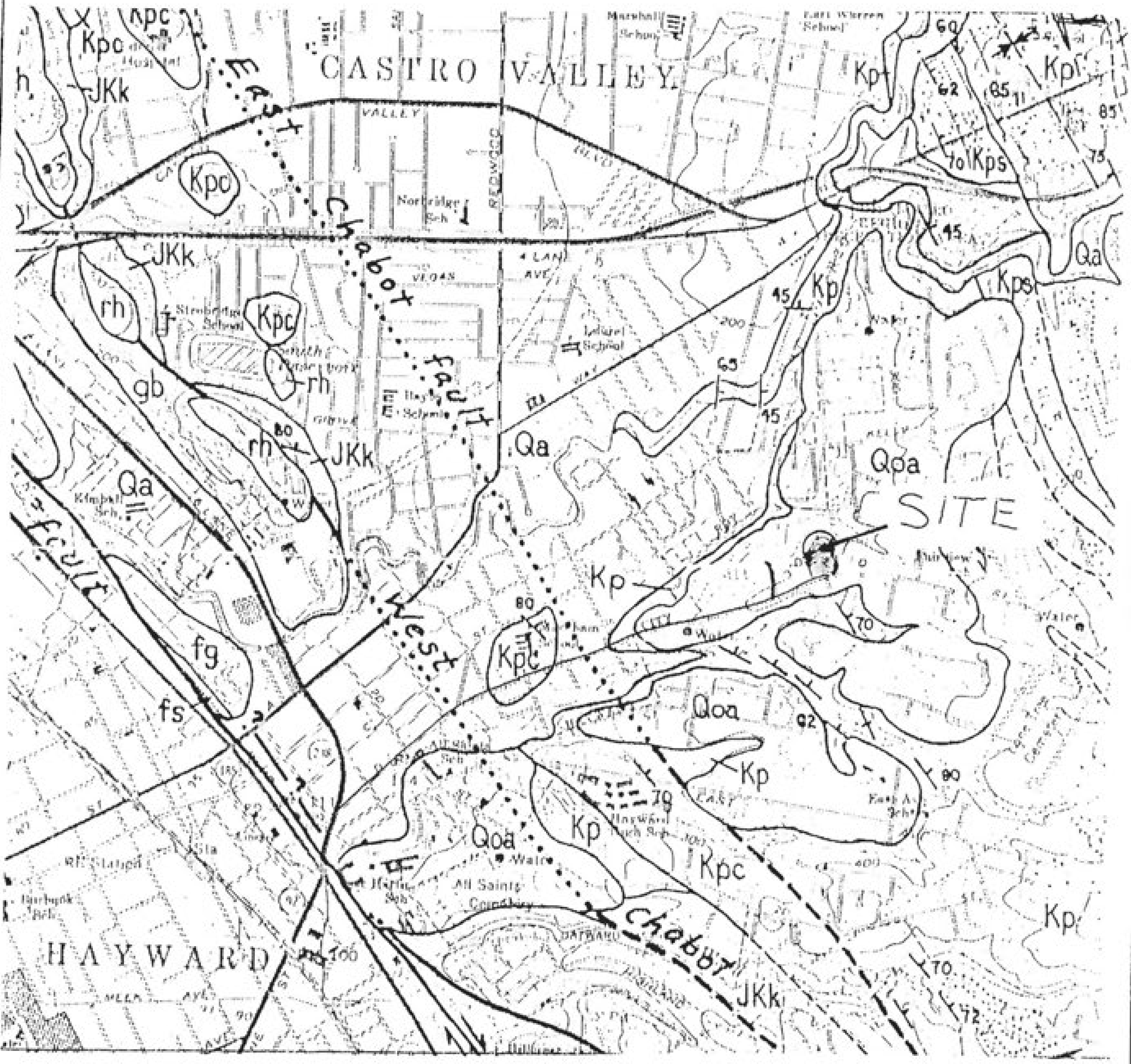
From Thomas Bros. Maps, 1993 ed.

Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	VICINITY MAP D Street Hayward, CA	Plate 1



Qoa- Older Alluvium
Kp- Panoche Formation

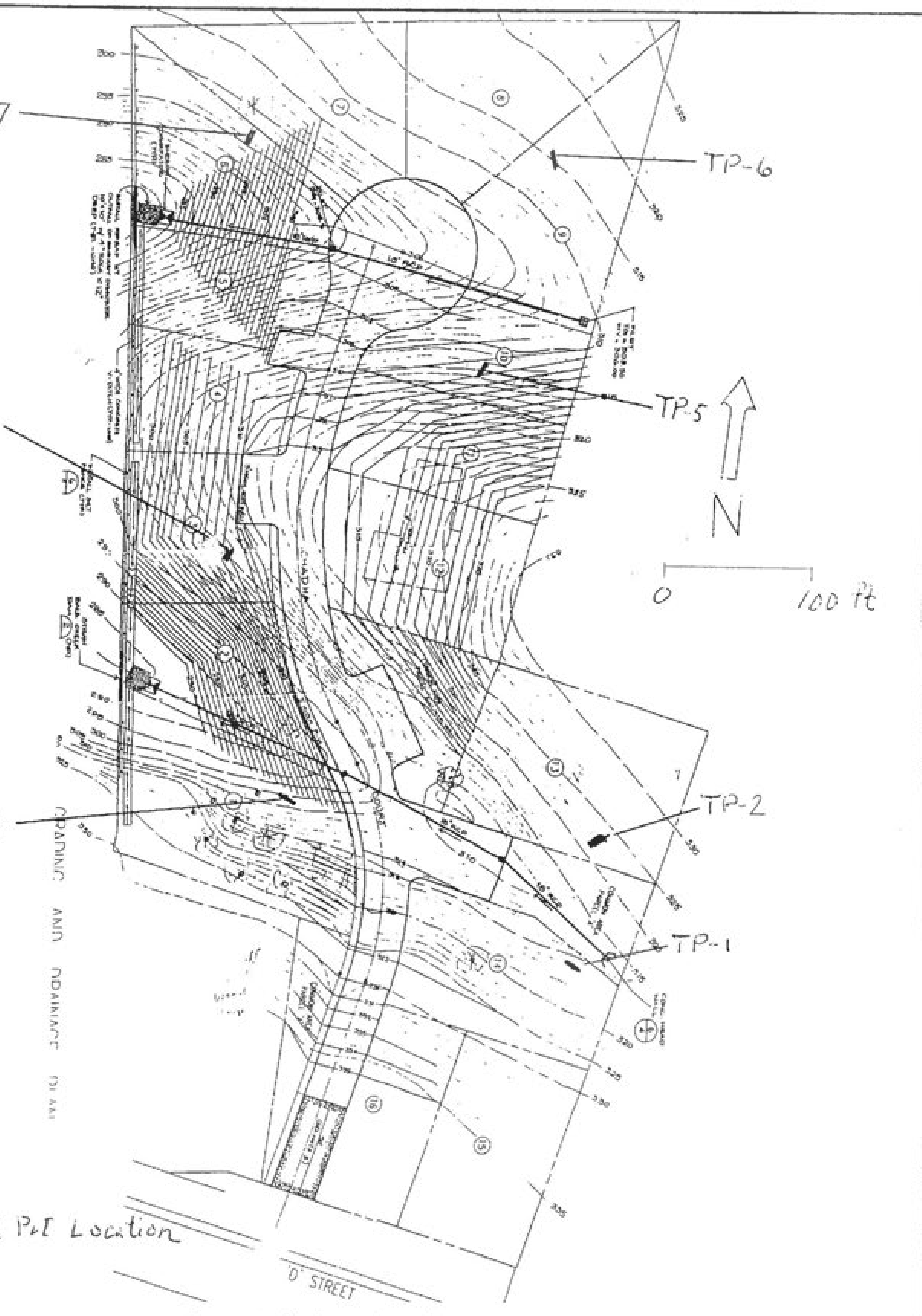
0 2000 ft.

N

From Dibblee, 1980; Hayward Quadrangle.

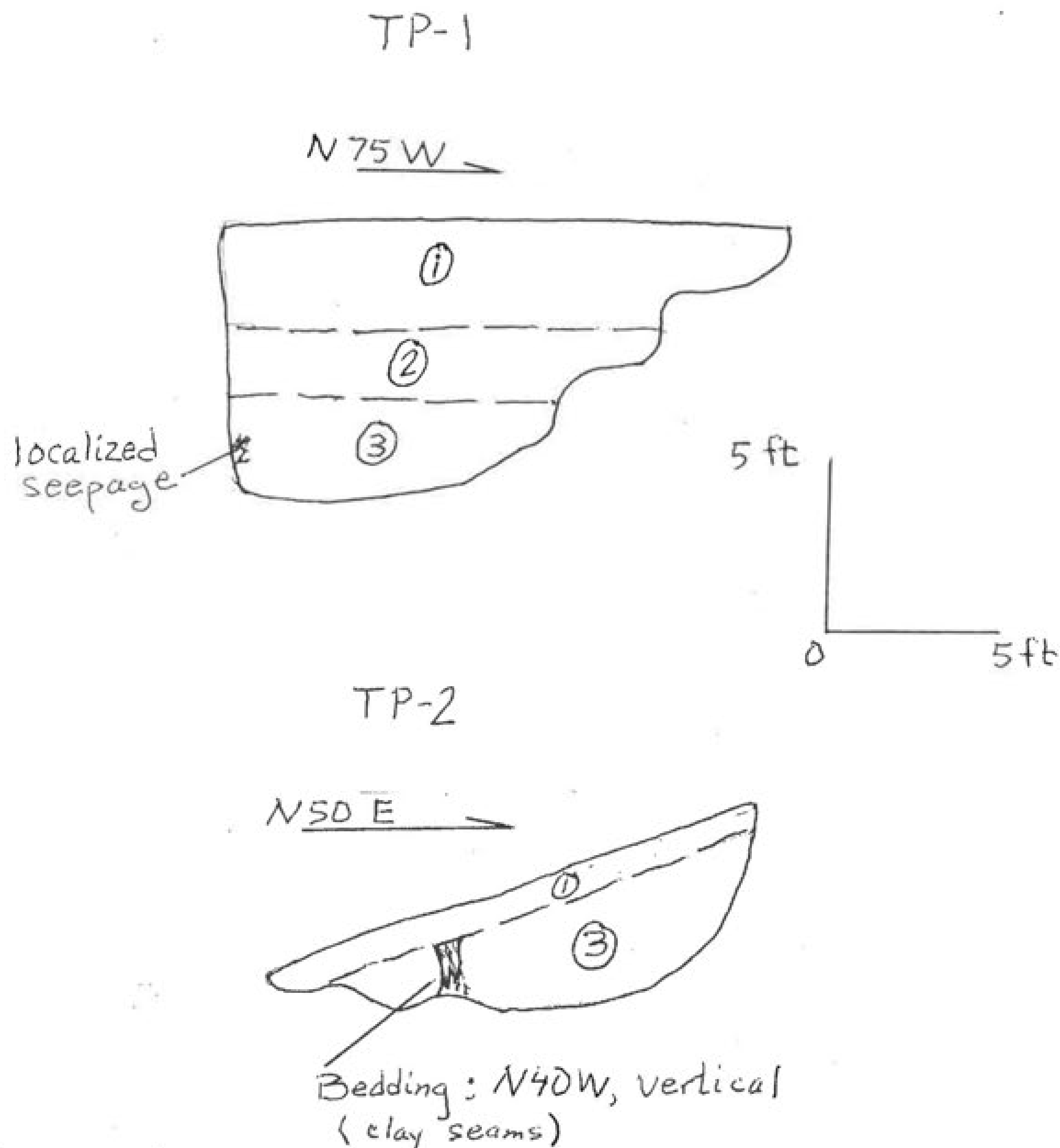
Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	GEOLOGIC MAP D Street Hayward, CA	Plate 2

Addendum Attachment E-5/p.7



From Grading Plan Provided by Client

Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	SITE PLAN D Street Hayward CA	Plate 3



① Gray-brown silty Clay

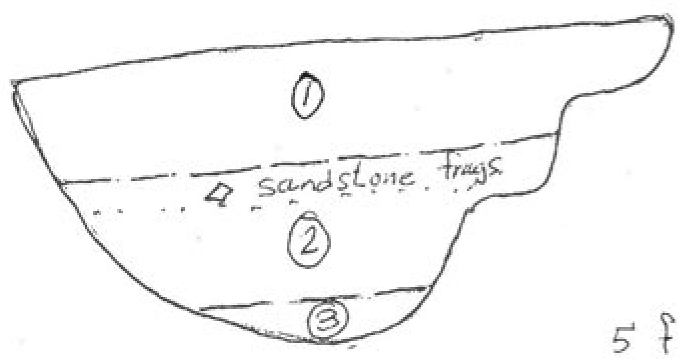
② Brown silty clay, more clayey with rock fragments

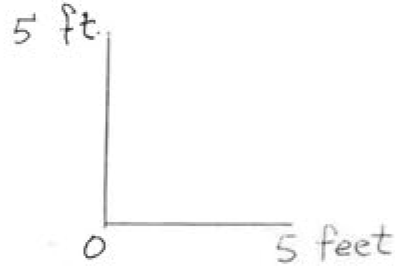
③ Brown fractured, massive, soft to hard, fine-grained weathered sandstone

Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 4

TP-3

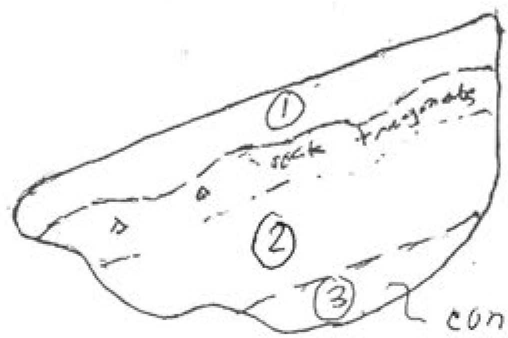
N60W





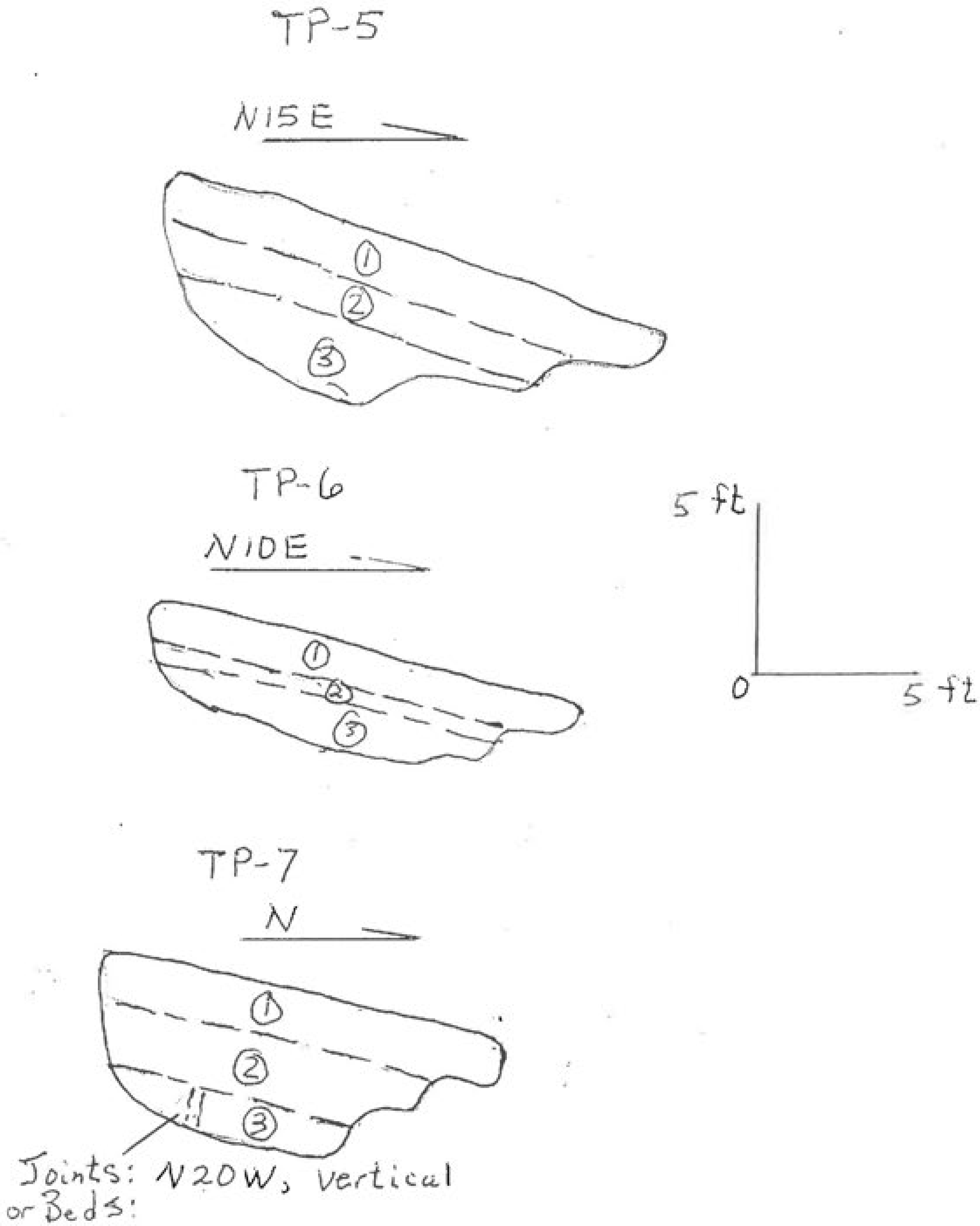
TP-4

N20E



contains siltstone and shale, but massive, no bedding observed

Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	**Plate** 5



Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 6

BUCKLEY ENGINEERING ASSOCIATES
Geotechnical Engineering and Geology
3452 Lisbon Drive
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Phone: 408/942-6952
Fax : 408/942-6952

August 21, 2002
Job #02505.1

Mr. Wayne Ting
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44360 South Grimmer Blvd.
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RE: GEOLOGIC INVESTIGATION
Proposed Subdivision
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We trust that this report provides the necessary information. If you have any questions, please call.

Very truly yours,

BUCKLEY ENGINEERING ASSOCIATES

David W. Buckley

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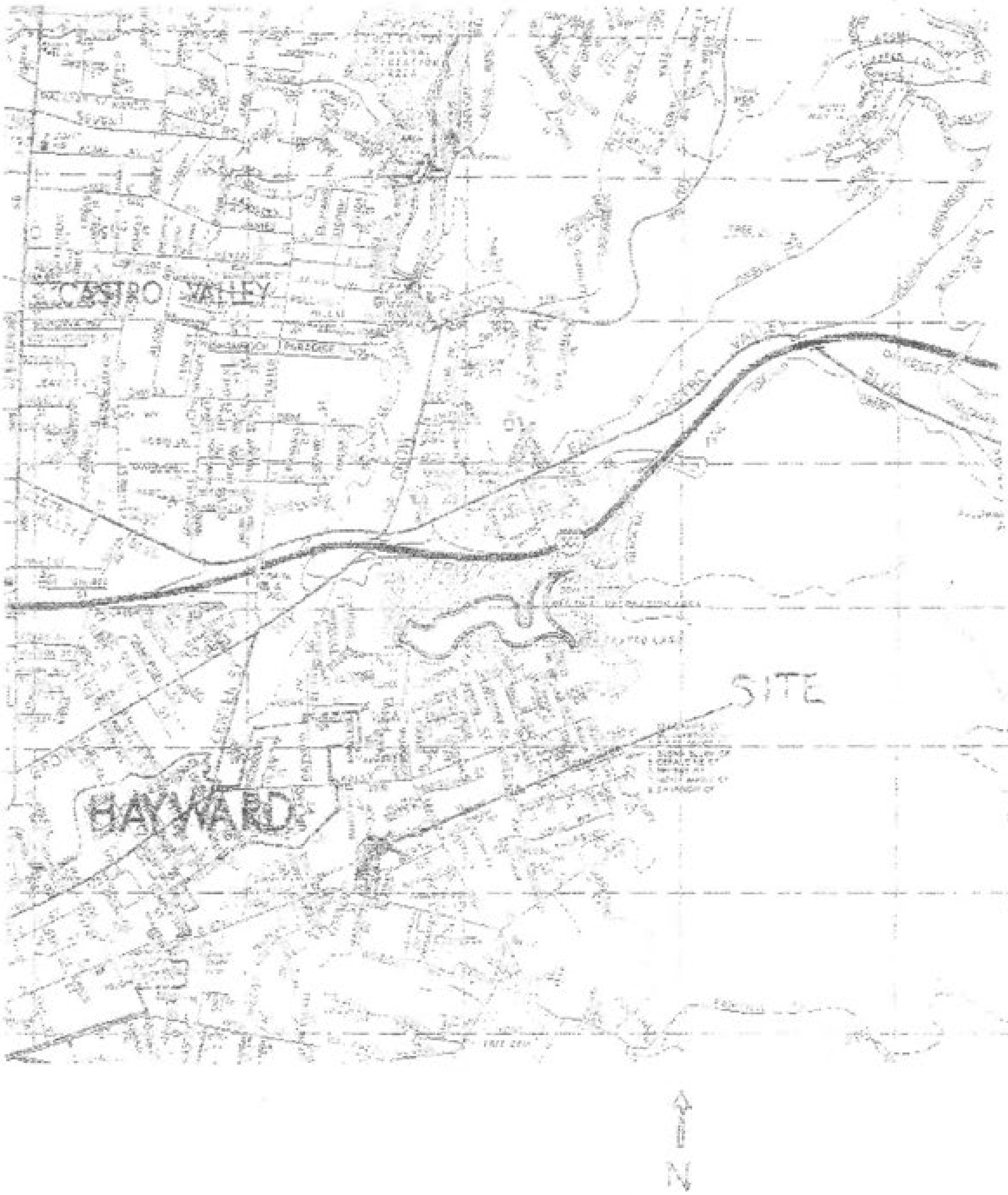
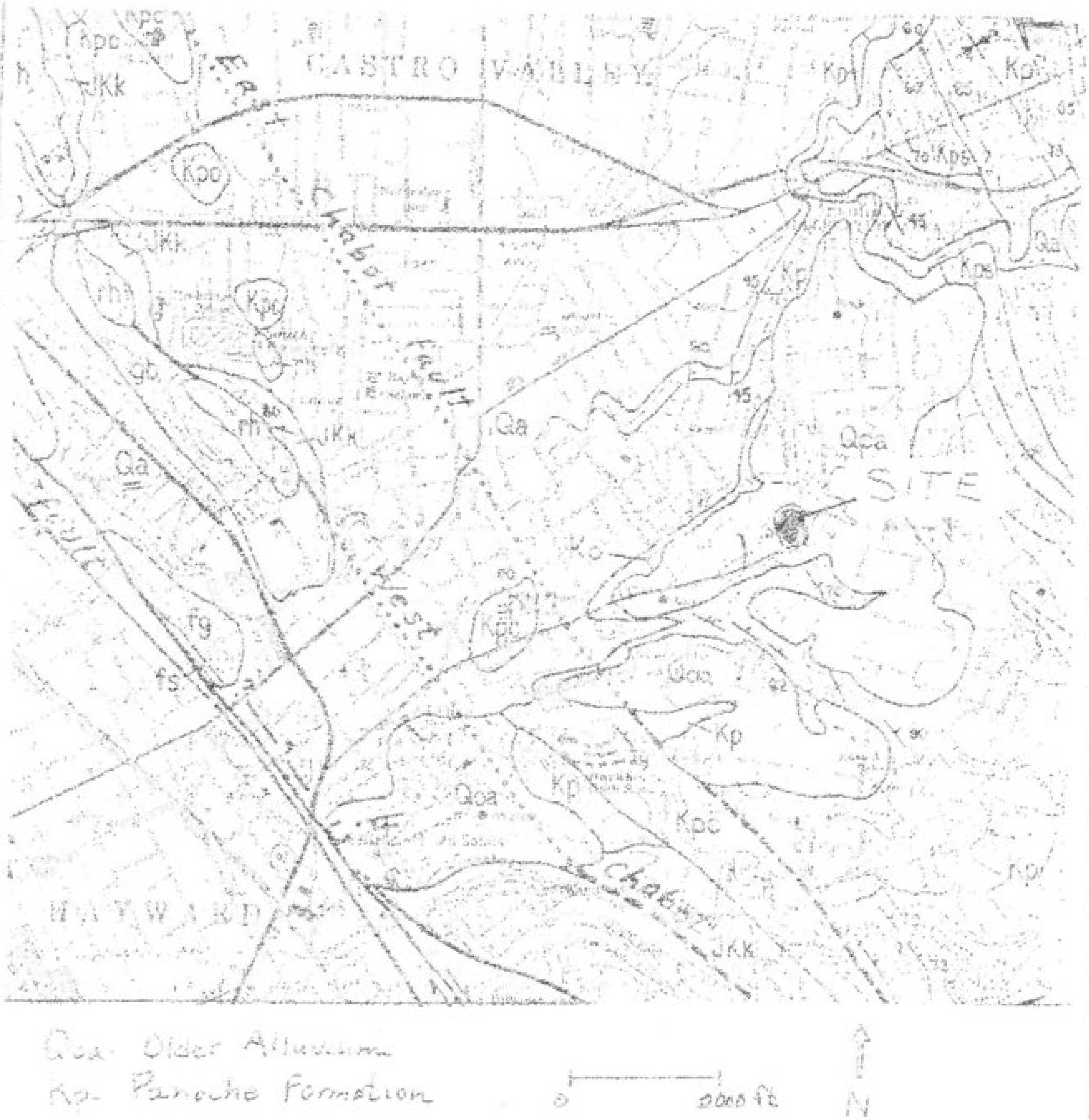


From Thomas Bros. Maps, 1993 ed.

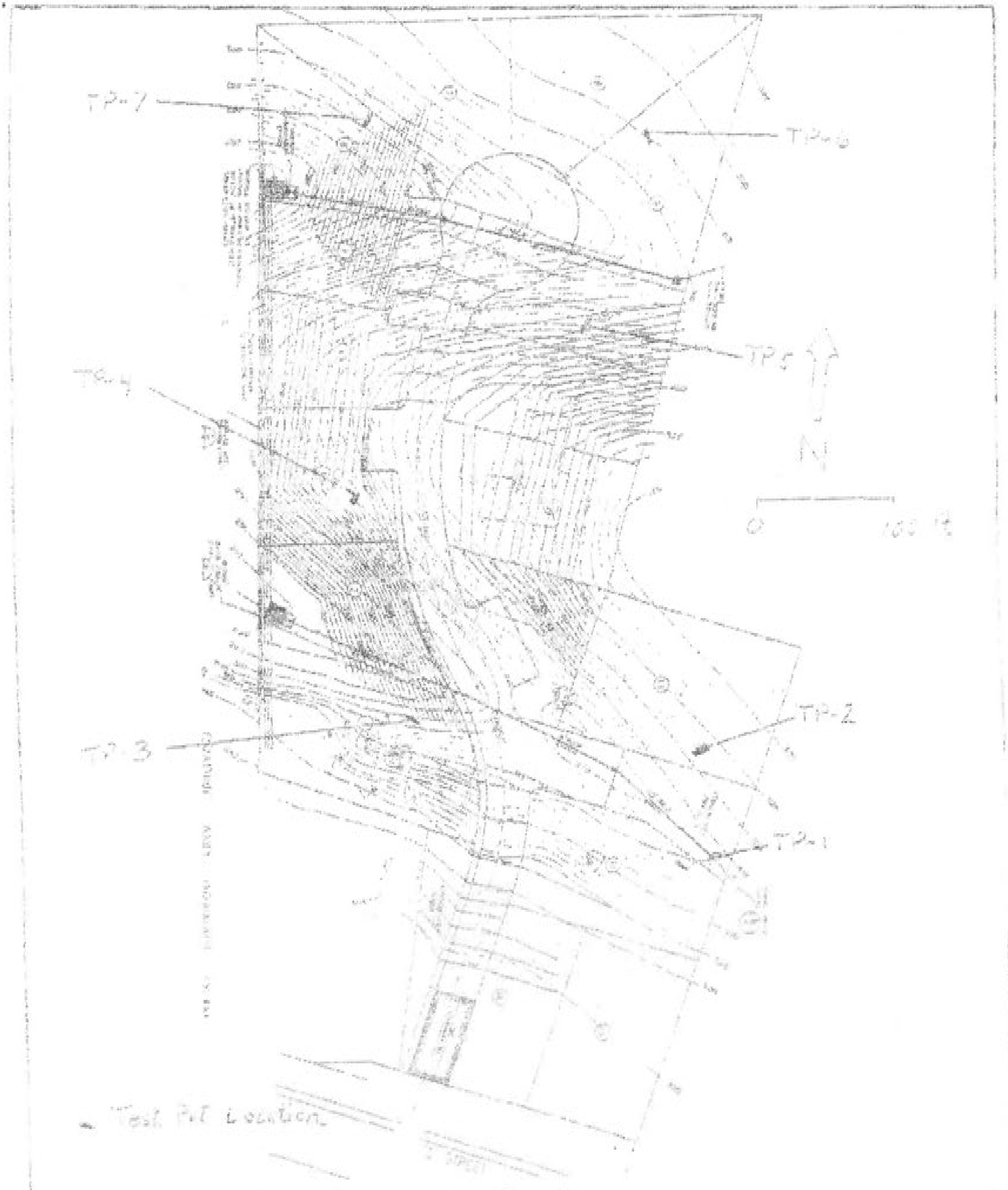
Buckley Engineering Associates	Job No. 02505.1 Approved [illegible] Date 8-21-02	VICINITY MAP D Street Hayward, CA	Plate 1



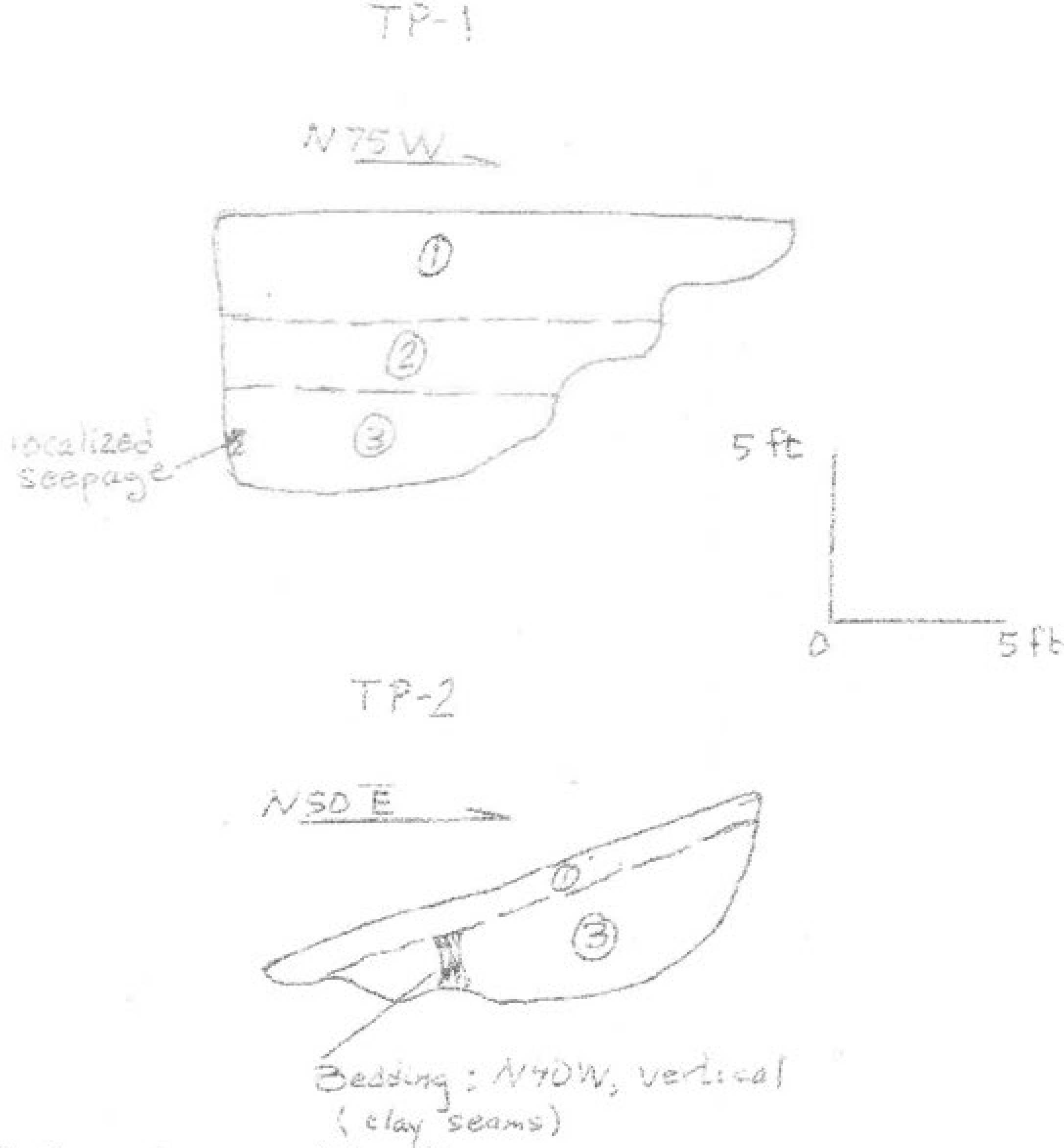
Qoa- Older Alluvium
Kp- Panoche Formation

From Dibblee, 1980, Hayward Quadrangle

Buckley Engineering Associates	Job No. 025051 Approval DWB Date 8-21-02	GEOLOGIC MAP D Street Hayward, CA	Plate 2



Buckley Engineering Associates	Job No. 025551 Approved Date 8-21-02	Site Plan D Street Hayward, CA	Plate 3

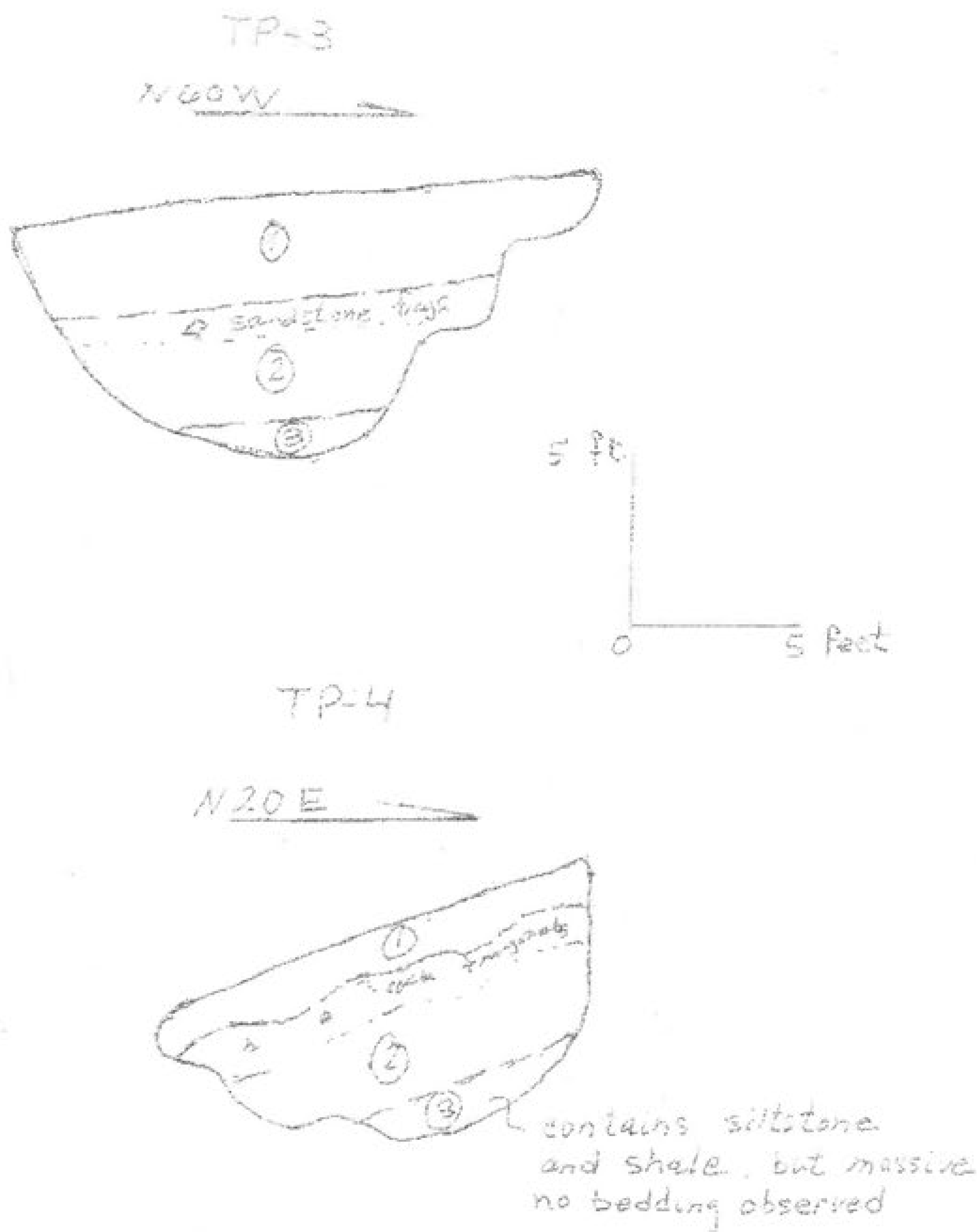


① Gray-brown silty Clay

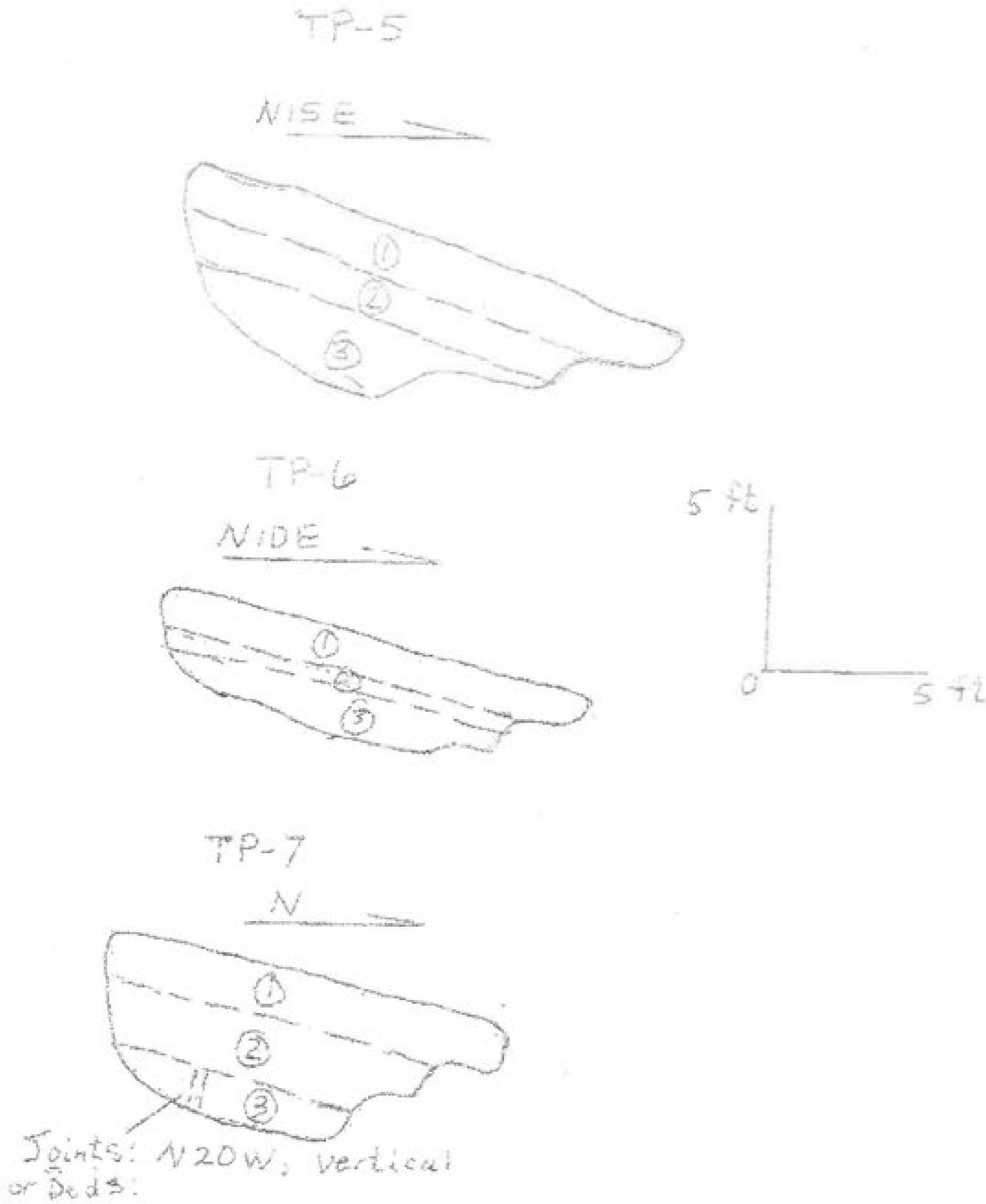
② Brown silty clay, more clayey with rock fragments

③ Brown, fractured, massive, soft to hard, fine-grained weathered sandstone

Buckley Engineering Associates	Job No. 02505.1 Approved DNB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 4



Buckley Engineering Associates	Job No. 02505 / Approved DWB / Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 5



Buckley Engineering Associates	Job No. 02505.1 Approved JMB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 6

GEOTECHNICAL INVESTIGATION

TRACT NO. 5965

2492 D STREET

ALAMEDA COUNTY, CALIFORNIA

CLEARY CONSULTANTS, INC.

Geotechnical Engineers and Geologists

Addendum Attachment E-7/p.1

CLEARY CONSULTANTS, INC.
Geotechnical Engineers and Geologists

July 7, 1989
Project No. 219.1A
Ser. 3930

Mr. Lubomir Peichev
106 West 43rd Avenue
San Mateo, California 94403

**RE: GEOTECHNICAL INVESTIGATION
TRACT NO. 5965
2492 D STREET
ALAMEDA COUNTY, CALIFORNIA**

Dear Mr. Peichev:

In accordance with your request, we have performed a geotechnical investigation for the proposed Tract 5965 at 2492 D Street in Alameda County, California. The accompanying report presents the results of our field investigation, laboratory testing, and engineering analyses. The site and subsurface conditions are discussed and recommendations for the soil and foundation engineering aspects of the project are presented. This report is contingent upon our review of the grading and foundation plans for the project and observation/testing of the earthwork and foundation installation phases of the project.

We refer you to the text of the report for detailed findings and recommendations. If you have any questions concerning our findings, please call.

Yours very truly,

CLEARY CONSULTANTS, INC.

Rick Swanson

Rick Swanson
Civil Engineer 38821

J. Michael Cleary

J. Michael Cleary
Engineering Geologist 352
Geotechnical Engineer 222

RS/JMC:ms
Copies: Addressee (2)
Marvin E. Smitherman, Jr., Consulting Engineer (2)
Arkady Faktorovich (1)
Gene St. Onge (1)

Addendum Attachment E-7/p.3

TABLE OF CONTENTS

DRAWINGS

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Tract 5965 at 2492 D Street in Alameda County, California. The property is located on the north side of D Street about 300 feet west of Stratton Court as shown on Drawing 1 - Site Vicinity Map. The purpose of this investigation was to determine the prevailing soil and bedrock conditions within the areas to be developed and provide soil and foundation engineering recommendations for the project design.

As shown on the Preliminary Grading and Tentative Map by Marvin Smitherman, the project will consist of twelve new single family lots at the 2.8 acre parcel. The project will also include construction of a new cul-de-sac street as shown on Drawing 2 - Site Plan.

Construction will consist of single family homes built close to existing grades. The homes will be one and two story, split level structures. It is anticipated that the homes will have raised wood floors in living areas and concrete slab-on-grade garage floors. Cuts and fills up to eight feet may be required for the street. Grading details for the building pads are not available at this time. Trench excavations 10 to 12 feet deep may be required for the planned gravity sewer.

We previously performed a geotechnical investigation of the site to provide soil and foundation engineering recommendations for a condominium project that was not built; the results of this study were presented in our report dated October 31, 1979. In addition to this report, we prepared a November 23, 1988, geotechnical feasibility update letter which concluded the presently proposed tract development is feasible from a geotechnical standpoint.

SCOPE

As presented in our proposal dated June 2, 1989, the scope of services for this investigation included:

1. A site reconnaissance and review of available geologic information for this area.

2. Subsurface exploration consisting of six borings drilled under the guidance of our engineering geologist.

3. Laboratory testing of samples obtained from the borings.

4. Soil and foundation engineering analyses using the field and laboratory data and preparation of a geotechnical investigation report. The report was to present findings and recommendations for:

   a) Suitability of the proposed building sites from a geotechnical standpoint.

   b) Site preparation and grading.

   c) Building foundation type, minimum depth, and allowable skin friction values.

   d) Treatment of expansive soils.

   e) Surface and subsurface drainage.

   f) Earth pressure criteria for retaining wall design.

   g) Excavation conditions and utility trench backfilling.

h) Flexible pavement sections for roadways and driveways.

i) Any other unusual design or construction conditions encountered during this study.

This report has been prepared for the specific use of Mr. Lubomir Peichev and his consultants in accordance with generally accepted soil and foundation engineering principles and practices. No other warranty, either expressed or implied, is made. In the event that any substantial changes in the nature, design, or location of the improvements are planned, the conclusions and recommendations of this report shall not be considered valid unless such changes are reviewed and the conclusions of this report modified or verified in writing.

METHOD OF INVESTIGATION

A site reconnaissance was performed by our engineering geologist on June 20, 1989. The subsurface exploration was also performed on June 20, 1989, using track-mounted, continuous flight auger drilling equipment. A total of six borings were drilled to a maximum depth of 16.5 feet at the locations shown on Drawing 2. A key describing the soil classification system and soil consistency terms used in this report is presented on Drawing 4 and the soil sampling procedures are described in Drawing 5. Logs of the borings are presented on Drawings 13 through 18. (Logs of the previous borings drilled for our 1979 study are included in this report as Drawings 7 through 12).

The borings were located in the field by pacing and interpolation of the features shown on the drawings provided us. These locations should be considered accurate only to the degree implied by the method used.

Samples of the soil materials from the borings were returned to our laboratory for classification and testing. The results of moisture content, dry density, percent finer than No. 200 sieve, unconfined compression, free swell, R-Value and plasticity index tests are shown on the boring logs. Drawing 19 presents additional information on the plasticity tests and Drawing 20 presents the results of an R-Value test. The laboratory test procedures followed during this investigation are summarized on Drawing 6.

A bibliography of references consulted during this investigation is included at the end of the text.

GEOLOGY AND SEISMICITY

The site is underlain by highly weathered siltstone and sandstone of the Panoche Formation (Dibblee, 1980) with occasional shale and claystone interbeds. Bedrock is exposed in cuts for the existing dwelling and access road at the site.

No active or inactive faults are known to pass through the site. The property, however, is located approximately 1 1/2 miles northeast of the Hayward fault, 20 miles northeast of the San Andreas fault and 7 miles southwest of the Calaveras fault, all of which are historically active. Therefore, as with the rest of the San Francisco Bay Area, the property is in a region of high seismic activity.

Although research on earthquake prediction has greatly increased in recent years, seismologists have not yet reached the point where they can accurately predict when and where an earthquake will occur. Nevertheless, on the basis of current technology, it is reasonable to assume that the proposed residences will be subjected to at least one moderate to severe earthquake during their design life. During such an earthquake, the danger from fault offset through the site is remote, but strong shaking is likely to occur.

SITE CONDITIONS

A. Surface

The property consists of a broad central ridge flanked by a sharply incised winter drainage to the south and a shallow minor swale to the north. The central ridge slopes westward at 7 to 14 percent and has 20 to 30 percent sideslopes on the north and 24 to 50 percent sideslopes on the south. (The slopes steepen near the bottom of the creek). Elevations vary from about 329 feet at the east central boundary to 282 feet in the northwest corner in the swale.

At the time of our investigation, there was a home near the center of the parcel and several small sheds in the north swale. Access to the property was provided by an asphalt paved driveway which is underlain by fill where it crosses the southern drainage. Vegetation consisted of a few trees in the northern swale and several large trees, shrubs, brushy debris, and weeds are in the southern portion of the parcel.

B. Subsurface

The borings encountered 0.5 to 4.5 feet of natural soil overlying bedrock. The natural soil consisted of very stiff to hard silty clay, sandy clay, and sandy silt and loose to medium dense silty sand and clayey sand. The bedrock consisted of highly weathered and fractured sandstone and siltstone of the Panoche Formation that extended to the maximum depth explored at the site (20.5 feet in Boring 5). Minor sandy claystone bedrock was encountered in Boring 10 from 4.0 to 6.0 feet deep. The bedrock became progressively stronger and more resistant with depth (drilling refusal was encountered in Borings 6, 9, 11, and 12 at depths of 10.5 to 15 feet in the hard sandstone bedrock).

The soil and bedrock materials have variable plasticity characteristics and have low to high expansion potentials (plasticity index = 8 to 30). The results of six Atterberg Limits tests are shown on the boring logs and on Drawing 19.

The attached boring logs and related information depict subsurface conditions only at the specific locations shown on Drawing 2 and on the particular dates designated on the logs. Soil and rock conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change of conditions at these boring locations due to environmental changes.

C. Groundwater

No free groundwater was encountered in the borings at the time of drilling. It should be noted, however, that fluctuations in the groundwater level may occur because of variations in rainfall, temperature, runoff, irrigation and other factors not evident at the time our measurements were made and reported herein.

CONCLUSIONS AND RECOMMENDATIONS

From a soil and foundation engineering standpoint, it is our opinion that the site is suitable for the proposed tract development provided the recommendations contained in this report are incorporated into the design and construction of the project. The gently to moderately inclined, rolling site is underlain by expansive soils to variable depths, consequently, we recommend that all residences and retaining walls be supported on well reinforced drilled pier and grade beam foundation systems. The drilled piers should be designed to obtain skin friction support in the bedrock materials that underlie the site. All concrete slabs should be underlain by a layer of non-expansive fill to minimize potential soil heave and shrinkage movements.

It is anticipated that conventional grading equipment can be used to grade the planned street and building pads. However, difficult drilling of the drilled pier holes may be encountered locally and we recommend that heavy duty drilling equipment in good working condition be used for the pier drilling. In addition, heavy duty trenching equipment and jackhammers may be required to excavate deep utility trenches in the planned street, such as the planned sewer trench. (Excavation contractors should review the boring logs and observe the bedrock outcrops at the site to evaluate the excavation characteristics of the bedrock).

Detailed recommendations for use in design and construction of the project are presented in the remainder of this report. These recommendations are contingent on our review of the earthwork and foundation plans for the project and our observation of the earthwork and foundation installation phases of construction.

A. Earthwork

1. Clearing and Site Preparation

Initially the site should be cleared of the residence, sheds, designated trees, brushy debris, and any other debris or underground obstructions encountered at this time. Any holes resulting from the removal of underground obstructions that extend below the planned finished grade should be cleared and backfilled with suitable material compacted to the requirements given below for engineered fill.

2. Recompaction of Surface Soils

After the site has been cleared and any underground obstructions removed and backfilled, the surface soils in areas to be filled should be recompacted. The recompaction should consist of ripping the upper eight inches, moisture conditioning the soils to approximately two percent above optimum and compacting them to at least 90 percent relative compaction as determined by ASTM Test

Designation D1557-78(C). Compaction should be performed using heavy compaction equipment such as a sheepsfoot roller or self-propelled compactor.

3. Placement of Fill on Slopes

Any fill placed for the road or buildings on slopes steeper than 6:1 (horizontal to vertical) should be keyed into firm undisturbed materials with a minimum key depth of three feet. As the fill is brought up, it should be benched into firm soil or rock with a series of two foot wide benches. The actual extent of keying and benching should be determined in the field by the soil engineer.

A subdrain should be placed at the back of the keyway in the planned fills across the swales as shown on Drawing 2. Details of the recommended keyway, subdrain, and bench installations are shown on Drawing 3 - Engineered Fill Section.

The outboard portion of the existing roadfill across the southern swale should be reworked in conjunction with the keying and benching operations for the new road. The inboard portion of the existing fill, although expected to be suitable in its present condition for support of the new road, should be tested during construction. The existing fill should have a minimum compaction of at least 90 percent relative compaction as determined by ASTM Test Designation D1557-78(C). If the fill does not meet 90 percent relative compaction, then the fill should be recompacted to at least 90 percent. The soil should be moisture-conditioned to about two percent above optimum and compacted in accordance with the recommendations presented below under Item A5, "Fill Placement and Compaction".

4. Slope Gradients

Permanent cut and fill slopes should be no steeper than 2:1 (horizontal to vertical). Cut and fill slopes should be planted to minimize erosion. Surface runoff should be diverted away from the top of slopes and carried to a suitable drainage collection system.

5. Fill Placement and Compaction

On-site soils having an organic content of less than three percent by volume can be used as fill. Fill material should not, however, contain rocks or lumps greater than six inches in greatest dimension with not more than 15 percent larger than 2.5 inches. All imported fill required at the site should be predominantly granular with a plasticity index of 12 or less.

Engineered fills should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557-78(C). Fill material should be spread and compacted in lifts not exceeding eight inches in uncompacted thickness. The moisture content of the soils utilized as fill should be adjusted to about two percent above their optimum moisture content.

Pavement subgrade and aggregate baserock in street and parking areas should be compacted to at least 95 percent relative compaction.

6. Trench Backfill

Pipeline trenches should be backfilled with engineered fill placed in lifts not exceeding eight inches in uncompacted thickness, except thicker lifts may be used with the approval of the soil engineer provided satisfactory compaction is achieved. If on-site soil is used, the material should be compacted to at least 85 percent relative compaction by mechanical means only. Imported sand can also be used for backfilling trenches provided it is compacted to at least 90 percent relative compaction. In pavement areas, the upper three feet of trench backfill should be compacted to at least 90 percent relative compaction for on-site soils, and 95 percent where imported sand backfill is used. In addition, the upper six inches of all trench backfill in pavement areas should be compacted to at least 95 percent relative compaction.

Crushed rock (3/4 inch maximum) can be used as trench backfill, particularly in the deeper portions of trenches, and as pipe bedding materials.

7. Drainage

Positive surface gradients of at least two percent should be maintained away from the structures so that water does not collect on slopes or in the vicinity of the building foundations. Water from roof downspouts, pavements, and slabs should be directed into drains and/or closed pipes and carried to suitable drainage facilities.

8. Construction Observation

The grading operations should be monitored and the earthwork should be tested by our representative for conformance with the project plans/specifications and our recommendations. This work includes site preparation, selection of satisfactory fill materials, and placement and compaction of the subgrade, baserock, and non-expansive fills. Sufficient notification prior to commencement of earthwork operations is essential to make certain that the work will be properly observed and tested.

B. Foundations

Drilled piers should be used to support the planned residences and retaining walls (except for certain low retaining walls as described below under Item D). The drilled pier foundations should consist of cast-in-place, straight shaft friction piers tied together with perimeter grade beams. Grade beams should be designed to span between drilled piers. Upslope-downslope tie beams spaced not more than 15 feet apart should be used to tie interior piers together. All piers should extend at least eight feet into the underlying bedrock. Piers should be spaced no closer than three diameters center to center and no further apart than eight to ten feet. The drilled piers should have a minimum diameter of 16 inches.

The portion of the drilled piers within bedrock may be designed on the basis of 500 psf skin friction with a 50 percent increase for wind and seismic conditions. Point bearing resistance should be neglected. For resistance to lateral loads, a passive equivalent fluid pressure of 350 pcf can be assumed to act over 1.5 times the projected area of the individual pier shaft. The passive pressure may be assumed to start at a depth where there is at least seven feet of horizontal confinement between the face of the pier and the edge of the nearest slope.

Because of the expansion potential of the on-site soils, we recommend that the grade beams be designed to withstand an uplift pressure of 1000 psf. Grade beams should be reinforced with at least 2 - #4 bars (top and bottom) reinforcement and as required to resist uplift pressure from the expansive subgrade materials.

The bottom of the pier excavations should be dry and relatively free of loose soil or fall-in prior to installing reinforcing steel and placing concrete. Since actual lengths of the piers may depend on the subsurface conditions encountered in the field, the excavation of piers should be performed under the observation of the soil engineer.

Addendum Attachment E-7/p.15

Reinforcement of the piers should be provided for their full length. Minimum pier reinforcement should consist of 4 - #5 bars tied in a cage. Greater reinforcement may be required as determined by the structural designer's analysis.

Hard drilling may be required to achieve the recommended pier hole depths. If drilling refusal is encountered, we should be consulted regarding possible alternate types of foundation support.

Settlements under building loads are expected to be within tolerable limits for the proposed construction.

C. Slabs-on-Grade

Slab-on-grade construction will be used for the planned garages and exterior slabs. We recommend that all slabs be supported on a minimum of nine inches of non-expansive fill consisting of granular soil with a plasticity index of twelve or less and no more than ten percent finer than #200 sieve. Reinforcement of slabs should be provided in accordance with their anticipated use and loading, but as a minimum, slabs should be reinforced with a 6x6 - 10/10 woven-wire mesh or number three bars at 18 inches on center, both ways.

Prior to final construction of slabs, the subgrade surface should be proofrolled to provide a smooth, firm support for the slab. In any areas where floor wetness would be undesirable, four inches of free draining gravel should be placed beneath the floor slab to serve as a capillary moisture break between the subgrade soil and the slab. In order to minimize vapor transmission, an impervious membrane should be placed over the gravel. The membrane should be covered with two inches of sand to protect it during construction. The sand should be lightly moistened just prior to placing the concrete. The sand, membrane and gravel can be used in lieu of six inches of the non-expansive fill required beneath slabs.

D. Retaining Walls

All retaining walls required for the project must be designed to resist lateral earth pressures and any additional lateral loads caused by surcharge loading. In general, walls should be supported on drilled pier foundations designed in accordance with our previous recommendations (Item B). However, any detached walls on level ground and less than three feet high may be supported on footings bearing in engineered fill, firm natural soil, or bedrock. The footings should have a minimum depth of 18 inches and width of 24 inches. The footings can be designed on the basis of 2000 psf allowable bearing pressure.

We recommend that unrestrained walls with level or sloping backfill no steeper than 4:1 be designed to resist an equivalent fluid pressure of 45 pcf. Walls with backfill sloping steeper than 4:1 should be designed to resist an equivalent fluid pressure of 60 pcf. Wherever walls will be subjected to areal surcharge loads, they should be designed for an additional lateral pressure equal to one-third the anticipated surcharge load.

Below grade retaining walls should be thoroughly waterproofed using two coats of hot mop asphalt or tar, or equivalent protection.

The preceding pressures assume that sufficient drainage is provided the retaining walls to prevent the build-up of hydrostatic pressures from surface or subsurface water infiltration. Adequate drainage may be provided by means of 3/4 inch drain rock material enclosed in a filter fabric, such as Mirafi 140, and a four inch diameter, perforated pipe placed at the base of the wall. The perforated pipe should be Schedule 40 PVC or equivalent and should be situated below interior finished floor grade, where applicable. The perforated pipe should be tied into a closed pipe and carried to a suitable discharge facility. Weepholes with drain rock material may be used instead of perforated pipe subdrains in detached walls.

Backfill placed behind retaining walls should be non-expansive and compacted to at least 90 percent relative compaction using light compaction equipment. If heavy compaction equipment is used, the walls should be appropriately temporarily braced.

E. Flexible Pavement

A sample of the surface soil along the planned street alignment was found to have an R-Value of 37 based on laboratory testing. Using an R-Value of 30 (reduced to account for variations in soil conditions), a Traffic Index of 4.5 for the street and parking areas as provided on the Preliminary Grading and Tentative Map, and Procedure 301-F of the California Department of Transportation, we recommend that the pavement section consist of two and one half (2 1/2) inches asphaltic concrete over six (6) inches Class 2 Aggregate Base.

The upper six inches of soil subgrade should be compacted to at least 95 percent within areas to be paved. Any fill required below the upper six inches of subgrade should be compacted to at least 90 percent.

Class 2 Aggregate Base should have an R-Value of at least 78 and conform to the requirements of Section 26, State of California "Cal Trans" Standard Specifications, latest edition. The aggregate base material should be placed in thin lifts in a manner to prevent segregation, and should be uniformly moisture conditioned and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface.

PLAN REVIEW AND CONSTRUCTION OBSERVATION

We recommend that we review the foundation and grading plans and specifications for the project. We should also be retained to provide monitoring and testing services during the grading and foundation installation phases of the project. This will provide the opportunity for correlation of the soil and rock conditions found in the investigation with those actually encountered in the field, and thus permit any necessary modifications in our recommendations resulting from changes in anticipated conditions.

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