REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED AUTUMN CARE
A RESIDENTIAL CARE FACILITY
SOUTHWEST CORNER OF GARFIELD AVENUE AND BROOKHURST STREET
HUNTINGTON BEACH, CALIFORNIA 92648

FOR

AUTUMN CARE DEVELOPMENT PARTNERS

PROJECT NO. 12-388-02

NOVEMBER 12, 2012

November 12, 2012

12-388-02

Autumn Care Development Partners
2700 Mandeville Canyon Road
Los Angeles, California 90049

Attention: Mr. Al Cabraloff

Subject: Report Of Geotechnical Investigation
Proposed Autumn Care
A Residential Care Facility
Southwest Corner Of Garfield Avenue And Brookhurst Street
Huntington Beach, California 92648

Gentlemen:

INTRODUCTION

This report presents the results of a geotechnical investigation for the subject project. During the course of this investigation, the engineering properties of the subsurface materials were evaluated in order to provide recommendations for design and construction of foundations. The investigation included subsurface exploration, soil sampling, laboratory testing, engineering evaluation and analysis, consultation and preparation of this report.

During the course of our investigation, the pervious geotechnical investigation report prepared by the offices of Foundation Engineering Consultants, Inc. (FEC), was used as reference. The report was dated August 22, 2012.

The enclosed Drawing No. 1, entitled Site Plan, shows the approximate locations of the exploratory borings in relation to the site boundaries and the proposed building. This drawing also shows the approximate locations of the borings drilled by FEC and approximate location of Cross Section A-A'. Drawing No. 2 shows the profile of the Cross Section A-A'.

Figure No. 1 shows the Seismic Hazard Zone Map. Figure No. 2 shows the Regional Geologic Maps. Figure Nos. 3 through 7 show the associated Site Location Maps, Seismic Hazard Maps as well as the Historically Highest Groundwater Contour Map of the site.

The attached Appendix I, describes the method of field exploration. Figure Nos. I-1 and I-2 present summaries of the materials encountered at the location of our exploratory test holes. Figure No. I-3 presents a key to the log of exploratory borings.

The attached Appendix II describes the laboratory testing procedures. Figure Nos. II-1 and II-2 present the results of direct shear and consolidation tests performed on selected undisturbed samples.

PROJECT CONSIDERATIONS

It is our understanding that the subject project will consist of construction of an assisted living complex at the subject site. At this time, two grading/construction options are being considered. The options are as follows:

Option No. 1 A 2-story wood frame structure constructed near grade (with the finished grade established near the levels of the existing Wallgreen parking lot to the north;

Option No. 2 Construction of a 2-story wood frame structure over basement, the grade of which will be established at some 10 feet below the level of the existing Wallgreen parking lot to the north;

The approximate location of the proposed building with respect to site boundaries and off-site improvements are shown on the enclosed Site Plan; Drawing No. 1. The profiles of the proposed building for both options are shown on the enclosed Cross Sections A-A' and A-A"; Drawing Nos. 2 and 3, respectively.

Structural loading data was not available during the course of preparation of this report. For the purpose of this report, however, it is assumed that the magnitude of the collected load (combined dead, plus frequently applied live loads) for Option Nos. 1 and 2 would be on the order of 100 kips and 250 kips, respectively,. Wall loads for Option Nos. 1 and 2 would be are expected to be on the order of 3 and 9 kips per lineal foot, respectively.

ANTICIPATED SITE GRADING WORK

For Option No. 1, site grading will involve raising the grade within the western portion of the site. This will require construction of a retaining wall along the west property line which will be backfilled with new fill.

For Option No. 1, imported soils will be required to accomplish the site grading work. All imported soils should be non-expansive and granular in nature. The granular soils should be placed within the upper portion of the new fill. Before any fill is placed on the site, the existing fill should be removed until native soils are exposed. The zone of removal and recompaction should be extended beyond the footprint of the proposed building a horizontal distance equal to the thickness of fill. All new fill should be constructed over native soils. The new fill will be used for support of structural foundations and grade slabs.

Although the existing fill is granular in nature, the native soils are fine grained and potentially expansive. Therefore, it is recommended that the site native soils, when excavated, be removed from the site and not be used in the areas of new fill.

For Option No. 2, site grading will involve excavation in order to create the proposed basement garage level. As part of the site grading work, subgrade preparation will be made to stabilize the finished grade for support of thickened slab "mat". For Option No. 2, site grading will also involve backfilling within the over-excavated areas behind the exterior walls of the basement garage. All wall backfill materials should consist of non-expansive/granular soils.

For both options, as part of the site grading work, the area of the proposed building should be pre-loaded, to remove the major portions of the post construction settlement. See the enclosed Drawing Nos. 2 and 3 for the profiles of the pre-loading for both options. The soils associated with pre-loading should be packed, but not necessarily compacted to 90 percent relative compaction. Also, random soils (gravel/sand/silt/clay).

Settlement plates should be installed at the finished grades of the pre-loading mass for subsequent monitoring.

SITE CONDITIONS

SITE SURFACE CONDITIONS

The subject site is located at the southwest corner of Garfield Avenue and Brookhurst Street. Huntington Beach, California. See the enclosed Site Plan; Drawing No. 1, for site location with respect to the existing, off-site improvements.

At the time of our investigation, the site was vacant and covered with grass. The ground surface rises some 9 feet from west to east side.

SUBSURFACE CONDITIONS

Correlation of the subsoil between the test borings was considered to be very good. Generally, the site, to the depths explored, was found to be underlain by existing fill underlain by natural deposits of mainly fine grained soils (silt-clay) with slight to little sand. Slightly organic pockets/lenses, were also found within the upper 20 feet of the subsoils. Native sand soils (silty sand) was found only in our Boring No. 1 near a depth of about 47 feet.

Thickness of the existing fill was found to range from 3 feet to as much as 6 feet in our borings drilled within the higher elevation of the site The lower portion of the site was not accessible by the drill rig). The original grade within the eastern portion, seem to have been raised previously.

The existing fill is considered to be inadequate for support of foundations and grade slabs at their present state. The existing fill, however, can be excavated and reused in the areas of new compacted fill. The new fill will support grade slabs and foundations.

The underlying native soils were found to be slightly firm to firm in-place. The ground firmness increased only slightly with depth. The results of our laboratory testing indicated that the site native soils were of relatively low strengths and were compressible.

The site native soils can receive new fill for support of structural foundations and grade slabs, only after proper pre-loading. The pre-loading will help reduce the magnitude of the post-construction settlement.

The existing fill soils were granular in nature (silty sand) and non-expansive. However, the underlying native soils were found to be fine grained in nature and expansive. The expansion index of the site upper soils were found to be 112.

During the course of our investigation, groundwater was encountered in our borings near a depth of about 7 feet. Due to method of drilling (use of continuous casing) caving was not detected in our borings.

GROUNDWATER CONDITION

Groundwater was encountered in our borings near a depth of about 7 feet. For Option 2, assuming a 10 feet excavation to reach the basement garage grade, water will be intercepted during the course of basement garage excavation. On this basis, de-watering (temporary and permanent) will be required for Option No. 2. Because of clayey nature and relatively low permeability coefficient, it is anticipated that temporary de-watering may be accomplished by creating small sumps extending at least 3 feet below the base of the footings (in the needed areas) to collect water.

The permanent de-watering can be in a form of a gravel bed below the "mat" contained a network of pipes carrying the collected water to a sump and then pumped to the street. For added safety, it is further recommended that the "mat" be designed based on hydrostatic uplift forces assuming a hydraulic gradient equivalent to 5 feet of water head. The bottom of the "mat" should be properly waterproofed in order to reduce the chances of dampness on the slab surface.

CAVING CONSIDERATIONS

Although not detected during our exploration, caving may be experienced in the drilled piles (supporting the retaining wall along the west property line). Within typical clay soils, however, drilling can be extended ahead of water. Any collected water at the bottom of the hole should be pumped out before concrete is placed. Alternatively, "treme" should be used to place the concrete from the bottom of the hole while dispersing water. In any case, casing should be available in the site during installation of the production piles. Also, when placing concrete below water, the strength of the concrete should be taken 1,000 psi higher than the project specifications.

SEISMIC DESIGN CONSIDERATIONS

In accordance with the 2010 California Building code (CBC 2010), the project site can be classified as site "E". The mapped spectral accelerations of S_S=1.665 (short period) and S_1 =0.599 (1-second period) can be used for this project. These parameters corresponds to site Coefficients values of F_a=0.90 and F_V=2.4, respectively.

The seismic design parameters would be as follows:

Sms= Fa (Ss) = 0.9 (1.665) = 1.499

Sm1=Fv (S1) = 2.4 (0.599) =1.438

Sds=2/3 (Sms) = 2/3 (1.499) = 1.00 and

Sd1=2/3 (Sm1) = 2/3 (1.438) = 0.959

EVALUATION OF LIQUEFACTION POTENTIAL

As part of our field exploration, two borings were drilled at the subject site to a maximum depth of 51.5 feet. Water was encountered in our borings near depths of about 6 to 7 feet. The available maps indicate that the historically highest groundwater level at the site was near a depth of about 5 feet. For the purpose of evaluating liquefaction potential at the site, therefore, SPT (Standard Penetration Test) were conducted from a depth of 5 feet.

The results of our liquefaction analysis using ground acceleration of 0.44g (2/3 of the Sds value of 1.11 above) and a moment magnitude of 6.9, indicated that the soil layers below the historically highest groundwater level have factors of safety greater than 1.1, against potential liquefaction. See the enclosed Engineering calculation sheets. On this basis, soil liquefaction will not occur at this site. However, the associated accumulated settlement at the ground surface associated with strong ground motion earthquake is estimated to be on the order of 0.12 inches. See the enclosed engineering calculation sheet. The magnitude of the differential settlement associated with seismic strong ground motion would be 2/3 of the value of 0.12, which is 0.08 inches. These values should be added to the settlements associated with gravity loads.

EVALUATION AND RECOMMENDATIONS

GENERAL

Based on the geotechnical engineering data derived from this investigation, the site can be developed as planned. For Option No. 1, spread footing foundation system can be used for support of the proposed building. The foundation bearing materials for Option No. 1, will consist of properly compacted fill soils which has been pre-loaded. For pre-loading configuration, see the enclosed Drawing No. 2 (Cross Section A-A' for Option No. 1). For Option No. 1, however, the new retaining wall along the west property line (supporting new backfill) should be supported through a deep foundation system consisting of friction piles.

For Option No. 1, before any fill is placed on the site, the existing fill should be shaved until firm native soils are exposed. The zone of removal and recompaction should be extended beyond the footprint of the proposed building a horizontal distance equal to the thickness of fill. All new fill should be constructed on native soils. The site native soils were found to be very moist and clayey in nature. Therefore, it is recommended that the site native soils, if excavated, no be used within the areas of new fill. For this option, imported soils will be required to accomplish the site grading work. All imported soils should be non-expansive and granular in nature. Before new fill is placed over the native soils, the bottom of fill should be stabilized. This will require placement of a gravel bed over the very moist, clayey soils, before the new, granular imported soils are placed. It is anticipated that the thickness of the gravel to stabilize the soft native soils will be on the order of 12 inches (the actual requirement for gravel thickness should be determined during the course of site grading work). Alternatively, a combination of gravel bed and lime/cement treated base can be used. In order to reduce the chances of sinking, when excavating soft soils, light equipment should be used. Grade slabs for Option No. 1, can be supported on the finished grades which would be properly compacted granular fill soils. For the purpose of this project, it is recommended that the grade slabs to have a minimum thickness of 5 inches and be reinforced with # 4 bars placed at every 18 inches on center.

For Option No. 2, the support system should be on a form of a thickened slab "mat" foundation. The "mat" should be supported on a gravel bed having a minimum thickness of 15 inches. The "mat" should also be designed based on hydrostatic uplift force assuming water at a depth of 5 feet below the "mat". For Option No. 2, before the gravel associated with "mat" is placed, the western portion of the site should be pre-loaded. For pre-loading configuration for Option No. 2, see the enclosed Drawing No. 3.

The following sections present our recommendations for temporary excavation, site grading, site drainage, foundations, lateral design, grade slabs, retaining walls, and observation during construction.

TEMPORARY EXCAVATION

Where space limitations permit, unshored temporary excavation slopes could be used. Based upon the engineering characteristics of the site upper soils, it is our opinion that temporary excavation slopes in accordance with the following table should be used:

Maximum Depth of Cut (Ft)	Maximum Slope Ratio (Horizontal:Vertical)
0-4	Vertical
>4	3/4:1

Water should not be allowed to flow over the top of the excavation in an uncontrolled manner. No surcharge should be allowed within a 45-degree line drawn from the bottom of the excavation. Excavation surfaces should be kept moist but not saturated to retard raveling and sloughing during construction.

It would be advantageous, particularly during wet season construction, to place polyethylene plastic sheeting over the slopes. This will reduce the chances of moisture changes within the soil banks and material wash into the excavation.

It should be noted that the recommendations presented in this section are for use in design and for cost estimating purposes prior to construction. The contractor is solely responsible for safety during construction.

GRADING RECOMMENDATIONS

For Option No. 1, site grading will involve raising the present grade to establish the proposed finished grades. In order to accomplish the site grading work, a pile supported retaining wall will need to be constructed along the west property line and backfilled. Before placing any fill on the site, the existing fill should be shaved until firm native soils are exposed. After proper subgrade stabilization, non-expansive, granular soils should be placed back and compacted, under engineering observation and testing, until the proposed finished grades are established. In order to reduce the magnitude of the post-construction settlement, the compacted core should be pre-loaded by packed/random soils. See the enclosed Drawing No. 2 (Cross Section A-A') for pre-loading configuration. Imported soils will be required to accomplish the site grading work. All imported soils should be non-expansive and granular in nature. The zone of removal and recompaction should be extended beyond the footprint of the proposed building a horizontal distance equal to the thickness of fill. Prior to placement of any new fill on the site, the fill bottom should be stabilized under observation of the Soil Engineer. It is anticipated that, in order to reduce pumping, a gravel bed with an estimated thickness of 18 inches (the actual requirement for gravel thickness should be determined during the course of site grading work) will be required. Alternatively, a combination of gravel bed and lime/cement treated base can be used.

In order to reduce the chances of equipment sinking, when excavating soft soils, light machines should be used.

All imported soils should be non-expansive and granular in nature and be free of organic and rocks larger than 6 inches in diameter. The granular fill should be placed within the upper portion of the fill. Before import soils are brought to the site, a 20-pound sample of the proposed import soils should be submitted to the Soil Engineer (at least 48 hours in advance) so that the maximum density and expansion character of the import materials can be determined.

For Option No. 2, site grading will involve excavation in order to create the proposed basement garage level. As part of the site grading work, subgrade preparation will be made to stabilize the finished grade for support of thickened slab "mat". For Option No. 2, site grading will also involve backfilling within the over-excavated areas behind the exterior walls of the basement garage. All wall backfill materials should consist of non-expansive/granular soils.

For both options, as part of the site grading work, the area of the proposed building should be pre-loaded, to remove the major portions of the post construction settlement. See the enclosed Drawing Nos. 2 and 3 for the profiles of the pre-loading for both options. The soils associated with pre-loading should be packed, but not necessarily compacted to 90 percent relative compaction. Also, random soils (gravel/sand/silt/clay).

General guidelines for grading are presented below in an itemized form which may be included in the earthwork specification. It is recommended that all fill be placed under engineering observation and in accordance with the following guidelines:

1. Only the excavated granular soils should be reused within the compacted fill core to be used for support of structural foundation and grade slabs.

2. Before wall backfilling, subdrain should be installed. The subdrain system should consist of 4-inch diameter perforated pipes embedded in about 1 cubic feet of free draining gravel per foot of pipe. An approved filter fabric should then be wrapped around the free draining gravel in order to reduce the chances of siltation. Non-perforated outlet pipes should then be used to pass through the wall into an interior sump. The subdrain pipes should be laid at a minimum grade of two percent for self cleaning.

3. Fill soils, approved by the Soil Engineer, should be placed in controlled layers. Each layer shall be compacted to at least 90 percent of the maximum unit weight as determined by ASTM designation D 1557 for the material used.

4. New fill shall be placed in layers which, when compacted, shall not exceed 8 inches per layer. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to insure uniformity of material in each layer.

5. When moisture content of the fill material is too low, water shall be added and thoroughly dispersed until the moisture content is near optimum. When the moisture content of the fill material is too high to obtain adequate compaction, the fill material shall be aerated by blading or other satisfactory methods until near optimum moisture content is achieve.

6. Inspection and field density tests should be conducted by the Soil Engineer during grading work to assure that adequate compaction is attained. Where compaction of less than 90 percent is indicated, additional compactive effort should be made with adjustment of the moisture content or layer thickness, as necessary, until at least 90 percent compaction is obtained.

SURFACE DRAINAGE

Adequate site drainage should be provided to divert roof and surface waters away from the proposed building and from the property through non-erodible drainage devices. In no case should the surface waters be allowed to pond adjacent to the buildings or anywhere on the building pads. A minimum surface slope of one and two percent are recommended for paved and unpaved areas, respectively.

FOUNDATIONS

For Option No. 1, conventional spread footing foundation system can be used for support of the proposed building. The foundation bearing materials for this option will be properly compacted (certified) fill which has been pre-loaded. The pre-loading (with configuration as shown on Drawing No. 2), should remain over the area of the proposed building for a period of at least seven months.

Exterior and interior footings should be at least 18 inches wide and should be placed at a minimum depth of 24 inches below the lowest adjacent final grades. The recommended allowable maximum bearing pressure for foundations placed in properly compacted fill soils may be taken as 1,800 pounds per square foot. This value is for the total of dead and frequently applied live loads. For short duration transient loading, such as wind or seismic forces, the given value may be increased by one-third.

Under the allowable maximum soil pressure, the magnitude of the post construction total settlement of footings carrying the assumed maximum concentrated loads of 100 kips is expected to settle on the order of 1.25 inches. Continuous footings, with loads of about 3 kips per lineal foot are expected to settle on the order of 7/8 of one inch. Maximum differential settlements relating to gravity loads are expected to be on the order of 1/2 of one inch. Total and differential settlements from all causes are expected to be less than 1.5 inches, and 0.75 inches, respectively.

For Option No. 1, the new retaining wall along the west property line (supporting new backfill) should be supported through a deep foundation system consisting of friction piles. The friction piles should have a minimum length of 15 feet.

For the purpose of estimating vertical capacity of individual piles, an allowable maximum skin friction value of 375 pounds per square foot may be used for the native soils. This value is for the total of dead, plus frequently applied live loads. For short duration transient loading; wind or seismic forces, the given skin friction value may be increased by one third.

For friction pile design, the weight of the shafts can be assumed to be taken by end-bearing, therefore, need not be added to the structural loads. All piles should be concreted as soon as they are excavated and, for safety, should not be left open overnight .

During the course of our field investigation, caving was not detected (due to the method of drilling which was use of continuous casing. However, caving may be experienced in the open holes below water. Therefore, casing should be available at the job site during installation of the production piles. Any collected water at the bottom of the holes should be pumped out before concrete is placed. Alternatively, "treme" should be used to place the concrete from the bottom of the hole to disperse the water. Also, when placing concrete below water, the strength of the concrete should be taken 1,000 psi higher than the project specifications.

The magnitude of the post construction total settlement of the pile-supported retaining wall is expected to be on the order of one inch. Maximum differential settlement is expected to be on the order of 3/4 of one inch.

For Option No. 2, the support system should be on a form of a thickened slab "mat" foundation. The "mat" should have a minimum thickness of 12 inches and be supported on a gravel bed having a minimum thickness of 15 inches. The "mat" should also be designed based on hydrostatic uplift force assuming water at a depth of 5 feet below the "mat". For Option No. 2, before the gravel associated with "mat" is placed, the western portion of the proposed building should be pre-loaded. The pre-loading (with configuration as shown on Drawing No. 3), should remain over the area of the proposed building for a period of at least six months. The "mat" should be designed as "beam on elastic foundation" using a modulus of subgrade reaction of 150 kips per cubic foot.

Total settlement at the center of "mat" is expected to be on the order of 1.25 inches. The magnitude of the settlement at the edges and corners of the "mat" are expected to be on the order of 1 inch and 0.75 inches, respectively. These settlements are expected to be time dependent.

LATERAL DESIGN

For Option No. 1, the lateral resistance at the base of footings in contact with compacted fill soils may be assumed to be the product of the dead load forces and a coefficient of friction of 0.30. Passive pressure on the face of footings may also be used to resist lateral forces. A passive pressure of zero at the finished grades and increasing at a rate of 250 pounds per square foot per foot of depth to a maximum value of 1,500 pounds per square foot may be used for footings poured against properly compacted fill soils.

For the vertical shafts supporting the west property line retaining wall which will be cast against the native soils, the magnitude of the passive pressure should be limited to 150 pounds per square foot per foot of depth. For piles having center-to-center spacing of greater than 3.5 time the pile diameter, the given passive pressure can be doubled. For the purpose of moment calculations, the point of fixity of the vertical shafts may be assumed at some 5 feet below the bottom of the pile cap.

For Option No. 2, the lateral resistance at the base of "mat" in contact with native soils may be assumed to be the product of the dead load forces and a coefficient of friction of 0.20. Passive pressure an the edges of "mat" may also be taken at a rate of 125 pounds per square foot per foot of depth to a maximum value of 1,200 pounds per square foot.

GRADE SLABS

Grade slabs, for Option No. 1, can be supported on the finished grades which would be properly compacted granular fill soils. For the purpose of this project, it is recommended that the grade slabs to have a minimum thickness of 5 inches and be reinforced with # 4 bars placed at every 18 inches on center.

In the areas where moisture sensitive floor covering is used and slab dampness can not be tolerated, moisture barrier should be used beneath the grade slabs. This normally consists of plastic sheeting covered with 2 inches of sand.

RETAINING WALLS

For Option No. 1, static design of cantilevered retaining walls supporting granular backfill, and level ground condition can be based on an equivalent fluid pressure of 40 pounds per square foot per foot of depth. This assumes that no hydrostatic pressure will occur behind the retaining walls.

This will require that proper subdrain be installed behind the basement garage walls. Subdrain normally consists of 4-inch diameter perforated pipes encased in free-draining gravel (at least one cubic foot per lineal foot of the pipes).

In order to reduce the chances of siltation and drain clogging, the free-draining gravel should be wrapped in filter fabric proper for the site soils. The backs of all retaining walls should be properly waterproofed.

In addition to the earth pressure, the retaining walls should also be designed for any applicable uniform surcharge loads by the proposed building and parking/motor court traffic. Uniform surcharge effects may be computed using a coefficient of 0.30 times the assumed uniform loads.

It is noted that, based on the new Code requirement, the retaining walls be designed not only for static, but also for seismic lateral earth pressures. For the purpose of this project, the magnitude of seismic lateral earth pressure should be assumed 20 pounds per square foot (½ of static lateral active pressure of 40 pounds per square foot per foot of depth) with reverse triangular shape (maximum at top to zero at the bottom of the wall). The point of application of the lateral thrust should be assumed 0.6 time the wall height, measured from the bottom of the wall.

For Option No. 2, static design of the perimeter walls of the basement garage which are restrained against rotation at their tops can be based on an "at rest" pressure with a magnitude of 120 pounds per square foot per foot of depth. This assumes that no hydrostatic pressure will occur behind the retaining walls. This will require installation of subdrain behind the walls and discussed above.

In addition to the lateral earth pressure, the basement garage walls should also be designed for any applicable uniform surcharge loads imposed from on the adjacent grounds (parking, building, etc.). Uniform surcharge effects may be computed using a coefficient of 0.40 times the assumed uniform loads.

The basement walls should be designed not only for static, but also for seismic lateral earth pressures. For the purpose of this project, the magnitude of seismic lateral earth pressure should be assumed zero at the base of the wall and increased upward at a rate of 60 pounds per square foot per decreasing depth to a maximum value at the ground surface. This can be considered ultimate and be added to the static lateral earth pressure value. The point of application of the lateral thrust of the seismic pressure should be assumed 0.6 time the wall height, measured from the bottom of the wall.

Where adequate space is available, granular fill (silty sand soils) should be placed and compacted behind the retaining walls (after the subdrain is installed) to a relative compaction of at least 90 percent. At least one field density tests should be taken for each 2 feet of the backfill. The degree of compaction of the wall backfill should be verified by the Soil Engineer.

Where space is limited, free-draining gravel should be placed behind the retaining walls. The gravel should then be capped with at least 18 inch thick site soils also compacted to a relative compaction of at least 90 percent. It should be noted that the backfill placed behind the basement garage walls should be made after the concrete decking is cast. All grading surrounding the building should be such to ensure that water drains freely from the site and does not pond.

CONSTRUCTION PROCEDURES FOR OPTION 1

1. Build retaining wall on west property line on piles.
2. Remove (shave) fill until firm native soil is exposed.
3. Place an 18" thick gravel bed on native soil to stabilize fill base.
4. Place new compacted soil, benched into native soil, using non-expansive granular materials, to rough finish grade, in 6"-8" lifts. This portion of the fill should be controlled, tested and certified by a representative of the soil engineer.
5. Preload the area using packed soil (need not be certified compacted).
6. Install settlement plates.
7. Monitor settlement until it reaches the estimated maximum settlement.
8. Remove the pre-loaded packed fill soils.
9. Excavate foundations in certified fill.

OBSERVATION DURING CONSTRUCTION

The presented recommendations in this report assume that all structural foundations will be established in properly compacted fill soils and the retaining wall along the west property line will be supported through deep foundation (for Option No. 1) and "mat" foundation for Option No. 2. All foundation excavations (spread footings and piles) should be observed by a representative of this office before reinforcing is placed. It is essential to assure that all excavations are made at proper dimensions, are established in the recommended bearing material and are free of loose and disturbed soils.

Site grading work should be conducted under observation and testing by a representative of this firm. All backfill soils should be properly compacted to at least 90 percent relative compaction. For proper scheduling, please notify this office at least 24 hours before any observation work is required.

CLOSURE

The findings and recommendations presented in this report were based on the results of our field and laboratory investigations combined with professional engineering experience and judgment. The report was prepared in accordance with generally accepted engineering principles and practice. We make no other warranty, either express or implied.

It is noted that the conclusions and recommendations presented are based on exploration "window" borings and excavations which is in conformance with accepted engineering practice. Some variations of subsurface conditions are common between "windows" and major variations are possible.

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The following Figures and Appendices are attached and complete this report:

Engineering Calculation Sheets Relating To Liquefaction
Site Plan - Drawing No. 1
Cross Section A-A' (Option No. 1) - Drawing No. 2
Cross Section A-A" (Option No. 2) - Drawing No. 3
- Figure No. 1 - Seismic Hazard Zone Map
- Figure No. 2 - Regional Geologic Maps
- Figure No. 3 - Site Location Map (Portion Of Thomas Bros. Maps)
- Figure No. 4 -Site Location Map (Portion Of Assessor Maps)
- Figure No. 5 - Seismic Hazard Zone Map (Peak Ground Acceleration For Alluvium Conditions)
- Figure No. 6 - Seismic Hazard Zone Map (Predominant Earthquake Magnitude And Distance)
- Figure No. 7 - Historically Highest Groundwater Contour Map
  - Appendix I-Method of Field Exploration
    - Figure Nos. I-1 through I-3
  - Appendix II-Methods of Laboratory Testing
    - Direct Shear Test - Figure No. II-1
    - Consolidation Tests - Figure No. II-2

Respectfully submitted,

Applied Earth Sciences

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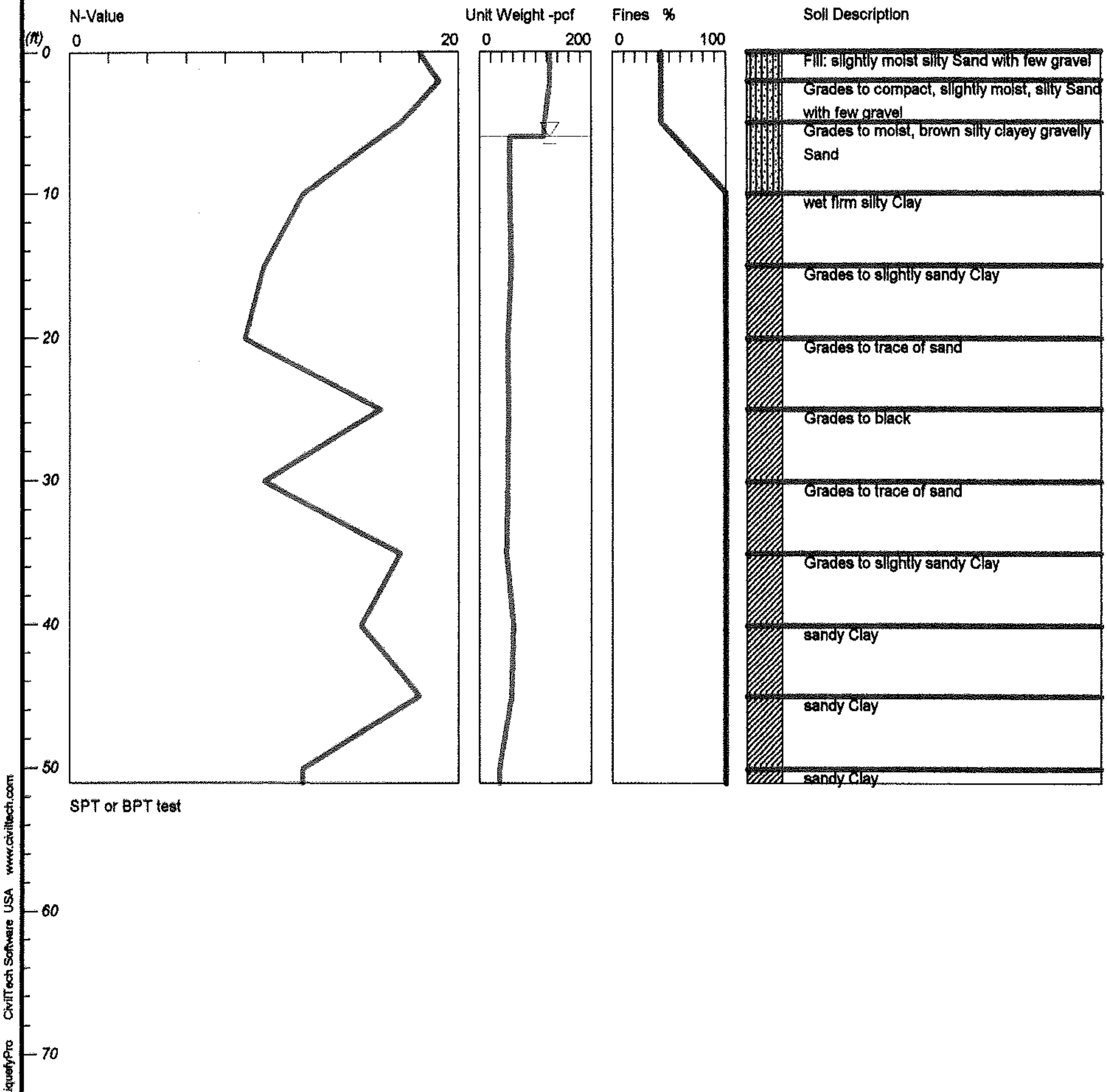
Distribution: (5)

LIQUEFACTION ANALYSIS

12-388-02 SW Corner of Garfield Ave & Brookhurst S

Hole No.=2 Water Depth=6 ft

Magnitude=6.9
Acceleration=.44g

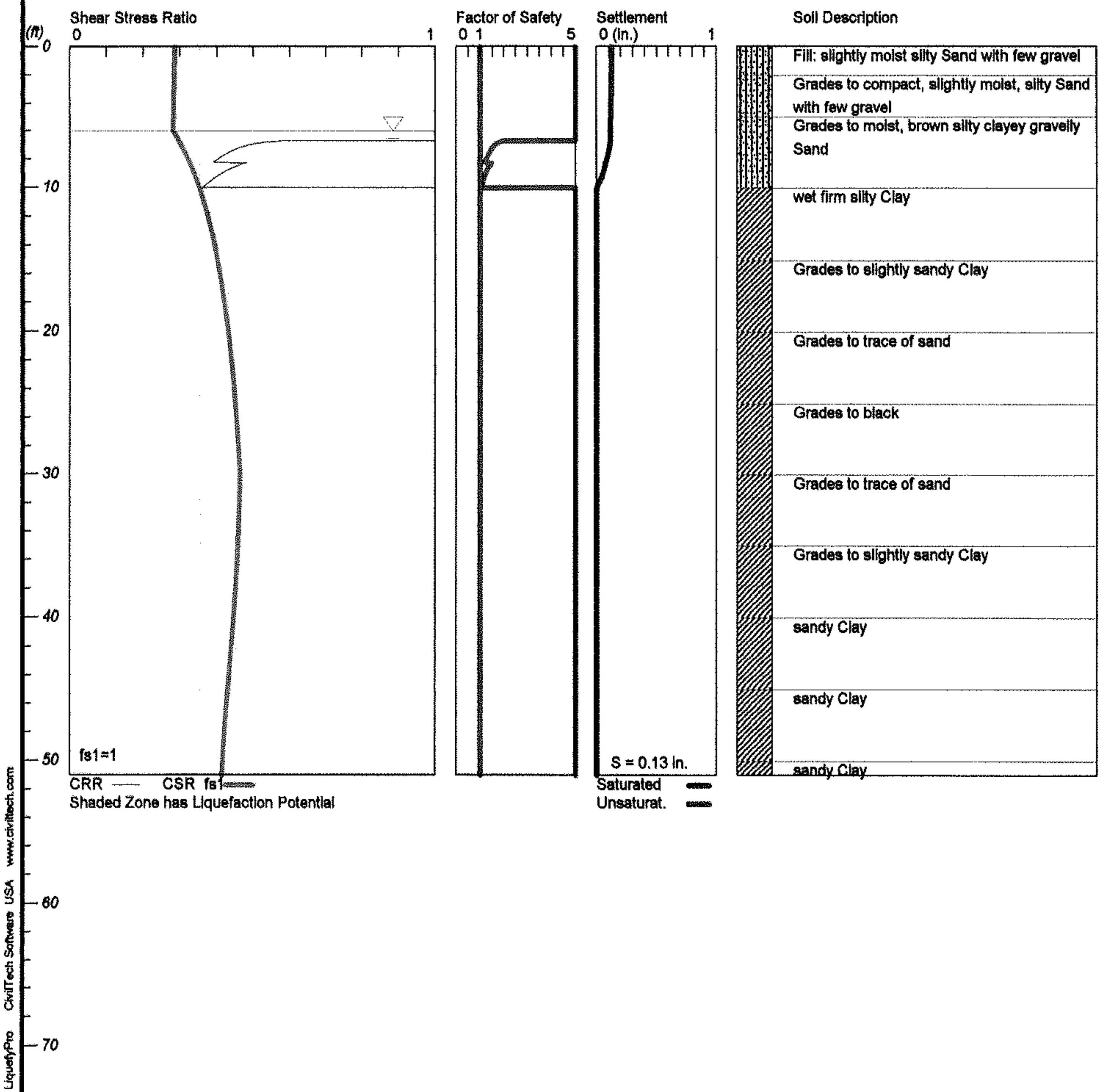


LIQUEFACTION ANALYSIS

12-388-02 SW Corner of Garfield Ave & Brookhurst S

Hole No.=2 Water Depth=6 ft

Magnitude=6.9
Acceleration=.44g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: G:\12-388-02 Garfield Ave - Brookhurst St No.2.liq
Title: 12-388-02 SW Corner of Garfield Ave & Brookhurst S
Subtitle: Southwest Corner of Garfield Ave & Brookhurst St

Surface Elev.=
Hole No.=2
Depth of Hole= 51.00 ft
Water Table during Earthquake= 6.00 ft
Water Table during In-Situ Testing= 6.00 ft
Max. Acceleration= 0.44 g
Earthquake Magnitude= 6.90

Input Data:

Surface Elev.=
Hole No.=2
Depth of Hole=51.00 ft
Water Table during Earthquake= 6.00 ft
Water Table during In-Situ Testing= 6.00 ft
Max. Acceleration=0.44 g
Earthquake Magnitude=6.90
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: Post Liquefaction
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.2
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1
   Plot one CSR curve (fs1=1)
10. Use Curve Smoothing: Yes*

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	18.00	126.00	43.00
2.00	19.00	126.00	43.00
5.00	17.00	116.00	43.00
10.00	12.00	117.00	NoLiq
15.00	10.00	119.00	NoLiq
20.00	9.00	113.00	NoLiq
25.00	16.00	114.00	NoLiq
30.00	10.00	113.00	NoLiq
35.00	17.00	111.00	NoLiq
40.00	15.00	123.00	NoLiq
45.00	18.00	120.00	NoLiq

50.00	12.00	98.00	NoLiq

Output Results:

Settlement of Saturated Sands=0.11 in.
Settlement of Unsaturated Sands=0.01 in.
Total Settlement of Saturated and Unsaturated Sands=0.13 in.
Differential Settlement=0.063 to 0.083 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.48	0.29	5.00	0.11	0.01	0.13
2.00	2.48	0.28	5.00	0.11	0.01	0.12
4.00	2.48	0.28	5.00	0.11	0.01	0.12
6.00	2.48	0.28	5.00	0.11	0.00	0.11
8.00	0.40	0.32	1.24	0.08	0.00	0.08
10.00	2.00	0.35	5.00	0.00	0.00	0.00
12.00	2.00	0.38	5.00	0.00	0.00	0.00
14.00	2.00	0.39	5.00	0.00	0.00	0.00
16.00	2.00	0.41	5.00	0.00	0.00	0.00
18.00	2.00	0.42	5.00	0.00	0.00	0.00
20.00	2.00	0.43	5.00	0.00	0.00	0.00
22.00	2.00	0.44	5.00	0.00	0.00	0.00
24.00	2.00	0.45	5.00	0.00	0.00	0.00
26.00	2.00	0.45	5.00	0.00	0.00	0.00
28.00	2.00	0.46	5.00	0.00	0.00	0.00
30.00	2.00	0.46	5.00	0.00	0.00	0.00
32.00	2.00	0.46	5.00	0.00	0.00	0.00
34.00	2.00	0.46	5.00	0.00	0.00	0.00
36.00	2.00	0.46	5.00	0.00	0.00	0.00
38.00	2.00	0.45	5.00	0.00	0.00	0.00
40.00	2.00	0.45	5.00	0.00	0.00	0.00
42.00	2.00	0.44	5.00	0.00	0.00	0.00
44.00	2.00	0.43	5.00	0.00	0.00	0.00
46.00	2.00	0.43	5.00	0.00	0.00	0.00
48.00	2.00	0.42	5.00	0.00	0.00	0.00
50.00	2.00	0.42	5.00	0.00	0.00	0.00

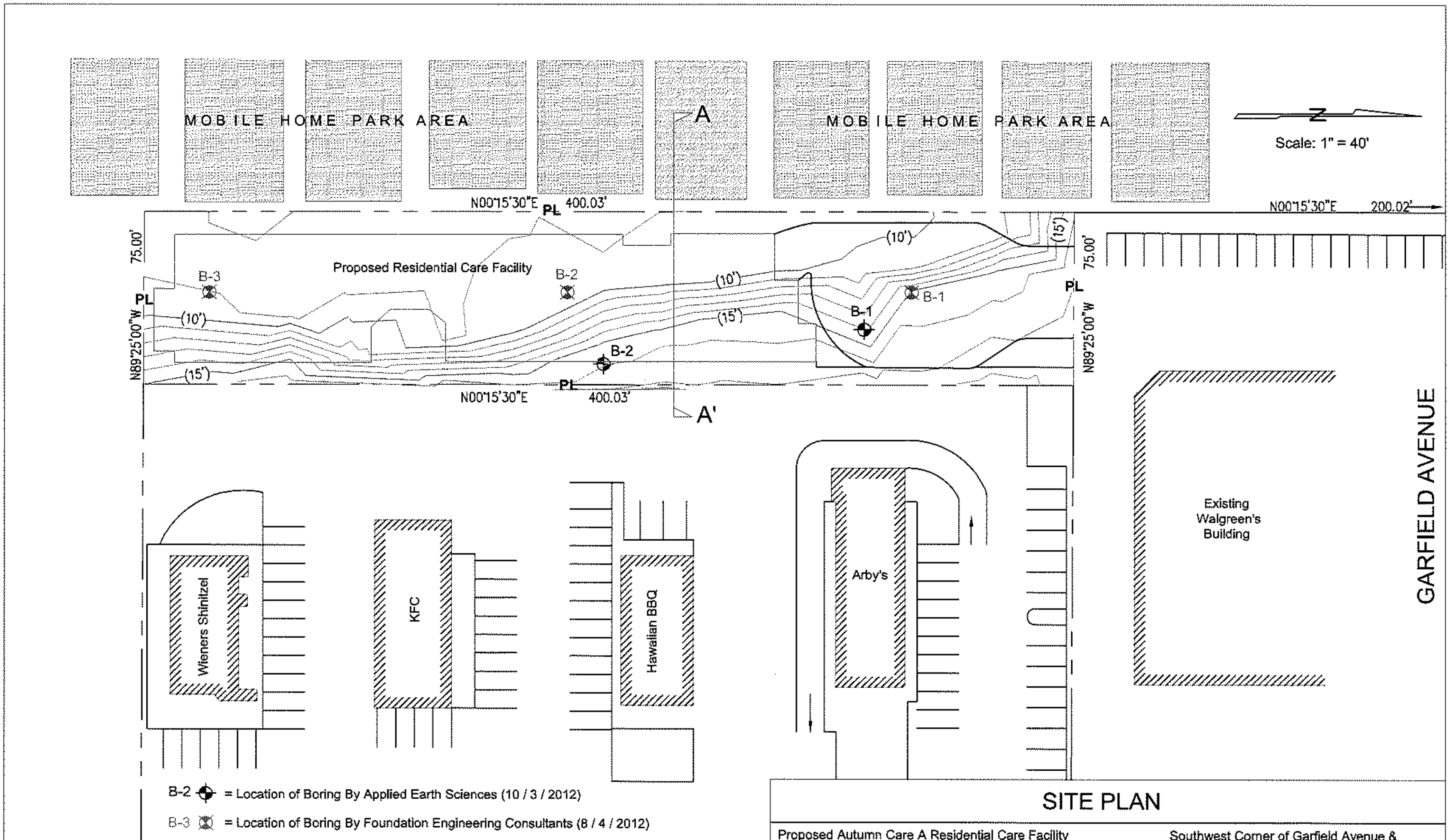
* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

---

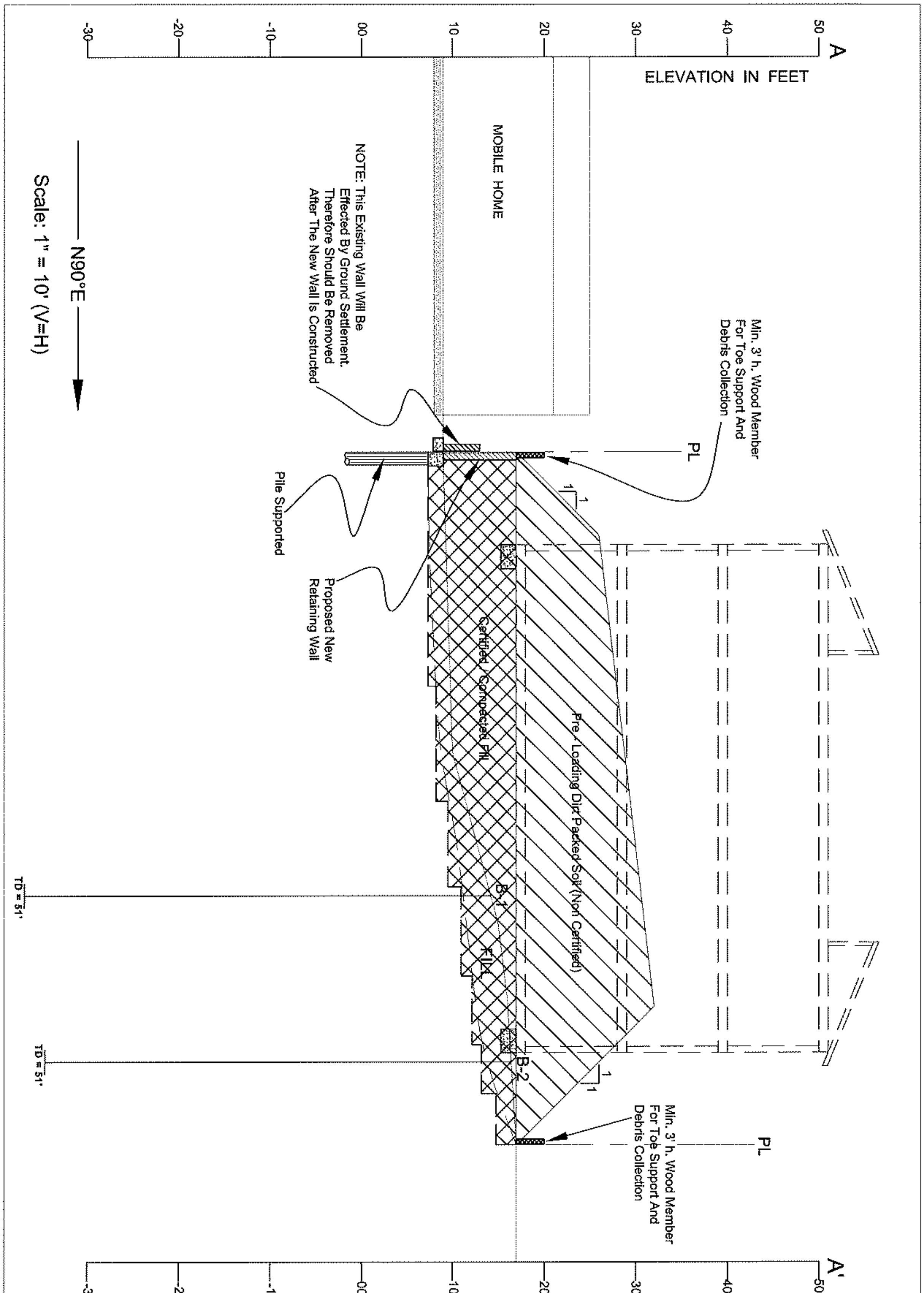
1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils



SITE PLAN

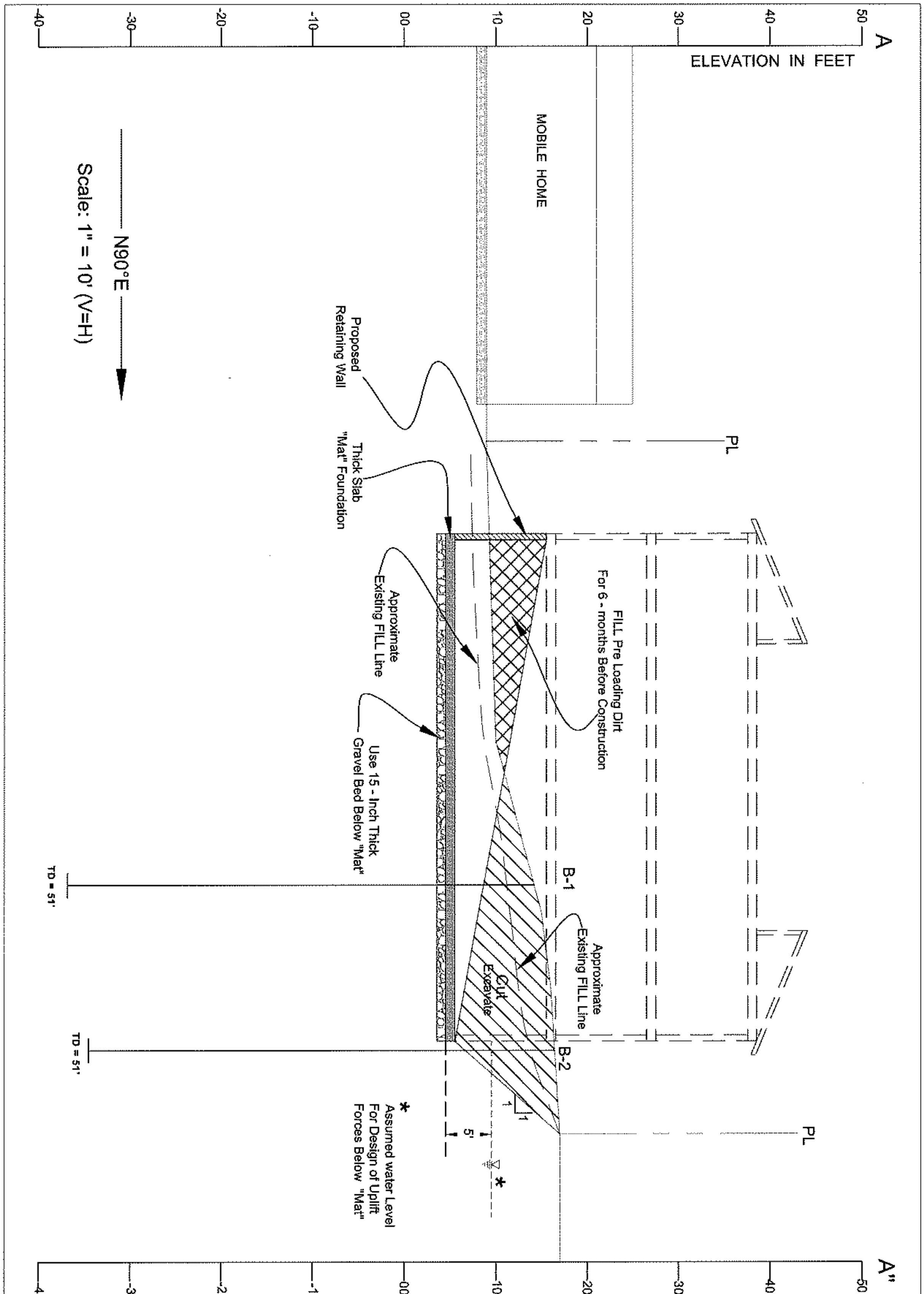
Proposed Autumn Care A Residential Care Facility		Southwest Corner of Garfield Avenue & Brookhurst Street, Huntington Beach, California	
FOR. Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No. 12-388-02	
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		DRAWING No. 1	



CROSS SECTION A-A' (OPTION 1)

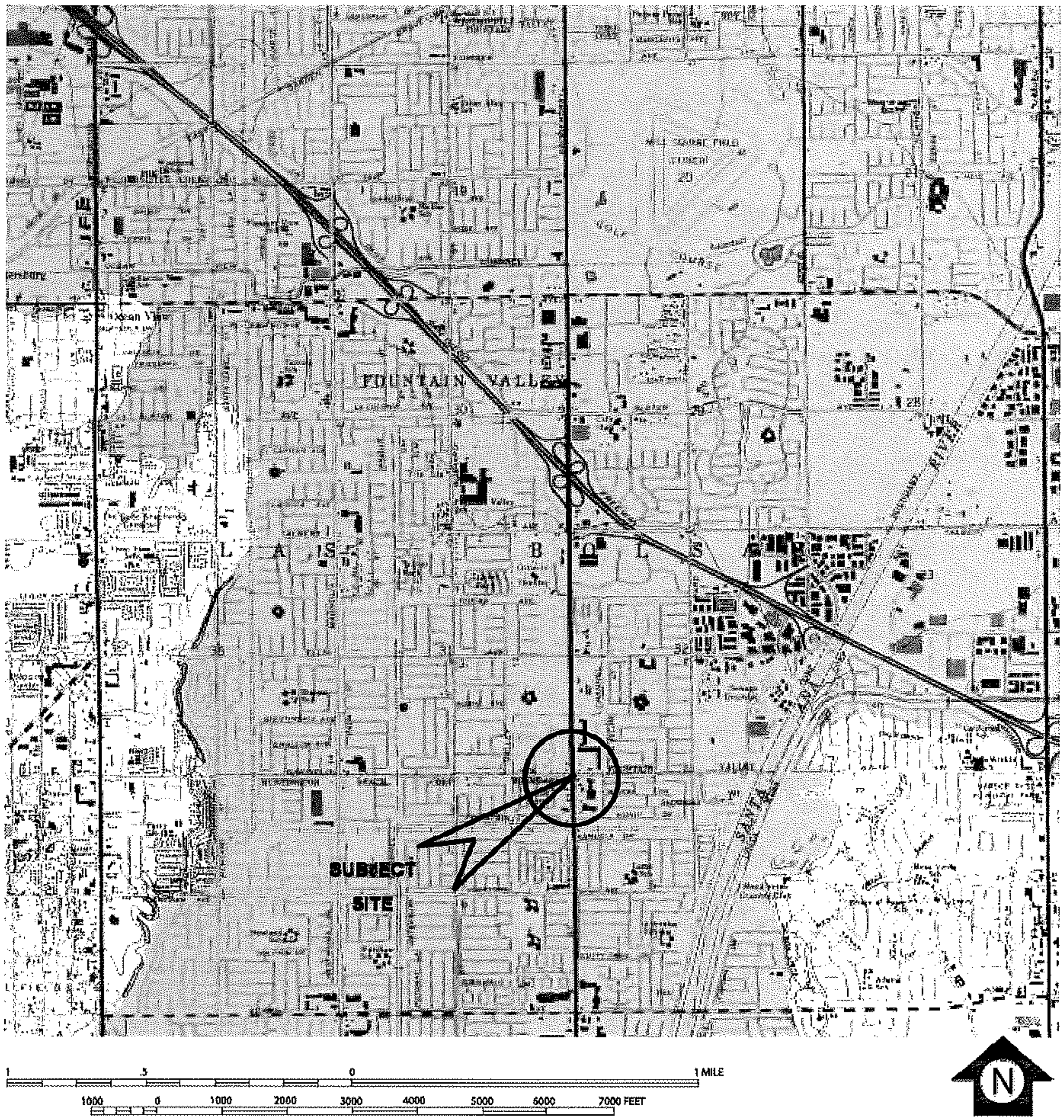
Proposed Autumn Care A Residential Care Facility	Southwest Corner of Garfield Avenue & Brookhurst Street, Huntington Beach, California

FOR. Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No. 12-388-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		DRAWING No. 2



CROSS SECTION A-A" (OPTION 2)

Proposed Autumn Care A Residential Care Facility		Southwest Corner of Garfield Avenue & Brookhurst Street, Huntington Beach, California	
FOR. Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No. 12-388-02	
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		DRAWING No. 3	



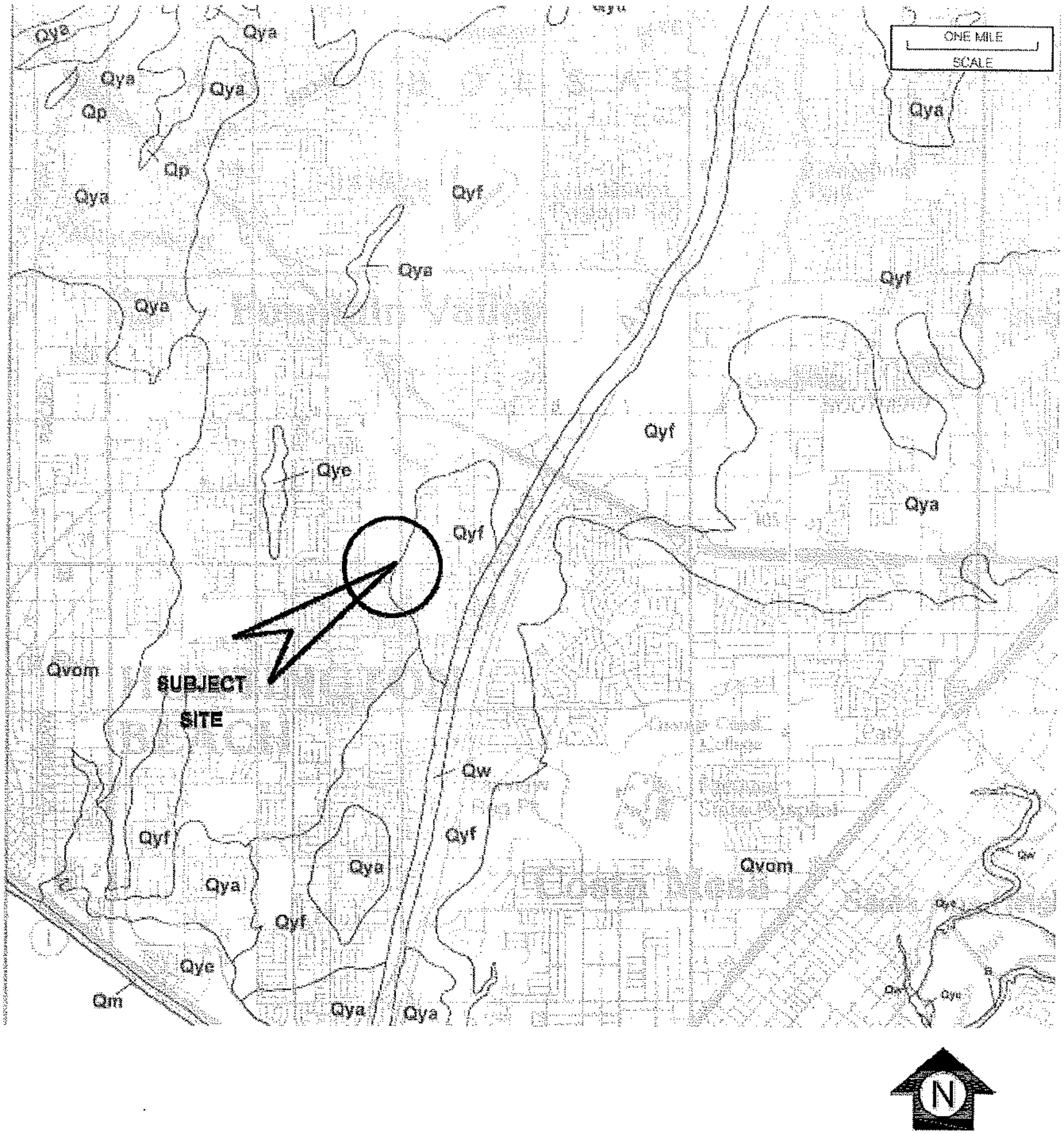
Reference: Portion of Seismic Hazard Zone Map of the Newport Quadrangle

SEISMIC HAZARD ZONES MAP

Proposed Residential Care Facility	Southwest Corner of Garfield Avenue and Brookhurst Street, Huntington Beach, California

FOR	DATE	PROJECT No.
Autumn Care Development Projects	10 / 30 / 2012	12-388-02

	FIGURE No.
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS	1

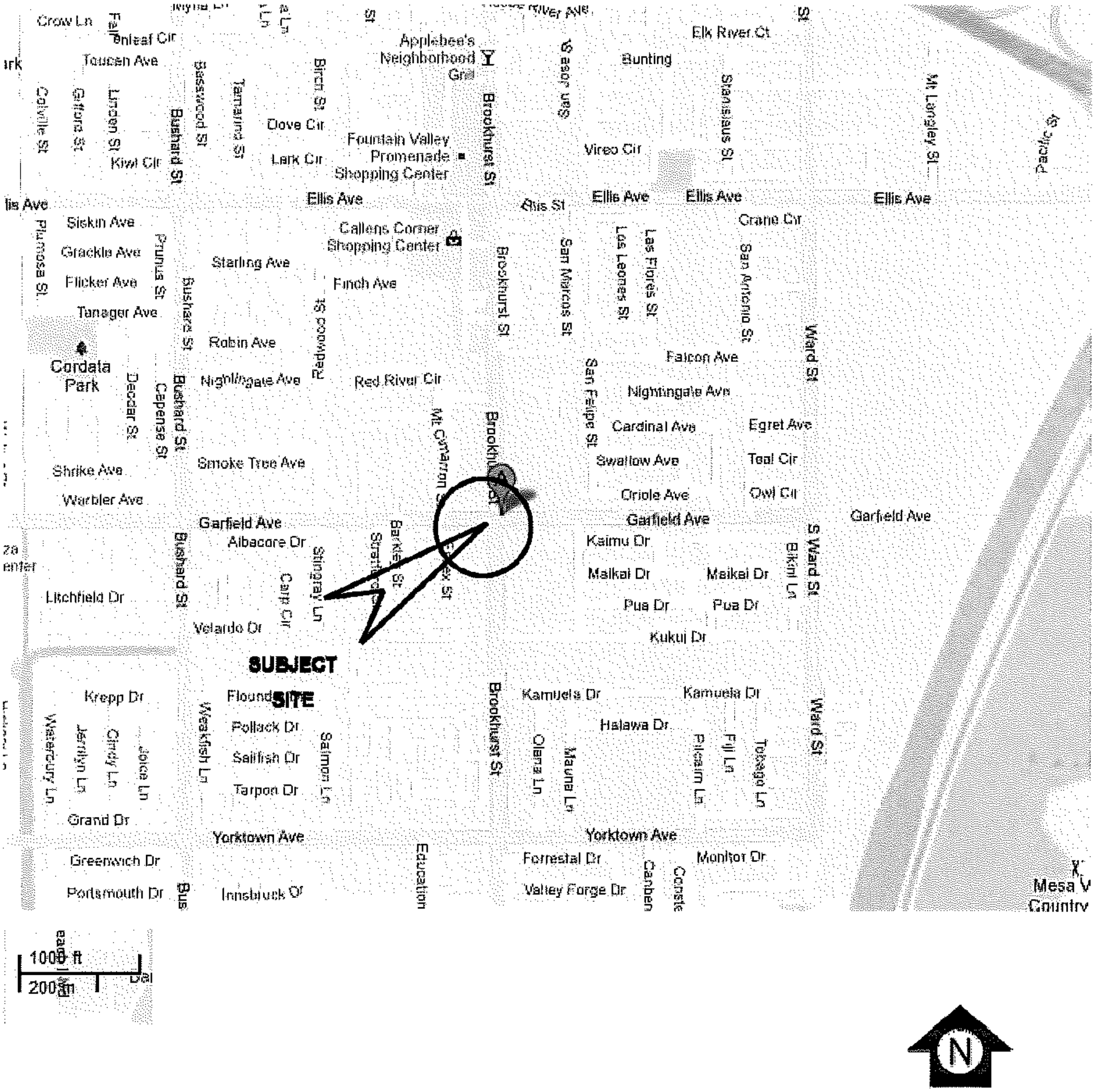


Reference: Newport 7.5 Minute Quadrangle

REGIONAL GEOLOGIC MAP

Proposed Residential Care Facility	Southwest Corner of Garfield Avenue and Brookhurst Street, Huntington Beach, California

FOR Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No. 12-388-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 2

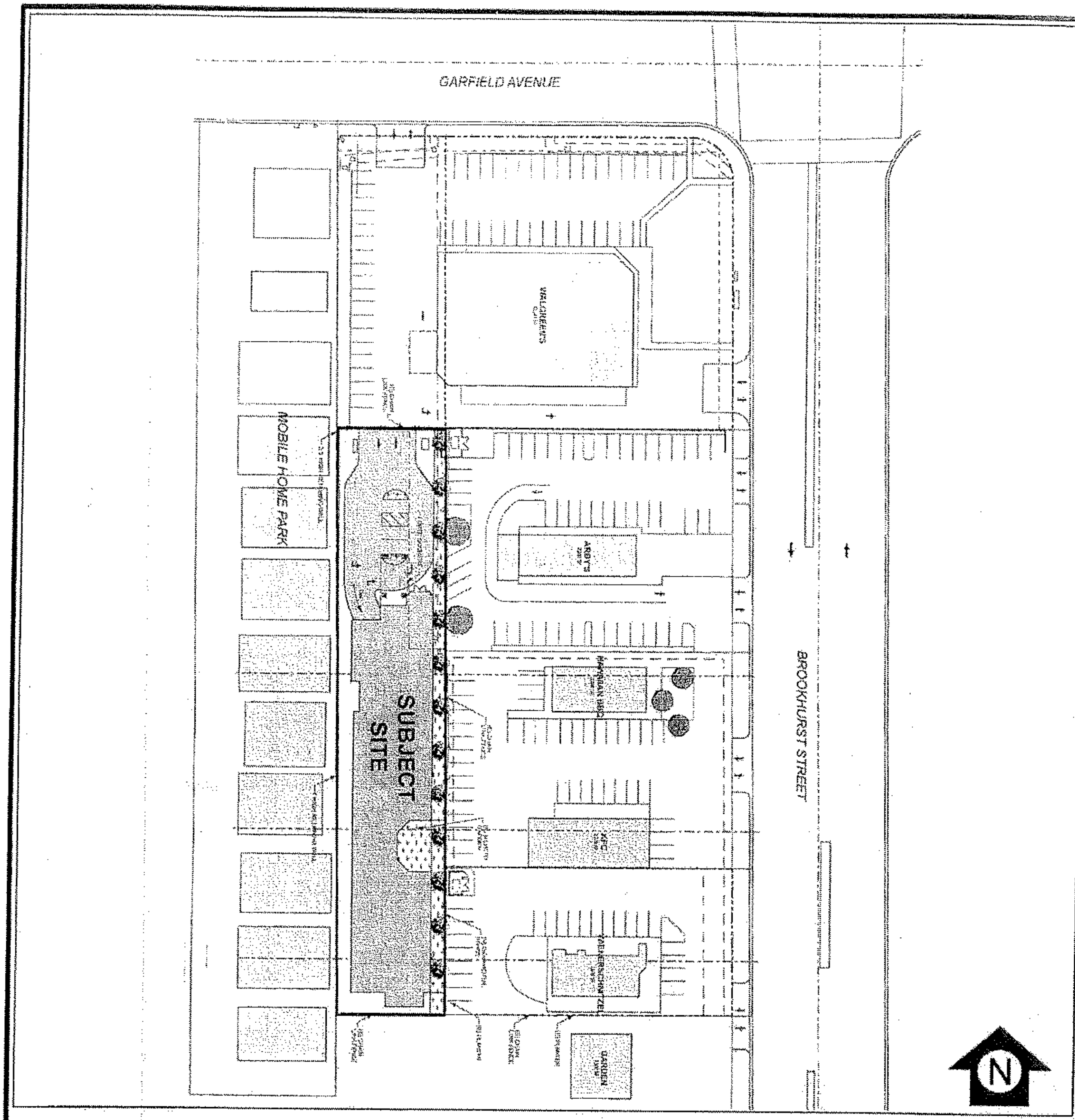


Reference: Portion of Google Maps

SITE VICINITY MAP

Proposed Residential Care Facility	Southwest Corner of Garfield Avenue and Brookhurst Street, Huntington Beach, California

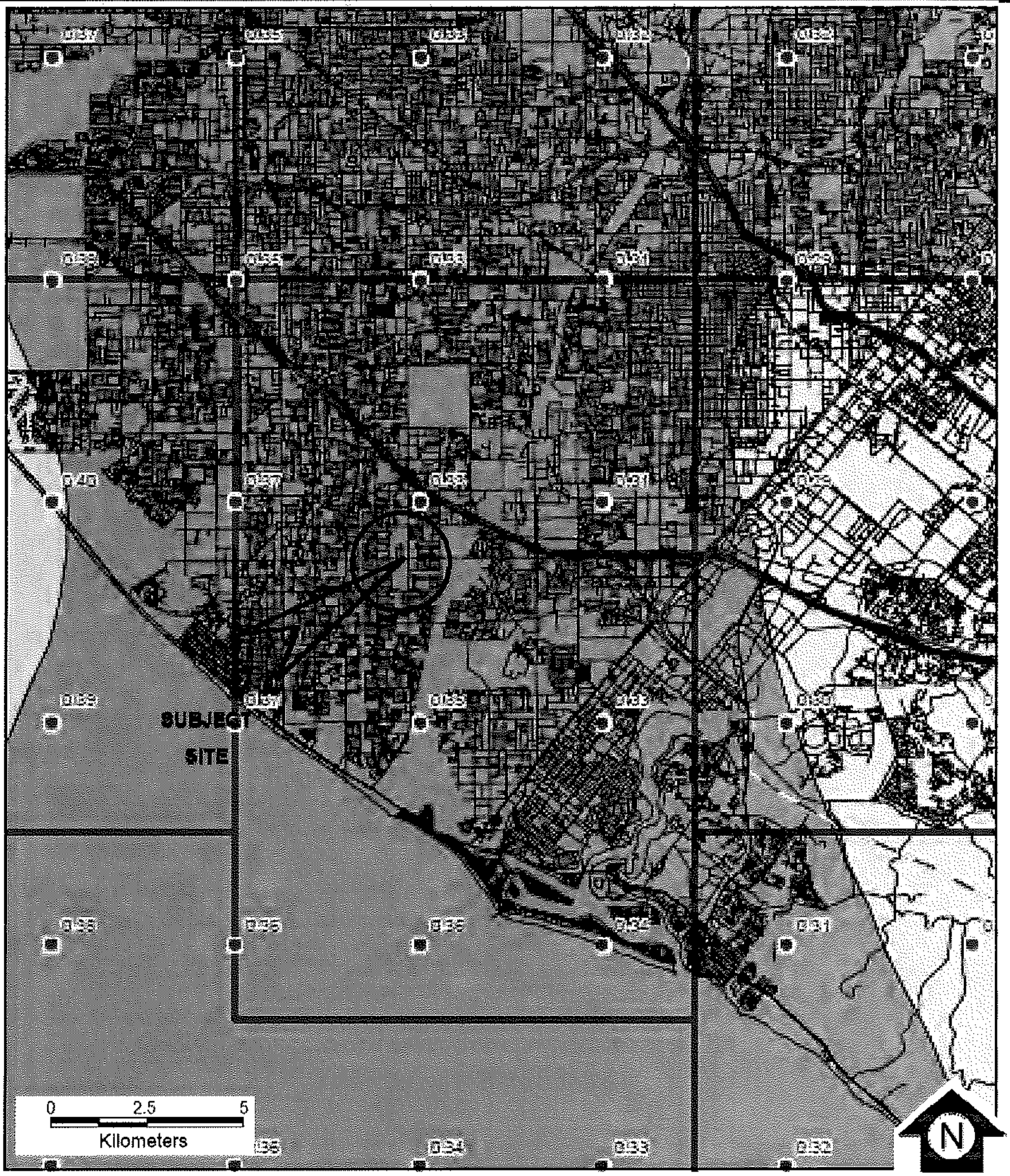
FOR Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No. 12-388-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 3



Scale: 1" = 100'

SITE LOCATION

Proposed Residential Care Facility		Southwest Corner of Garfield Avenue & Brookhurst Street, Huntington Beach, California	
FOR. Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No.	12-388-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No.	4



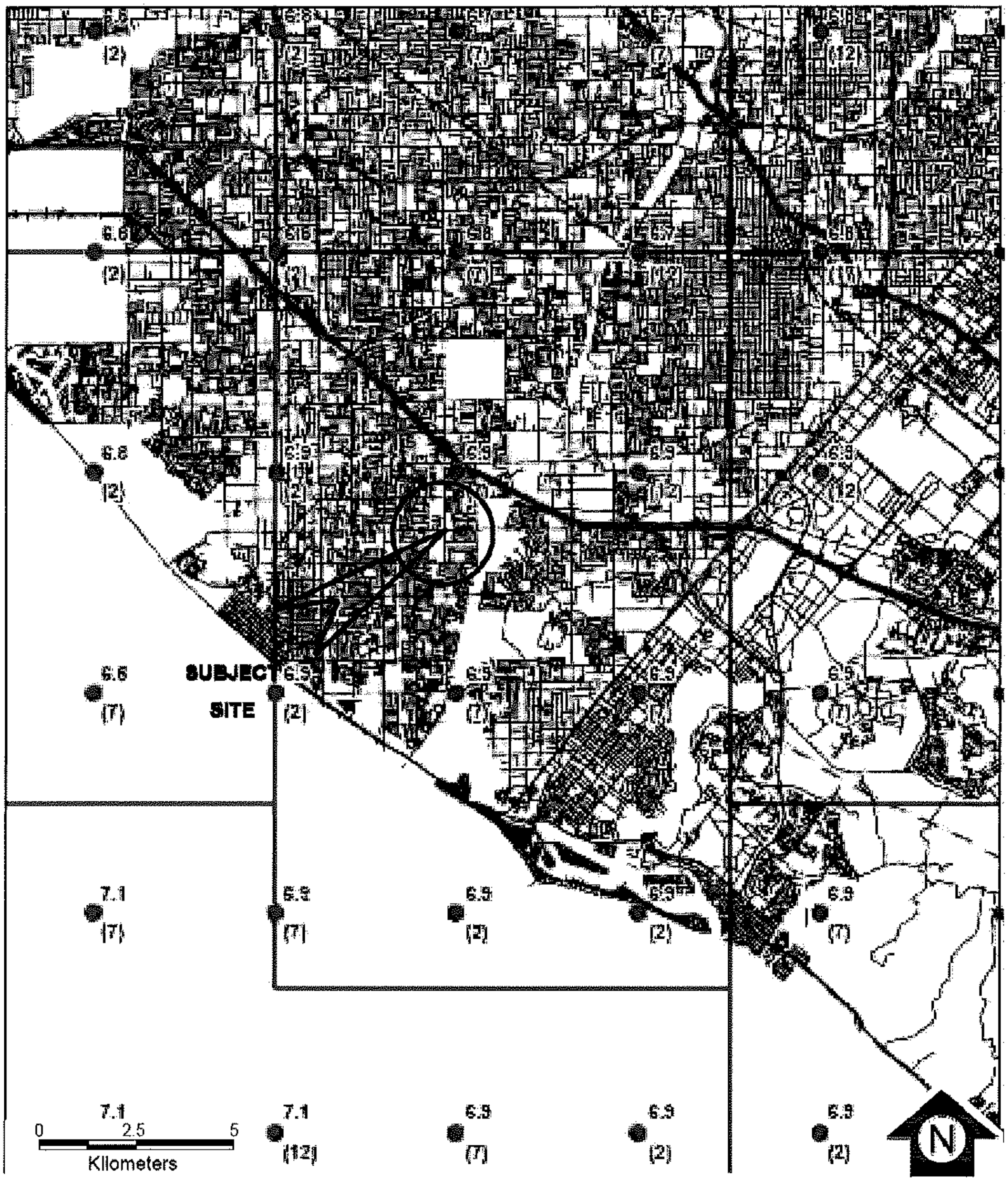
Reference: Newport 7.5 Minute Quadrangle

SEISMIC HAZARD MAP (Alluvium Conditions)

Proposed Residential Care Facility	Southwest Corner of Garfield Avenue and Brookhurst Street, Huntington Beach, California

FOR Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No 12-388-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 5

PREDOMINANT EARTHQUAKE Magnitude (Mw)

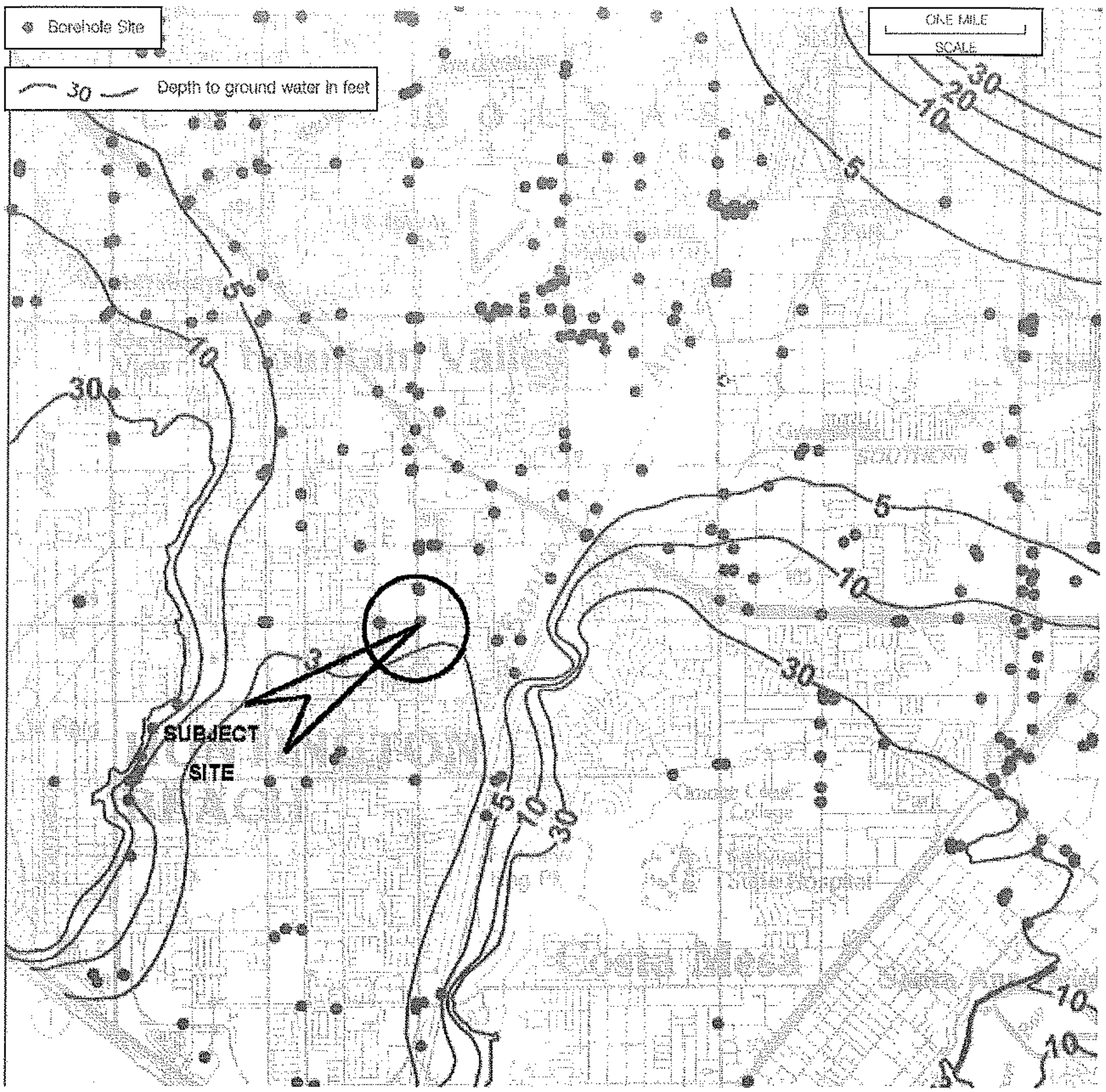


Reference: Newport 7.5 Minute Quadrangle

SEISMIC HAZARD MAP (Predominant Earthquake)

Proposed Residential Care Facility	Southwest Corner of Garfield Avenue and Brookhurst Street, Huntington Beach, California

FOR Autumn Care Development Projects	DATE 10 / 30 / 2012	PROJECT No. 12-388-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 6



Reference: Newport 7.5 Minute Quadrangle

HISTORICALLY HIGHEST GROUNDWATER (Contour Map)

Proposed Residential Care Facility	Southwest Corner of Garfield Avenue and Brookhurst Street, Huntington Beach, California

FOR Autumn Care Development Project	DATE 10 / 30 / 2012	PROJECT No. 12-388-02
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 7

APPENDIX I
METHOD OF FIELD EXPLORATION

In order to define subsurface conditions two borings were drilled at the site. The approximate locations of the borings with respect to the existing building are shown on the enclosed Site Plan. The borings were extended to a maximum depth of about 51.5 feet below the existing grades. The borings were drilled with a hollow stem drilling machine. The approximate locations of the drilled borings are shown on the enclosed Site Plan.

Continuous logs of the subsurface conditions, as encountered in the test borings, were recorded during the field work and are presented on Figure Nos. I-1 and I-2 within this Appendix. These figures also show the number and approximate depths of each of the recovered soil samples.

With hollow stem drilling, relatively undisturbed samples of the subsoil were obtained by driving a steel sampler with successive drops of a 140-pound sampling hammer free-falling a vertical distance of about 30 inches. The number of blows required for one foot of sampler penetration was recorded at the time of drilling and are shown on the log of exploratory borings. The relatively undisturbed soil samples were retained in brass liner rings 2.5 inches in diameter and 1.0 inch in height.

Field investigation for this project were performed on October 3, 2012. The materials excavated from the test borings were placed back and compacted upon completion of the field work. Such materials may settle. The owner should periodically inspect these areas and notify this office if the settlements create a hazard to person or property.

BORING No. 1

DATE EXCAVATED: 10/03/12 **LOGGED BY: RAFFI**

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (%DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
					SAND (SM)		Fill: Moderately Compact, slightly moist, medium brown, silty, few gravel.
	128	7	39	25	(SM)		Grades to compact, slightly moist, brown, silty, gravelly.
5							
	104	10	55	16 (SPT)	CLAY (CL)		Stiff, moist, brown, sandy, silty clay.
	87	35	94	9	(CL)		Grades to firm, very moist to wet, gray, silty, trace of sand.
10							
	86	39	92	9 (SPT)	(CL)		Grades to slightly sandy, light gray with slightly organic black pocket.
	74	42	95	5	(CL)		Grades to trace of sand.
15							
	83	37	91	8 (SPT)	(CL)		Grades to slightly sandy.
20							
	77	49	94	7 (SPT)	(CL)		Grades to blue gray, trace of sand.
25							
	75	52	94	7 (SPT)	(CL)		Grades to black.
30							
	71	58	97	8 (SPT)	(CL)		Grades to black- gray.

LOG OF BORING

JOB NAME: SW Corner Of Garfield Avenue & Brookhurst Street JOB No. 12-388-02

APPLIED EARTH SCIENCES
GEOTECHNICAL ENGINEERING CONSULTANTS

FIGURE NO : I-1.1

BORING No. 1 (continued)

DATE EXCAVATED: 10/03/12 **LOGGED BY: Raffi**

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (%DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
							Continued from previous page
35	78	47	97	9 (SPT)	(CL)		Grades to trace of sand.
40	85	34	78	11 (SPT)	(CL))		Grades to sandy.
45	97	30	77	26 (SPT)	(CL)		Grades to stiff.
50	102	26	30	24 (SPT)	SAND (SM)		Dense, wet, gray, silty, clayey, fine grained sand.
							End of Boring @ 51 feet Water @ 6 feet

LOG OF BORING

JOB NAME: SW Corner Of Garfield Avenue & Brookhurst Street JOB No. 12-388-02

APPLIED EARTH SCIENCES
GEOTECHNICAL ENGINEERING CONSULTANTS FIGURE NO: I-1.2

BORING No. 2

DATE EXCAVATED: 10/03/12 **LOGGED BY: RAFFI**

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (%DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
					SAND (SM)		**Fill:** Moderately Compact, slightly moist, medium brown, silty, few gravel.
	119	6	43	23	(SM)		Grades to compact, slightly moist, brown, silty, few gravel.
5	106	9	43	17 SPT			Grades to moist, brown, silty, clayey, gravelly.
					CLAY (CL)		Firm, very moist to wet, gray, silty, slightly sandyclay
10	83	41	98	10 SPT	(CL)		Grades to firm, wet, gray, trace of sand.
15	84	42	90	10 SPT	(CL)		Grades to slightly sandy, gray-black.
20	71	59	97	9 (SPT)	(CL)		Grades to trace of sand.
25	73	56	94	16 (SPT)	(CL)		Grades to black.
30	74	53	97	10 (SPT)	(CL)		Grades to gray, trace of sand.

LOG OF BORING

JOB NAME: SW Corner Of Garfield Avenue & Brookhurst Street JOB No. 12-388-02

APPLIED EARTH SCIENCES
GEOTECHNICAL ENGINEERING CONSULTANTS

FIGURE NO : I-2.1

BORING No. 2 (continued)

DATE EXCAVATED: 10/03/12 **LOGGED BY: Raffi**

DEPTH IN FEET	DRY DENSITY (PCF)	FIELD MOISTURE (%DRY WEIGHT)	% PASSING #200	BLOWS PER FOOT	MATERIAL TYPE	MATERIAL SYMBOL	MATERIAL DESCRIPTION
							Continued from previous page
35	72	54	92	17 (SPT)	(CL)		Grades to slightly sandy.
40	87	41	84	15 (SPT)	(CL)		Grades to firm, gray, sandy.
45	90	33	79	18 (SPT)	(CL)		Grades to stiff.
50	70	40	91	12 (SPT)	(CL)		Grades to slightly sandy.
							End of Boring @ 51 feet Water @ 6 feet

LOG OF BORING

JOB NAME: SW Corner Of Garfield Avenue & Brookhurst Street — JOB No. 12-388-02

APPLIED EARTH SCIENCES
GEOTECHNICAL ENGINEERING CONSULTANTS

FIGURE NO : I-2.2

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAME
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Organic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Organic clay of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Organic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Organic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO 200 — NO. 40 — NO. 10 — NO. 4 — ¾ in. — 3 in. — (12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

JOB NAME : Proposed Residential Care Facility
Southwest Corner of Garfield Avenue &
Brookhurst Street, Huntington Beach, California

JOB No. 12-388-02

APPLIED EARTH SCIENCES
GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS

FIGURE No. I-3

APPENDIX II

LABORATORY TESTING PROCEDURES

Moisture Density

The moisture-density information provides a summary of soil consistency for each stratum and can also provide a correlation between soils found on this site and other nearby sites. The tests were performed using ASTM D 2216-04 Laboratory Determination of water content Test Method. The dry unit weight and field moisture content were determined for each undisturbed sample, and the results are shown on log of exploratory borings.

Shear Tests

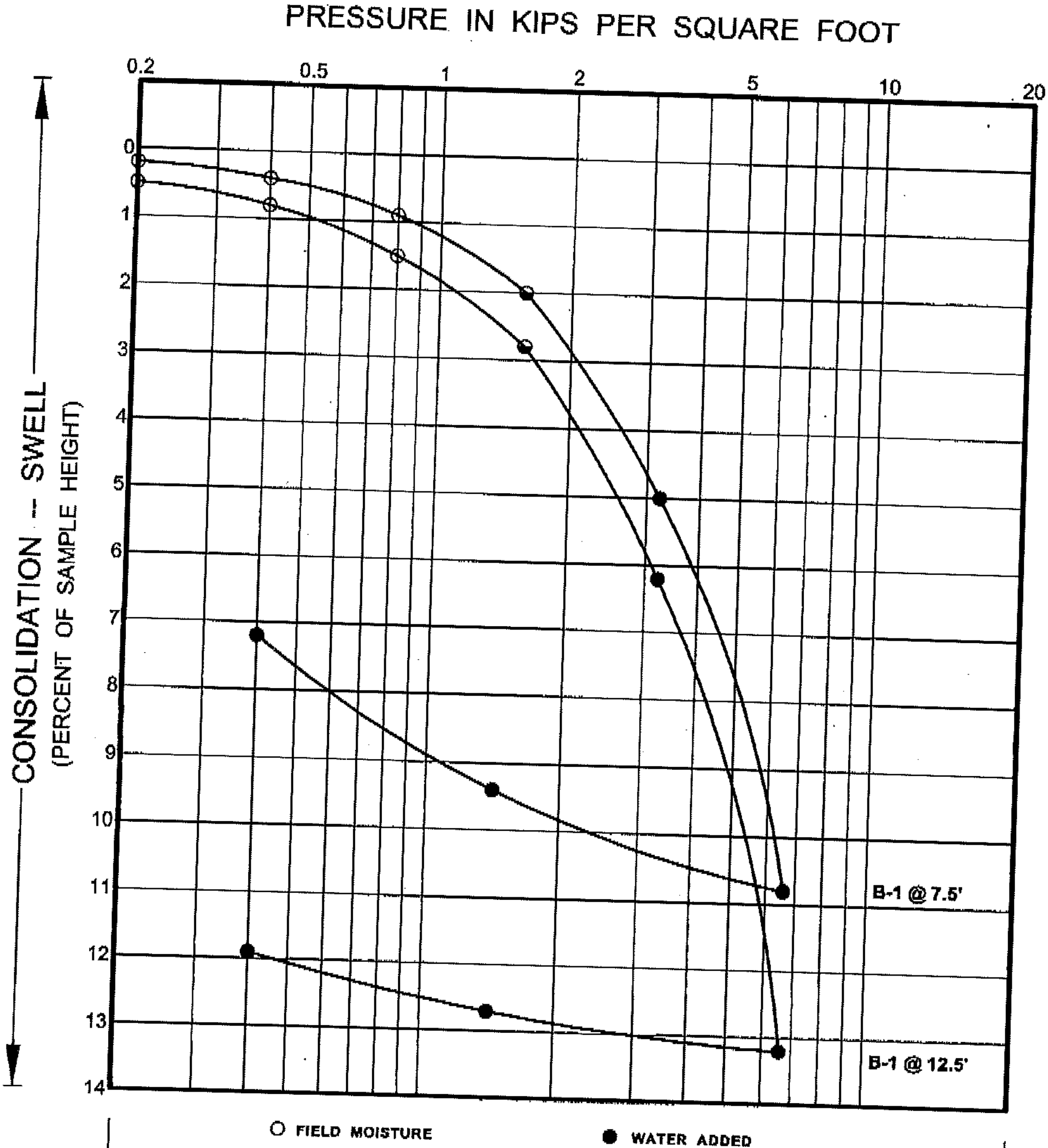
Shear tests were made with a direct shear machine at a constant rate of strain. The machine is designed to test the materials without completely removing the samples from the brass rings. The rate of shear was determined through determination of the rate of consolidation of the foundation bearing materials. Considering that such soils are fine grained in nature with a t90 value of less than 27 seconds, the rate of shearing was selected as 0.005 inches per minute.

A range of normal stresses was applied vertically, and the shear strength was progressively determined at each load in order to determine the internal angle of friction and the cohesion. The tests were performed using ASTM D 3080-04 Laboratory Direct Shear Test Method. The Ultimate shear strength results of direct shear tests are presented on Figure No. II-1 within this Appendix.

Consolidation

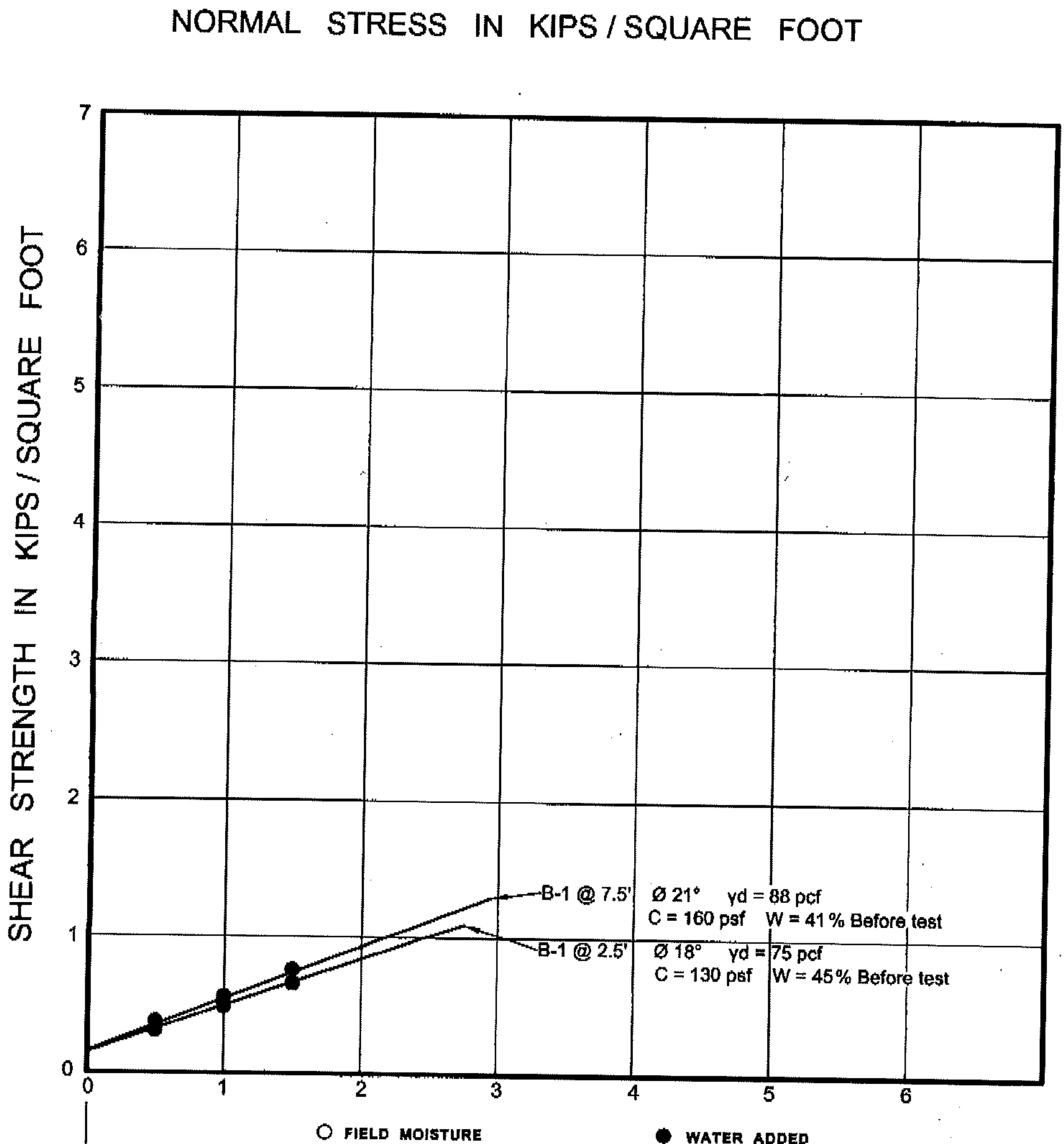
The apparatus used for the consolidation tests is designed to receive the undisturbed brass ring of soil as it comes from the field. Loads were applied to the test specimen in several increments, and the resulting deformations were recorded at time intervals. Porous stones were placed in contact with the top and bottom of the specimen to permit the ready addition or release of water. ASTM D 2435-04 Laboratory Consolidation Test Method.

Undisturbed specimens were tested at the field and added water conditions. The test results are shown on Figure No. II-2 within this Appendix.



SWELL - CONSOLIDATION TESTS

JOB NAME	Proposed Residential Care Facility Southwest Corner of Garfield Avenue & Brookhurst Street, Huntington Beach, California	JOB No.	12-388-02
	APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS	FIGURE No.	II - 2



DIRECT SHEAR TESTS

JOB NAME: Proposed Residential Care Facility
Southwest Corner of Garfield Avenue &
Brookhurst Street, Huntington Beach, California

JOB No. — —

APPLIED EARTH SCIENCES
GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS