GEOTECHNICAL INVESTIGATION REPORT
PROPOSED NORWALK SHOPPING CENTER
12623 NORWALK BOULEVARD

Norwalk, California

Prepared for:
MR. DAN ASHOURI

Prepared by:
GEOBODEN INC.
Irvine, CA 92620

January 16, 2015

Project No. Norwalk-1-01

GEOBODEN INC.

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J.N. Norwalk-1-01

GEOBODEN, INC.
Geotechnical Consultants

January 16, 2015 Project No. Norwalk-1-01

Mr. Dan Ashouri
14431 Ventura Boulevard, Suite 601
Los Angeles, California 91423

Subject: Geotechnical Investigation Report
Proposed Norwalk Shopping Center
12623 Norwalk Boulevard
Norwalk, California

GeoBoden, Inc. is pleased to provide you with this report on our geotechnical investigation for the proposed Norwalk Shopping Center to be constructed on the subject site. Based upon the findings of our investigations, we have concluded that the proposed development of the site is feasible from the geotechnical perspective.

Please do not hesitate to contact the undersigned if you have any questions or if we may be of any additional assistance. We look forward to assisting you during the construction of the proposed structure.

Respectfully submitted,
GEOBODEN, INC.

Cyrus Radvar, G.E.#2742
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
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PROPOSED NORWALK SHOPPING CENTER
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NORWALK, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed by GeoBoden, Inc. (GeoBoden) for the proposed Norwalk Shopping Center to be located at 12623 Norwalk Boulevard in the city of Norwalk, California. The general location of the project is shown on site "Vicinity Map", Figure 1.

The purposes of this investigation were to determine the geotechnical properties of subsurface soil conditions, to evaluate their in-place characteristics, evaluate site seismicity, and to provide geotechnical recommendations with respect to site grading and for design and construction of buildings foundations and other site improvements.

The scope of the authorized investigation included performing a site reconnaissance, conducting field exploration and laboratory testing programs, performing engineering analyses, and preparing this Geotechnical Investigation Report. The scope of work was outlined in our "Proposal for Geotechnical Investigation". Evaluation of environmental issues or the potential presence of hazardous materials was not within the scope of services provided.

This report has been prepared for Pacific Land Management and their other project team members, to be used solely in the development of facilities described herein. This report may not contain sufficient information for other uses or the purposes of other parties.

2.0 PROJECT LOCATION AND DESCRIPTION

The site is located at 12623 Norwalk Boulevard, Norwalk, California. The subject site is presently occupied by an existing restaurant building and surface parking. The existing building and surface parking will be demolished to accommodate the new construction.

3.0 PROPOSED GRADING AND CONSTRUCTION

Based on our review of the Site Plan (Figure 2), the proposed development will include construction of Buildings A and B. Building A will accommodate construction of retail and Restaurant. Building B will consist of an IHOP. Other associated improvements include construction of concrete flatwork such as patio-type slabs, walk- and driveways and surface parking.

Although construction details are not available at the present time, it is expected that the proposed structures will be one story in height, and of wood frame construction with slab-on-grade floors. We understand the no basements are planned for any of the retail buildings. Proposed drainage facilities are expected to consist of sloping graded surfaces, area drains and sloped concrete flatwork.

4.0 GEOTECHNICAL INVESTIGATION

Our geotechnical investigation included a field exploration program and a laboratory testing programs. These programs were performed in accordance with our proposal for "Geotechnical Investigation". The field exploration and laboratory testing programs are briefly described below. A more detailed description of the laboratory testing programs is provided in Appendix B.

4.1 FIELD EXPLORATION PROGRAM

The current field exploration program was completed under the technical supervision of the undersigned geotechnical engineer. A total of 4 exploratory borings were drilled using a truck-mounted drilling rig equipped with 6-inch diameter hollow stem augers. The borings were advanced to depths ranging from 21.5 to 51.5 feet (below ground surface). The approximate locations of exploratory borings are shown on Figure 2.

Logs of subsurface conditions encountered in the borings were prepared in the field by a representative of our firm. Soil samples consisting of relatively undisturbed brass ring samples and Standard Penetration Tests (SPT) samples were collected at approximately 5-foot depth intervals and were returned to the laboratory for testing. A Bulk sample was collected at depths of 1 to 5 feet below ground surface (bgs). The SPTs were performed in accordance with

ASTM D 1586. Final boring logs were prepared from the field logs and are presented in Appendix A.

4.2 LABORATORY TESTING

Selected samples collected during drilling activities were tested in the laboratory to assist in evaluating controlling engineering properties of subsurface materials at the site. Physical tests performed included moisture and density determination, No. 200 Wash sieve, Atterberg limits, consolidation, expansion, and corrosion. The results of laboratory testing are presented in Appendix B.

5.0 DISCUSSION OF FINDINGS

The following discussion of findings for the site is based on the results of the field exploration and laboratory testing programs.

5.1 SITE AND SUBSURFACE CONDITIONS

The site is underlain by undocumented artificial fill material overlying native silty sand, sandy sandy silt, and sandy clay. The artificial fill depths varied from approximately 3 to 4 feet below existing grade within the areas of our borings. The artificial fill soils were comprised primarily of silty sand and sand with silt similar in composition to the near surface native soils.

Based on blow counts recorded during sampling, the sandy soils encountered within borings were found to be medium dense to dense. Cohesive soils were found to be stiff to hard. For more detailed descriptions of the subsurface materials refer to the boring logs included in Appendix A of this report.

5.2 GROUNDWATER CONDITIONS

Groundwater was not encountered within any of our exploratory borings to the maximum depths explored (51.5 feet bgs). Historically highest groundwater contours prepared by California Division of Mines and Geology (CDMG, Open File Report No. 98-28), renamed California Geologic Survey (CGS) indicate that groundwater has been as shallow as approximately 10 feet bgs at the subject site.

Fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during the rainy season. Irrigation of landscaped areas can also lead to an increase in soil moisture content and fluctuations of intermittent shallow perched groundwater levels.

5.3 PERCOLATION (INFILTRATION) TEST RESULTS

Borings B-1 and B-4 was backfilled to 5 feet and percolation testing was performed in general accordance with the "Technical Guidance Document" procedures of the Infiltration BMPs. The purpose of this testing was to assess the general percolation rates of the onsite soils for the design of an onsite infiltration system.

The continuous pre-soak (falling-head) test procedure was utilized for testing. Water was allowed to presoak in each test hole prior to obtaining test readings. Following the presoak period, the drop in water level in each hole was monitored every 10 to 30 minutes to determine the appropriate method for testing. Test holes were refilled following each reading or when the water depth was below 6 inches. Test times ranged from 120 to 150 minutes. The drop in water level was recorded to the nearest 1/10th inch to produce conservative water level readings.

Tests results are summarized below:

Test Hole No.	**Rate (IN/HOUR)**
1	1.4
2	2.1

It should be noted that the infiltration rates determined are ultimate rates based upon field test results. An appropriate safety factor should be applied to account for subsoil inconsistencies and potential silting of the percolating soils. The safety factor should be determined with consideration to other factors in the storm water retention system design (particularly storm water volume estimates) and the safety factors associated with those design components.

5.4 SOIL ENGINEERING PROPERTIES

Physical tests were performed on the relatively undisturbed samples to characterize the engineering properties of the native soils. Moisture content and dry unit weight results are shown on the boring logs in Appendix A.

5.5 CONSOLIDATION CHARACTERISTICS

Consolidation test was performed on sample of the existing native overburden soils recovered from the boring. Results of the consolidation tests indicate that the overburden material will have low compressibility under the anticipated loads. These characteristics are compatible with the allowable bearing capacity values and corresponding settlement estimates presented in Section 9.7 (Foundations).

5.6 EXPANSION POTENTIAL

Laboratory testing of representative samples of onsite soils indicate that these materials exhibit LOW expansion potentials.

6.0 CALIFORINA BUILDING CODE (CBC) SEISMIC DESIGN PARAMETERS

To accommodate effects of ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2013 edition of the California Building Code (CBC). Table below, 2013 CBC Seismic Parameters, lists (next) seismic design parameters based on the 2013 CBC methodology, which is based on ASCE/SEI 7-10:

2013 CBC Seismic Design Parameters	Value
Site Latitude (decimal degrees)	33.9163
Site Longitude (decimal degrees)	-118.0729
Site Class Definition (ASCE 7 Table 20.3-1)	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s (Figure 1613.3.1(1))	1.929
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.687
Short Period Site Coefficient at 0.2s Period, F_a (Table 1613.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, F_v (Table 1613.3.3(2))	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS} (Eq. 16-37)	1.929
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16-38)	1.031
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16-39)	1.286
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16-40)	0.687

7.0 LIQUEFACTION POTENTIAL

For liquefaction to occur, all of three key ingredients are required: liquefaction-susceptible soils, groundwater within a depth of 50 feet or less, and strong earthquake shaking. Soils susceptible to liquefaction are generally saturated loose to medium dense sands and non-plastic silt deposits below the water table.

According to the "Map of Seismic Hazard Zones", prepared by the California Geologic Survey, the site is located within a liquefaction hazard zone; therefore a site liquefaction evaluation consistent with the guidelines contained in CDMG Special Publication 117a has been performed as part of the current investigation. A historic high water level of 10 feet was adopted for liquefaction analysis.

To evaluate the site-specific liquefaction potential, we computed the geometric mean peak ground acceleration (PGA) for the ground motion with a 2% probability of being exceeded in 50 years (Peak Ground Acceleration = 0.738g) from USGS website for MCE_R. We used USGS 2008 Interactive Deaggregations website tool. The results of the probabilistic seismic hazard analysis indicate the modal seismic event is Moment Magnitude (Mw) 6.58. The SPT consists of driving a standard sampler, as described in the ASTM 1586 Standard Method, using a 140 pound hammer falling 30 inches. An Automatic Trip Hammer was used to drive samplers 18 inches into the soil. For the measured 79 percent hammer efficiency of an automatic hammer, the energy ratio 1.32 was used in our liquefaction evaluation. We have used the borehole diameter correction factor (CB) 1.0 in our liquefaction evaluation.

For liquefaction of sandy, silty and clayey soils, we used methods proposed by Tokimatsu and Seed (1987). Based on our analysis and under the current site conditions, we estimate that the maximum total liquefaction-induced ground settlements at the site would of about 0.64 inches during the postulated earthquake. Differential settlements of approximately 0.43 inch or less could occur over a span of 40 feet. The computer outputs are attached for reference. It is our opinion that potential for liquefaction at the site is low and will not adversely impact the proposed construction. We recommend that the proposed medical office development be supported on shallow foundation system.

8.0 DESIGN RECOMMENDATIONS

Based upon the results of our investigation, the proposed structures are considered geotechnically feasible provided the recommendations presented herein are incorporated into the design and construction. If changes in the design of the structure are made or variations or changed conditions are encountered during construction, GeoBoden should be contacted to evaluate their effects on these recommendations. The following geotechnical engineering recommendations for the proposed buildings are based on observations from the field investigation program and the physical test results.

8.1 EARTHWORK

All earthworks, including excavation, backfill and preparation of subgrade, should be performed in accordance with the geotechnical recommendations presented in this report and applicable portions of the grading code of local regulatory agencies. All earthwork should be performed under the observation and testing of a qualified geotechnical engineer.

8.2 SITE CLEARING

All active or inactive utilities within the construction limits should be identified for relocation, abandonment, or protection prior to grading. Any subsurface piping encountered should be abandoned in-place by being filled with sand/cement slurry. The adequacy of existing backfill around utilities to remain in place under new structures should be evaluated; loose or dumped trench backfill should be removed and replaced with properly compacted backfill.

In general, all fill soils within the proposed buildings footprints should be overexcavated and replaced with engineered fill. As a minimum, removals should extend to competent native soils. At least 3 feet of compacted fill should be provided underneath all spread footings and floor slabs. The compacted fill should extend laterally a minimum of 5 feet beyond the foundation footprints, where possible. All existing low-density, near-surface soils will require removal to competent material from areas to receive newly compacted fill. The basis for establishing a competent exposed surface on which to place fill should consist of competent materials exhibiting an in-place relative compaction of at least 85 percent. Prior to placing structural fill, exposed bottom surfaces in each removal area approved for fill should first be scarified to a depth of at least 6 inches, water or air dried as necessary to achieve near optimum moisture conditions, and then recompacted in place to a minimum relative compaction of 90 percent.

Based on the observations made in our borings and the results of pertinent laboratory tests, anticipated depths of removal of unsuitable soils will be about 5 feet. However, actual removal depths will have to be determined during grading on the basis of in-grading observations and testing performed by a representative of geotechnical consultants.

8.3 GEOTECHNICAL OBSERVATIONS

Exposed bottom surfaces in each removal area should be observed and approved by the project geotechnical consultant prior to placing fill. No fill should be placed without prior approval from the geotechnical consultant.

The project geotechnical consultant should be notified at the appropriate times to provide observation and testing services during clearing operations to verify compliance with the above recommendations. In addition, should any buried structures or unusual or adverse soil conditions be encountered during grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

8.4 FILL PLACEMENT AND COMPACTION REQUIREMENTS

Material for engineered fill should be select free of organic material, debris, and other deleterious substances, and should not contain fragments greater than 3 inches in maximum dimension. On-site excavated soils that meet these requirements may be used to backfill the excavated building pad area.

All fill should be placed in 6-inch-thick maximum lifts, watered or air dried as necessary to achieve near optimum moisture conditions, and then compacted in place to a maximum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557. A representative of the project consultant should be present on-site during grading operations to verify proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

8.5 IMPORTED SOILS

Imported soils are required to complete the planned finished grades, these soils should consist of clean materials devoid of rock exceeding a maximum dimension of 3 inches, as well as organics, trash and similar deleterious materials. Imported soils should also exhibit an expansion potential no greater than LOW, as classified in accordance with ASTM D4829. Prospective import soils should be observed, tested and approved by this firm prior to importing the soils to the site.

8.6 FOUNDATION DESIGN RECOMMENDATIONS

Following the site and foundation preparation recommended above, foundation for load bearing walls and interior columns may be designed as discussed below.

8.6.1 Bearing Capacity and Settlement

Load bearing walls and interior columns may be supported on continuous spread footings and isolated spread footings, respectively, and should bear entirely upon properly engineered fill. Continuous and isolated footings should have a minimum width of 14 inches and 24 inches, respectively. All footings should be embedded a minimum depth of 18 inches measured from the lowest adjacent finish grade. Continuous and isolated footings placed on such materials may be designed using an allowable (net) bearing capacity of 2,000 pounds per square foot (psf). Allowable increases of 250 psf for each additional 1 foot in width and 250 psf for each additional 6 inches in depth may be utilized, if desired. The maximum allowable bearing pressure should be 3,000 psf. The maximum bearing value applies to combined dead and sustained live loads. The allowable bearing pressure may be increased by one-third when considering transient live loads, including seismic and wind forces.

Based on the allowable bearing value recommended above, total settlement of the shallow footings are anticipated to be less than one inch, provided foundation preparations conform to the recommendations described in this report. Differential settlement is anticipated to be approximately half the total settlement for similarly loaded footings spaced up to approximately 40 feet apart.

Foundation settlement will also occur due to the potential hydro-collapse of the native onsite soils left in place beneath the fill materials. Based on our mitigation recommendations provided in this report, it is anticipated that settlement characteristics of the native soils are not going to have a profound impact on site development.

8.6.2 Lateral Load Resistance

Lateral load resistance for the spread footings will be developed by passive soil pressure against sides of footings below grade and by friction acting at the base of the concrete footings bearing on compacted fill. An allowable passive pressure of 250 psf per foot of depth may be used for design purposes. An allowable coefficient of friction 0.35 may be used for dead and sustained live load forces to compute the frictional resistance of the footings constructed directly on compacted fill. Safety factors of 2.0 and 1.5 have been incorporated in development of allowable passive and frictional resistance values, respectively. Under seismic and wind loading conditions, the passive pressure and frictional resistance may be increased by one-third.

8.6.3 Footing Reinforcement

Reinforcement for footings should be designed by the structural engineer based on the anticipated loading conditions. Footings for moderately loaded wood-frame structures that are supported in low expansive soils should have No. 4 bars, two top and two bottom.

8.6.4 Footing Observations

All footing trenches should be observed by a representative of the project geotechnical consultant to verify that they have been excavated into competent soils prior to placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square. All loose, sloughed or moisture-softened soils and/or any construction debris should be removed prior to placing of concrete. Excavated soils derived from footing and/or utility trenches should not be placed in building slab-on-grade areas or exterior concrete flatwork areas unless the soils are compacted to at least 90 percent of maximum dry density.

8.7 CONCRETE SLAB ON-GRADE

Concrete slabs will be placed on properly compacted fill as outlined in Section 8.2. Moisture content of subgrade soils should be maintained near the optimum moisture content. At the time of the concrete pour, subgrade soils should be firm and relatively unyielding. Any disturbed soils should be excavated and then replaced and compacted to a minimum of 90 percent relative compaction.

Slabs should be designed to accommodate low expansive fill soils. The structural engineer should determine the minimum slab thickness and reinforcing depending upon the expansive soil condition intended use. Unless a more stringent design is recommended by the structural engineer, we recommend a minimum slab thickness of 4 inches, and reinforcement consisting of No. 3 bars spaced a maximum of 18 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid depth.

If moisture-sensitive floor covering is planned, a layer of open-graded gravel, at least 4 inches thick, should be placed below the concrete slab to form a capillary break. Alternately, moisture-proof membrane (such as 10-mil) may be utilized. The vapor barrier should be placed between sand layers (2 inches above and below) to protect the membrane from damage during construction. Gravel for use under a concrete floor slab should be clean, crushed rock that meets the gradation requirements presented below.

Sieve Size	Percentage
1 inch	100
¾ inch	90-100
No. 4	0-10

8.8 PRELIMINARY PAVEMENT DESIGN

Pavement design should be confirmed at the completion of site grading when the subgrade soils are in-place. This should include sampling and R-Value testing of the actual subgrade soils and an analysis based upon the anticipated traffic loading.

For a preliminary pavement design, recommendations for pavement design section of asphalt parking areas are provided below. These values are based on an assumed R-value of 45.

For pavement design, Traffic indexes (TI) of 4.0 and 5.5 were used for the parking areas and auto driveways, respectively. The preliminary flexible pavement layer thickness is as follows:

RECOMMMENDED ASPHALT PAVEMENT SECTION LAYER THICKNESS

Pavement Material	Recommended Thickness	
	TI = 4.0	**TI = 5.5**
Asphalt Concrete Surface Course	3 inches	4 inches
Class II Aggregate Base Course	6 inches	8 inches
Compacted Subgrade Soils	12 inches	12 inches

Asphalt concrete should conform to Sections 203 and 302 of the latest edition of the Standard Specifications for Public Works Construction ("Greenbook").

Class II aggregate base should conform to Section 26 of the Caltrans Standard Specifications, latest edition. The aggregate base course should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Method D 1557.

8.9 CORROSION AND CHEMICAL ATTACK

Concrete subject to exposure to sulfates shall comply with the requirements set forth in ACI 318, Section 4.3. Based on the available water soluble sulfate results the corrosion potential to buried concrete is negligible, i.e., exposure Class S_0, per ACI 318, Table 4.2.1. Consequently, injurious sulfate attack is not a concern with a minimum 28-day compressive strength of 2,500 psi.

Per CBC 2010, Section 1904.4, concrete reinforcement should be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.4.

The corrosion potential of the on-site materials to buried steel was evaluated in accordance with Caltrans corrosive environment evaluation criteria. Caltrans considers a site corrosive, if at least one of the following conditions exists:

- Chloride content ≥ 500 ppm;
- Soluble sulphate content ≥ 2,000 ppm;

- pH ≤ 5.5.

Observations and laboratory tests indicate that based on the Caltrans' criteria the soils at the site are considered non-corrosive. If additional recommendations are desired, it is recommended that a corrosion specialist be consulted regarding suitable types of piping and necessary protection for underground metal conduits.

8.10 UTILITY TRENCH BACKFIL

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Trench backfill materials should be placed in lifts no greater than approximately 6 inches in thickness, watered or air-dried as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should probe and test the backfills to verify adequate compaction.

As an alternative for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand exhibiting a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to verify an adequate degree of compaction and that the backfill will not be subject to settlement.

Where utility trenches enter the footprint of the buildings, they should be backfilled through their entire depths with on-site fill materials, sand-cement slurry, or concrete rather than with any sand or gravel shading. This "Plug" of less- or non-permeable materials will mitigate the potential for water to migrate through the backfilled trenches from outside of the buildings to the areas beneath the foundations and floor slabs.

9.0 CONSTRUCTION CONSIDERATIONS

Based on our field exploration program, earthwork can be performed with conventional construction equipment.

9.1 TEMPORARY DEWATERING

Groundwater was not encountered within our exploratory borings. Based on the anticipated excavation depths, it is unlikely that dewatering will be required during construction.

9.2 CONSTRUCTION SLOPES

An Excavation during construction should be conducted so that slope failure and excessive ground movement will not occur. The short-term stability of excavation depends on many factors, including slope angle, engineering characteristics of the subsoils, height of the excavation and length of time the excavation remains unsupported and exposed to equipment vibrations, rainfall and desiccation.

Where space permits, and providing that adjacent facilities are adequately supported, open excavations may be considered. In general, unsupported slopes for temporary construction excavations should not be expected to stand at an inclination steeper than 1:1 (horizontal:vertical). The temporary excavation side walls may be cut vertically to a height of 3 feet and then laid back at a 1:1 slope ratio above a height of 3 feet.

Surcharge loads should be kept away from the top of temporary excavations a horizontal distance equal to at least one-half the depth of excavation. Surface drainage should be controlled along the top of temporary excavations to preclude wetting of the soils and erosion of the excavation faces. Even with the implementation of the above recommendations, sloughing of the surface of the temporary excavations may still occur, and workmen should be adequately protected from such sloughing.

Special care should be exercised when excavating adjacent to the property boundaries. Excavation along the property boundaries should be performed in a repeating “ABC” sequence to prevent exposing significant lengths of the existing building foundation at any one time. First, all the slots designated as “A” should be excavated, backfilled and recompacted. The procedure should continue with the “B” slots and end with the “C” slots. The width of each slot should not exceed 8 feet. If any evidence of potential instability is observed, revised recommendations such as narrower slot cuts may be necessary. All slot excavation and backfilling procedures should be performed under the observation and testing of a qualified geotechnical engineer.

9.3 POST INVESTIGATION SERVICES

Final project plans and specifications should be reviewed prior to construction to confirm that the full intent of the recommendations presented herein have been applied to design and construction. Following review of plans and specifications, observation should be performed by the geotechnical engineer during construction to document that foundation elements are founded on/or penetrate onto the recommended soils, and that suitable backfill soils are placed upon competent materials and properly compacted at the recommended moisture content.

10.0 CLOSURE

The conclusions, recommendations, and opinions presented herein are: (1) based upon our evaluation and interpretation of the limited data obtained from our field and laboratory programs; (2) based upon an interpolation of soil conditions between and beyond the borings; (3) are subject to confirmation of the actual conditions encountered during construction; and, (4) are based upon the assumption that sufficient observation and testing will be provided during construction.

If parties other than GeoBoden are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or providing alternate recommendations.

If pertinent changes are made in the project plans or conditions are encountered during construction that appear to be different than indicated by this report, please contact this office. Significant variations may necessitate a re-evaluation of the recommendations presented in this report.

11.0 REFERENCES

California Building Code, 2013 Volume 2.

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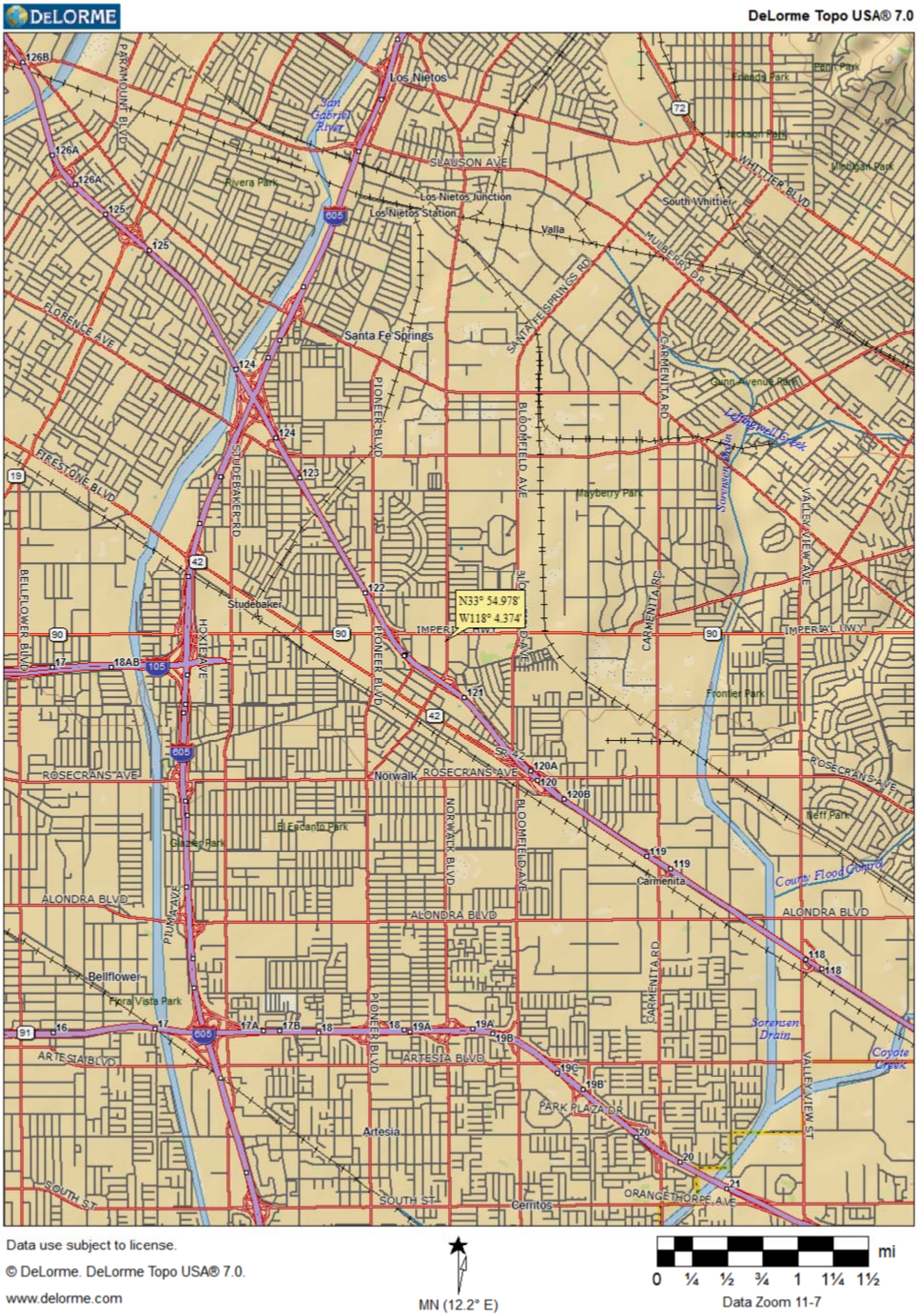
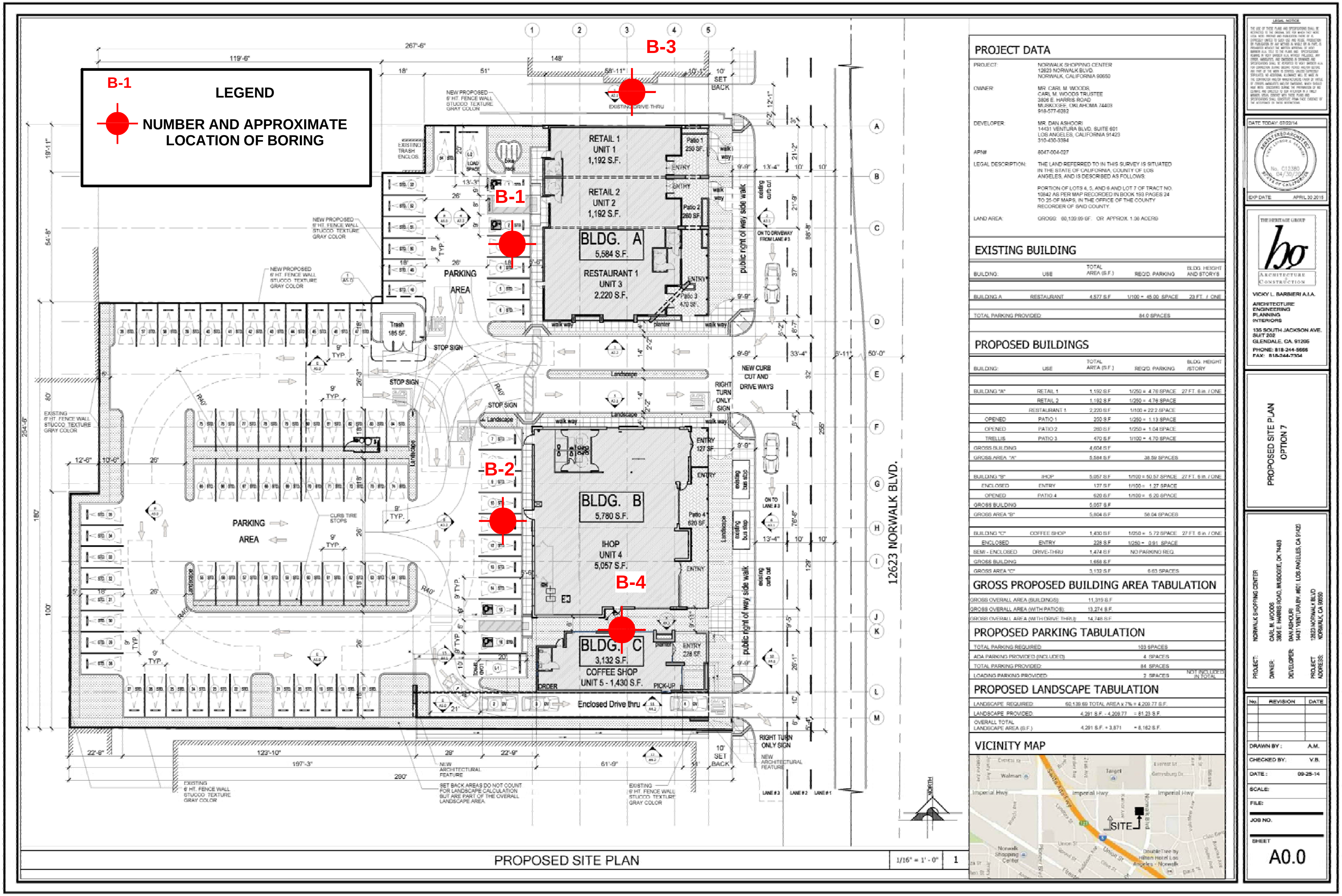
FIGURES



GEOBODEN INC. Geotechnical Consultants	SITE VICINITY MAP Proposed Norwalk Shopping Center 12623 Norwalk Boulevard Norwalk, California	Figure By S.R. Map No. XX Date 01-16-15	Project No. Norwalk-1-01 Figure No. 1



GEOBODEN INC.

Geotechnical Consultants

BORING LOCATION PLAN
Proposed Norwalk Shopping Center
12623 Norwalk Boulevard
Norwalk, California

Figure By S.R.	Project No. Norwalk-1-01
Map No. XX	Figure No.
Date 01-16-14	2

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM

PROPOSED NORWALK SHOPPING CENTER
12623 NORWALK BOULEVARD
NORWALK, CALIFORNIA

Prior to drilling, the proposed borings were located in the field by measuring from existing site features.

A total of 4 exploratory borings (B-1 through B-4) were drilled using a CME drill rig equipped with 6-inch outside diameter (O.D.) hollow-stem augers. GeoBoden of Irvine, California, performed the drilling. The boring locations are shown on Figure 2.

Depth-discrete soil samples were collected at selected intervals from the exploratory borings using a 2 ½ -inch inside diameter (I.D.) modified California Split-barrel sampler fitted with 12 brass ring of 2 ½ inches in O.D. and 1-inch in height and one brass liner (2 ½ -inch O.D. by 6 inches long) above the brass rings. The sampler was lowered to the bottom of the boreholes and driven 18 inches into the soil with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler the lower 12 inches is shown on the blow count column of the boring logs.

After removing the sampler from the boreholes, the sampler was opened and the brass rings and liner containing the soil were removed and observed for soil classification. Brass rings containing the soil were sealed in plastic canisters to preserve the natural moisture content of the soil. A Bulk sample of near surface soil was collected from selected exploratory borings and placed in plastic bags. Soil samples and bulk sample collected from exploratory borings were labeled, and submitted to the Cal Land laboratory for physical testing.

Standard Penetration Tests (SPTs) were also performed at alternative depths in Borings. The SPT consists of driving a standard sampler, as described in the ASTM 1586 Standard Method, using a 140-pound hammer falling 30 inches. The number of blows required to drive the SPT sampler the lower 12 inches of the sampling interval is recorded on the blow count column of the boring logs.

The undersigned geotechnical engineer recorded the soil classifications and descriptions on field logs using the Unified Soil Classification System as described by the American Society for Testing and Materials (ASTM) D 2488-90, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)." The final boring logs were prepared from the field logs and are presented in this Appendix.

At the completion of the sampling and logging, the exploratory borings were backfilled with the drilled cuttings.

BORING NUMBER B-1

GEOBODEN, INC.

CLIENT Mr. Dan Ashouri
PROJECT NAME Proposed Norwalk Shopping Center
PROJECT NUMBER Norwalk-1-01
PROJECT LOCATION 12623 Norwalk Boulevard, Norwalk, CA
DATE STARTED 1/8/15 **COMPLETED** 1/8/15
GROUND ELEVATION **HOLE SIZE** 6 inches
DRILLING CONTRACTOR GeoBoden Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY C.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	SILTY SAND (SM): light olive brown, moist, ~70% fine sand, ~30% fines [FILL]										
	SILTY SAND (SM): olive brown, moist, ~30% fines, ~70% fine sand [NATIVE]										
5		SS S-1		16			11				
	SAND w. SILT (SP-SM): light olive brown, moist, ~10% fines, ~90% sand										
10		SS S-2		19							12
	SANDY SILT (ML): light olive brown, moist, ~30% fine sand, ~70% fines										
15		SS S-3		26			19	43	31	12	67
	SILTY SAND (SM): dark grayish brown, moist, ~30% fines, ~70% sand										
20		SS S-4		19							32
25		MC S-5		21							
	POORLY-GRADED SAND w. SILT (SP-SM): light gray, moist, ~10% fines, ~90% fine to medium grained sand										
30		SS S-6		26							
	SANDY SILT (ML): light olive brown, moist, ~30% fine sand, ~70% fines										
35											

(Continued Next Page)

GEOBODEN, INC.

CLIENT Mr. Dan Ashouri
PROJECT NAME Proposed Norwalk Shopping Center
PROJECT NUMBER Norwalk-1-01
PROJECT LOCATION 12623 Norwalk Boulevard, Norwalk, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
35		SANDY SILT (ML): light olive brown, moist, ~30% fine sand, ~70% fines *(continued)*	SS S-7		32				39	29	10	69
		SILTY SAND (SM): pale olive, moist, ~30% fines, ~70% fine sand										
40			SS S-8		39							
45		~40% fines, ~60% fine sand	SS S-9		38							
50			SS S-10		36							

Bottom of borehole at 51.5 feet below ground surface. No groundwater was encountered. Boring was backfilled with cuttings.

BORING NUMBER B-2

GEOBODEN, INC.

CLIENT Mr. Dan Ashouri
PROJECT NAME Proposed Norwalk Shopping Center
PROJECT NUMBER Norwalk-1-01
PROJECT LOCATION 12623 Norwalk Boulevard, Norwalk, CA
DATE STARTED 1/8/15 **COMPLETED** 1/8/15
GROUND ELEVATION **HOLE SIZE** 6 inches
DRILLING CONTRACTOR GeoBoden Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY C.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		SAND w. SILT (SP-SM): olive brown, moist, ~90% fine sand, ~10% fines [FILL]										
3		POORLY-GRADED SAND (SP): light olive brown, moist, ~95% medium sand, ~5% fines [NATIVE]										
5			MC R-1		34		105	5				
10			MC R-2		34		102	4				
13		SANDY CLAY (CL): dark grayish brown, moist, ~30% fine sand, ~70% fines										
15			MC R-3		33		101	16				
18		POORLY-GRADED SAND w. SILT (SP-SM): light olive brown, moist, ~10% fines, ~90% sand										
20			MC R-4		32							

Bottom of borehole at 21.5 feet below ground surface. No groundwater was encountered. Boring was backfilled with cuttings.

BORING NUMBER B-3

GEOBODEN, INC.

CLIENT Mr. Dan Ashouri
PROJECT NAME Proposed Norwalk Shopping Center
PROJECT NUMBER Norwalk-1-01
PROJECT LOCATION 12623 Norwalk Boulevard, Norwalk, CA
DATE STARTED 1/8/15 **COMPLETED** 1/8/15
GROUND ELEVATION **HOLE SIZE** 6 inches
DRILLING CONTRACTOR GeoBoden Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY C.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SAND w. SILT (SP-SM): grayish brown, moist, ~90% fine sand, ~10% fines [FILL]										
		POORLY-GRADED SAND (SP): grayish brown, moist, ~95% medium sand, ~5% fines [NATIVE]										
5			MC R-1		22		103	7				
10			MC R-2		30		106	5				
		SANDY CLAY (CL): dark grayish brown, moist, ~30% fine sand, ~70% fines										
15			MC R-3		22		101	18				
		POORLY-GRADED SAND (SP-SM): light olive brown, moist, ~5% fines, ~95% sand										
20			MC R-4		34		108	4				

Bottom of borehole at 21.5 feet below ground surface. No groundwater was encountered. Boring was backfilled with cuttings.

BORING NUMBER B-4

GEOBODEN, INC.

CLIENT Mr. Dan Ashouri

PROJECT NAME Proposed Norwalk Shopping Center

PROJECT NUMBER Norwalk-1-01

PROJECT LOCATION 12623 Norwalk Boulevard, Norwalk, CA

DATE STARTED 1/8/15 **COMPLETED** 1/8/15

GROUND ELEVATION **HOLE SIZE** 6 inches

DRILLING CONTRACTOR GeoBoden Inc.

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Auger

AT TIME OF DRILLING ---

LOGGED BY C.R. **CHECKED BY**

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SAND w. SILT (SP-SM): grayish brown, moist, ~90% fine sand, ~10% fines [FILL]										
3		POORLY-GRADED SAND (SP): dark grayish brown, moist, ~95% medium sand, ~5% fines [NATIVE]										
5			MC R-1		27		104	6				
10			MC R-2		32		102	4				
13		SANDY CLAY (CL): dark grayish brown, moist, ~30% fine sand, ~70% fines										
15			MC R-3		29		104	19				
18		SILTY SAND (SM): olive brown, moist, ~20% fines, ~80% sand										
20			MC R-4		31		106	3				

Bottom of borehole at 21.5 feet below ground surface. No groundwater was encountered. Boring was backfilled with cuttings.

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

PROPOSED NORWALK SHOPPING CENTER
12623 NORWALK BOULEVARD
NORWALK, CALIFORNIA

Laboratory tests were performed on selected samples to assess the engineering properties and physical characteristics of soils at the site. The following tests were performed:

- moisture content and dry density
- No. 200 Wash sieve
- Atterberg limits
- consolidation
- expansion potential
- corrosion

Test results are summarized on laboratory data sheets or presented in tabular form in this appendix.

Moisture Density Tests

The field moisture contents, as a percentage of the dry weight of the soils, were determined by weighing samples before and after oven drying. The dry density, in pounds per cubic foot, was also determined fir all relatively undisturbed ring samples collected. These analyses were performed in accordance with ASTM D 2937. The results of these determinations are shown on the boring logs in Appendix A.

No. 200 Wash Sieve

Quantitative determination of the percentage of soil finer than 0.075 mm was performed on selected soil sample by washing the soil through the No. 200 sieve. Test procedures were performed in accordance with ASTM Method D1140. The results of the tests are shown on the boring logs.

Atterberg Limits

Liquid limit, plastic limit, and plasticity index were determined for selected soil sample in accordance with ASTM D 4318. The soil sample was air-dried and passed through a No. 40 sieve and moisturized. The liquid and plastic limit tests were performed on the fraction passing the No. 40 sieve. Results of the Atterberg limits tests are shown graphically and presented in this Appendix.

Consolidation

The test was performed in accordance with ASTM Test method D 2345. The compression curve from the consolidation test is presented in this Appendix.

Expansion Potential

Expansion index test was performed on a representative sample of the on-site soils in accordance with Uniform Building Code. The result of the expansion test is summarized in Table B-1.

TABLE B-1 (Expansion Index Test Data)

Boring Designation	**Depth (ft)**	Expansion Index (EI)
B-1	0-5	23

Corrosion Potential

A selected soil sample was tested to determine the corrosivity of the site soil to steel and concrete. The soil sample was tested for soluble sulfate (Caltrans 417), soluble chloride (Caltrans 422), and pH and minimum resistivity (Caltrans 643). The results of corrosion tests are summarized in Table B-2.

TABLE B-2 (Corrosion Test Results)

Boring No.	**Depth (ft)**	**Chloride Content (Calif. 422)**	**Sulfate Content (Calif. 417) % by Weight**	**pH (Calif. 643)**	**Resistivity (Calif. 643) Ohm*cm**
B-1	0-5	37	0.0146	7.3	2,650

CONSOLIDATION TEST

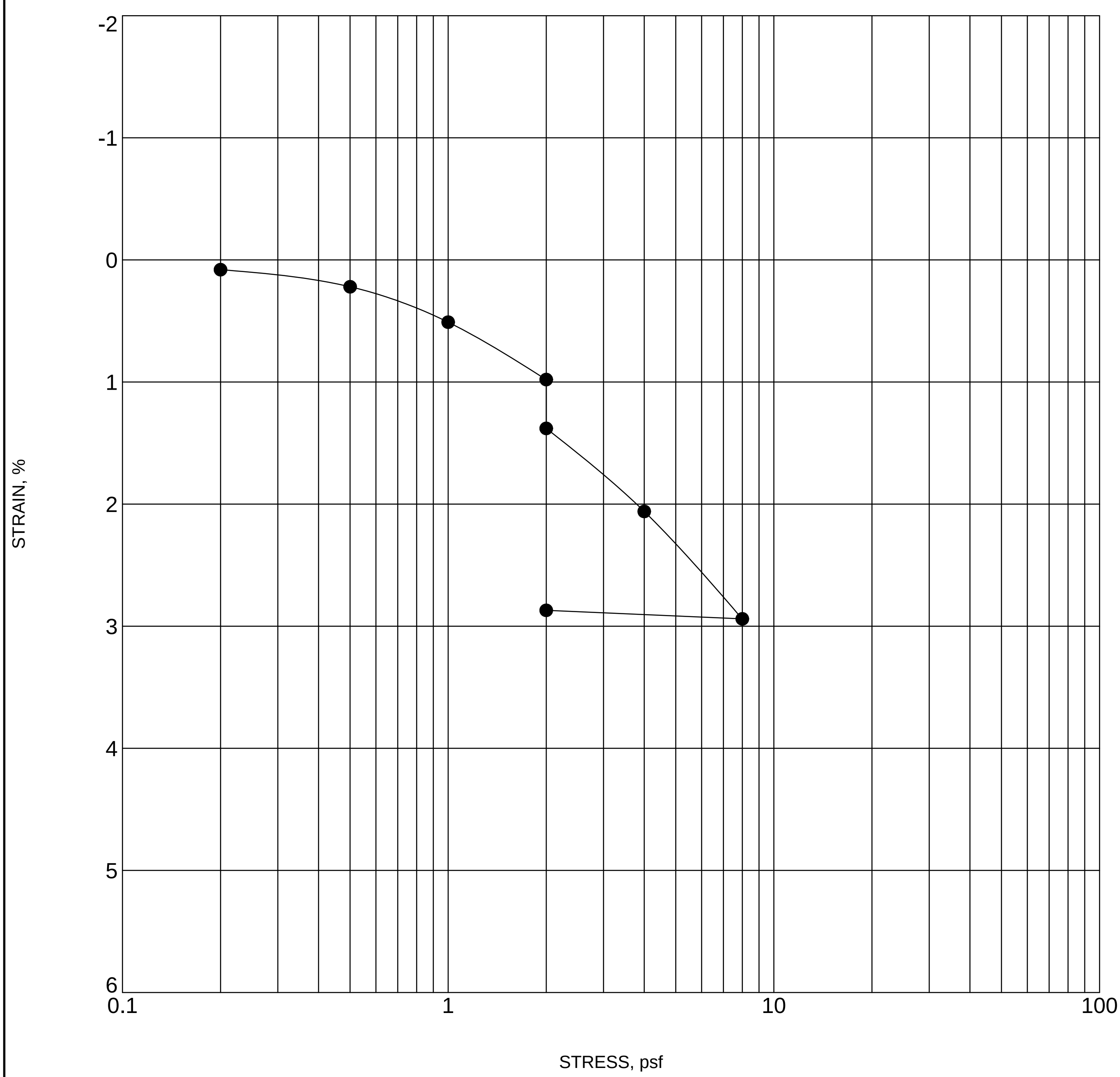
GEOBODEN, INC.

CLIENT Mr. Dan Ashouri

PROJECT NAME Proposed Norwalk Shopping Center

PROJECT NUMBER Norwalk-1-01

PROJECT LOCATION 12623 Norwalk Boulevard, Norwalk, CA



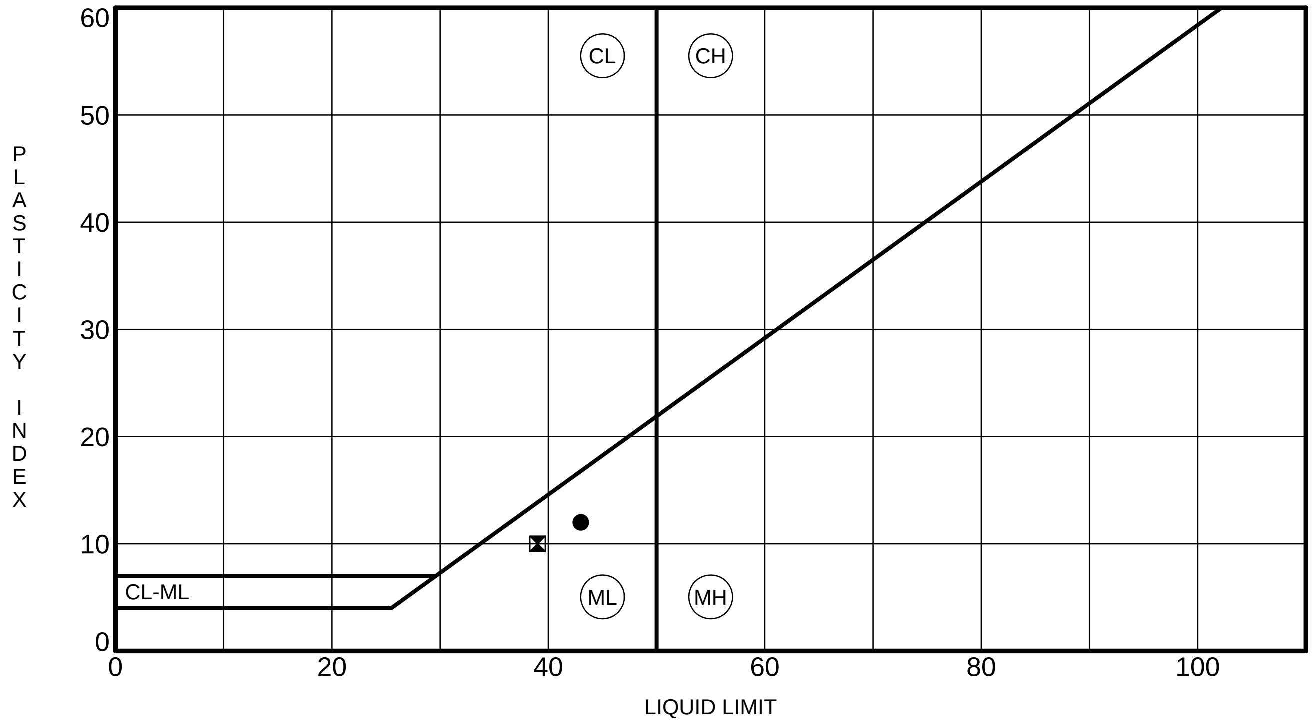
	Specimen Identification		Classification	γ_d	MC%
●	**B-2**	**10.0**	**POORLY-GRADED SAND (SP)**	**102**	**4**

ATTERBERG LIMITS' RESULTS

GEOBODEN, INC.

CLIENT Mr. Dan Ashouri **PROJECT NAME** Proposed Norwalk Shopping Center

PROJECT NUMBER Norwalk-1-01 **PROJECT LOCATION** 12623 Norwalk Boulevard, Norwalk, CA



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**15.0**	**43**	**31**	**12**	**67**	**SANDY SILT(ML)**
⧗	**B-1**	**35.0**	**39**	**29**	**10**	**69**	**SANDY SILT(ML)**

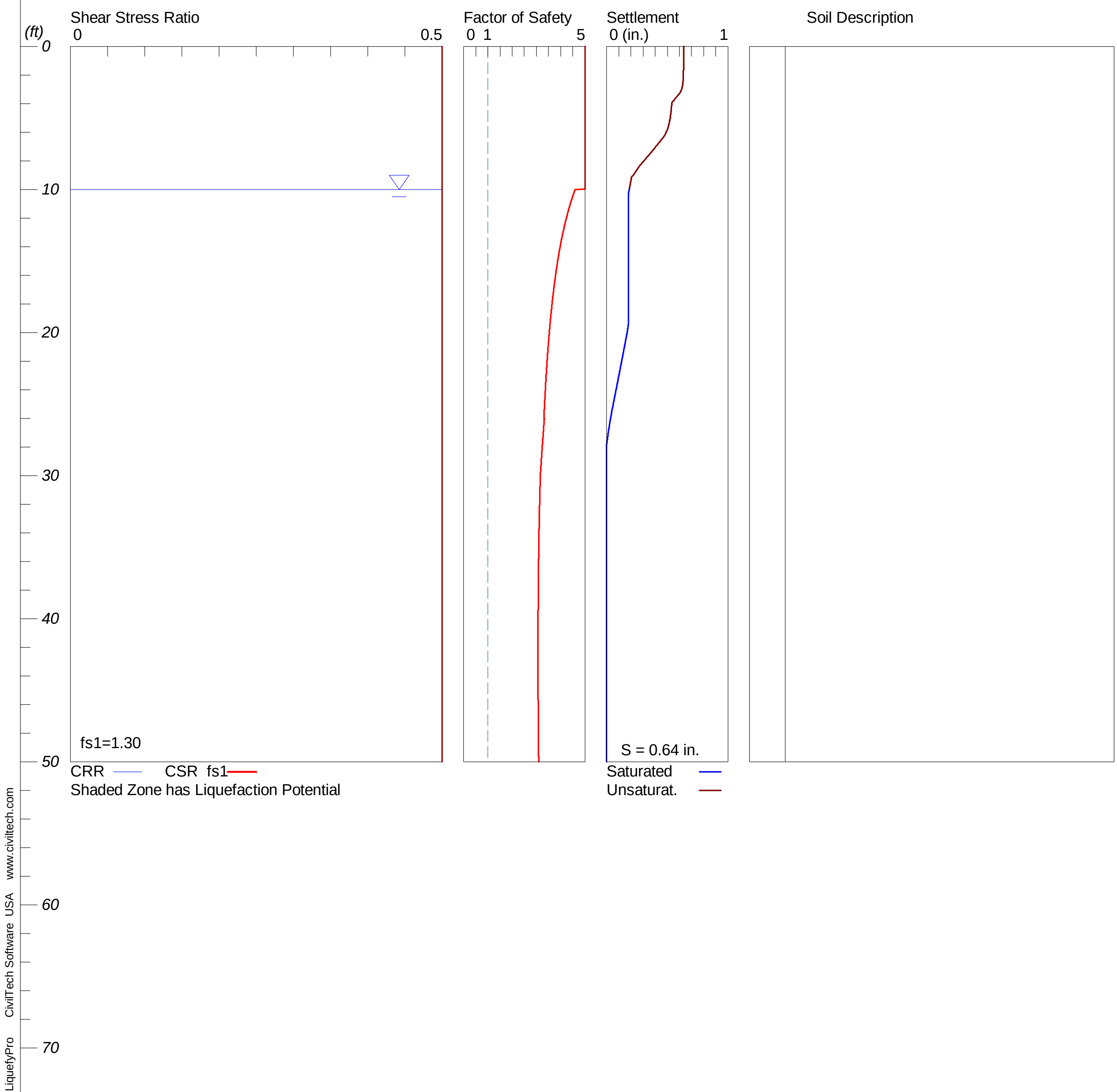
APPENDIX C
LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

Proposed Norwalk Shopping Center

Hole No.=B-1   Water Depth=10 ft

Magnitude=6.58
Acceleration=0.738g



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LIQUEFACTION ANALYSIS SUMMARY

****
Font: Courier New, Regular, Size 8 is recommended for this report.
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Input File Name: C:\Passport\GBI\Norwalk\B-1.liq
Title: Proposed Norwalk Shopping Center
Subtitle: 12623 Norwalk Boulevard, California

Surface Elev.=
Hole No.=B-1
Depth of Hole= 50.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 50.00 ft
Max. Acceleration= 0.74 g
Earthquake Magnitude= 6.58

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=50.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 50.00 ft
Max. Acceleration=0.74 g
Earthquake Magnitude=6.58
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Tokimatsu/Seed
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	16.00	125.00	0.00
10.00	19.00	125.00	12.00
15.00	26.00	125.00	67.00
20.00	19.00	125.00	32.00
25.00	21.00	125.00	32.00
30.00	26.00	125.00	32.00
35.00	32.00	125.00	69.00
40.00	39.00	125.00	69.00
45.00	38.00	125.00	69.00
50.00	36.00	125.00	69.00

Output Results:
Settlement of Saturated Sands=0.19 in.
Settlement of Unsaturated Sands=0.45 in.
Total Settlement of Saturated and Unsaturated Sands=0.64 in.
Differential Settlement=0.319 to 0.420 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.79	0.62	5.00	0.19	0.45	0.64
1.00	2.79	0.62	5.00	0.19	0.45	0.64
2.00	2.79	0.62	5.00	0.19	0.45	0.63
3.00	2.79	0.62	5.00	0.19	0.43	0.62
4.00	2.79	0.62	5.00	0.19	0.35	0.54
5.00	2.79	0.62	5.00	0.19	0.34	0.52
6.00	2.79	0.61	5.00	0.19	0.30	0.49
7.00	2.79	0.61	5.00	0.19	0.22	0.40
8.00	2.79	0.61	5.00	0.19	0.12	0.31

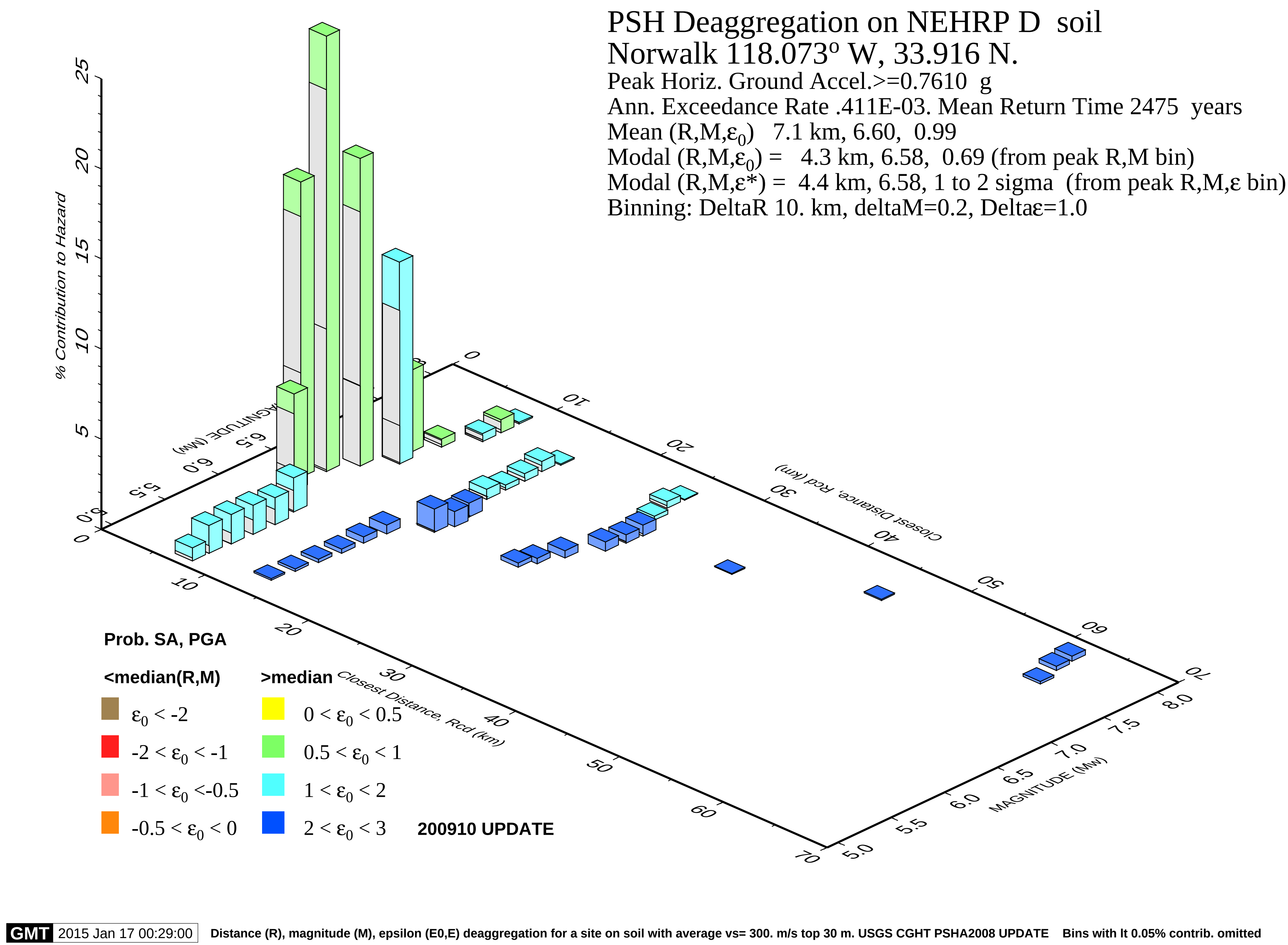
9.00	2.79	0.61	5.00	0.19	0.03	0.22
10.00	2.79	0.61	4.59	0.19	0.00	0.19
11.00	2.79	0.64	4.39	0.18	0.00	0.18
12.00	2.79	0.66	4.23	0.18	0.00	0.18
13.00	2.79	0.68	4.09	0.18	0.00	0.18
14.00	2.79	0.70	3.97	0.18	0.00	0.18
15.00	2.79	0.72	3.87	0.18	0.00	0.18
16.00	2.79	0.74	3.78	0.18	0.00	0.18
17.00	2.79	0.75	3.71	0.18	0.00	0.18
18.00	2.79	0.77	3.64	0.18	0.00	0.18
19.00	2.79	0.78	3.58	0.18	0.00	0.18
20.00	2.79	0.79	3.53	0.17	0.00	0.17
21.00	2.79	0.80	3.48	0.15	0.00	0.15
22.00	2.79	0.81	3.44	0.12	0.00	0.12
23.00	2.79	0.82	3.40	0.10	0.00	0.10
24.00	2.79	0.83	3.37	0.08	0.00	0.08
25.00	2.79	0.84	3.33	0.06	0.00	0.06
26.00	2.79	0.85	3.31	0.03	0.00	0.03
27.00	2.80	0.85	3.28	0.01	0.00	0.01
28.00	2.78	0.86	3.24	0.00	0.00	0.00
29.00	2.76	0.86	3.20	0.00	0.00	0.00
30.00	2.75	0.87	3.16	0.00	0.00	0.00
31.00	2.73	0.87	3.14	0.00	0.00	0.00
32.00	2.71	0.87	3.13	0.00	0.00	0.00
33.00	2.70	0.87	3.12	0.00	0.00	0.00
34.00	2.68	0.86	3.10	0.00	0.00	0.00
35.00	2.67	0.86	3.10	0.00	0.00	0.00
36.00	2.65	0.86	3.09	0.00	0.00	0.00
37.00	2.64	0.86	3.08	0.00	0.00	0.00
38.00	2.62	0.85	3.07	0.00	0.00	0.00
39.00	2.61	0.85	3.07	0.00	0.00	0.00
40.00	2.59	0.85	3.07	0.00	0.00	0.00
41.00	2.58	0.84	3.06	0.00	0.00	0.00
42.00	2.57	0.84	3.06	0.00	0.00	0.00
43.00	2.55	0.83	3.06	0.00	0.00	0.00
44.00	2.54	0.83	3.06	0.00	0.00	0.00
45.00	2.53	0.82	3.07	0.00	0.00	0.00
46.00	2.51	0.82	3.07	0.00	0.00	0.00
47.00	2.50	0.81	3.07	0.00	0.00	0.00
48.00	2.49	0.81	3.08	0.00	0.00	0.00
49.00	2.47	0.80	3.08	0.00	0.00	0.00
50.00	2.46	0.80	3.09	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



PSH Deaggregation on NEHRP D soil
Norwalk 118.073° W, 33.916 N.
Peak Horiz. Ground Accel.>=0.7610 g
Ann. Exceedance Rate .411E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 7.1 km, 6.60, 0.99
Modal (R,M,ε_0) = 4.3 km, 6.58, 0.69 (from peak R,M bin)
Modal (R,M,ε*) = 4.4 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
Closest Distance, Rcd (km)
MAGNITUDE (Mw)
Prob. SA, PGA
<median(R,M)
>median
ε_0 < -2
-2 < ε_0 < -1
-1 < ε_0 <-0.5
-0.5 < ε_0 < 0
0 < ε_0 < 0.5
0.5 < ε_0 < 1
1 < ε_0 < 2
2 < ε_0 < 3
200910 UPDATE
GMT 2015 Jan 17 00:29:00
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 300. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted