SUPPLEMENTAL
AGENDA MATERIAL
for Supplemental Packet 1

Meeting Date: **November 12, 2019**

Item Number: **24**

Item Description: **ZAB Appeal: 2701 Shattuck Avenue, Use Permit #ZP2016-0244**

Submitted by: **Timothy Burroughs, Director, Planning and Development Department**

This supplemental material includes a letter from the City's consulting Geotechnical Engineer (as referenced on page 7 of the staff report to City Council), providing its peer review of the 2013 geotechnical report submitted by the applicant team, which is also included within Supplemental Attachment 1 for reference.

Staff have also provided Supplemental Attachment 2, a shadow study summary provided by the applicant for staff's evaluation of the development "Plan B" proposed for approval by Council.

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

October 30, 2019
Z5169

TO: Sharon Gong
Senior Planner
CITY OF BERKELEY
1947 Center Street, 2nd floor
Berkeley, California 94704

SUBJECT: **Geotechnical Peer Review**
RE: Lin; New Multi-Story and Multi-Use Structure
ZP2016-0244
2701 Shattuck Avenue

At your request, we have completed a geotechnical peer review of the proposed land use permit application at the subject property using:

- Geotechnical Investigation (report) prepared Amso Consulting Engineers, Inc., dated July 15, 2013; and
- Geotechnical Engineer's Response to Appellant Concerns (letter-report) prepared by The Sutton Group, Inc., dated August 27, 2019.

In addition, we have reviewed pertinent technical geologic maps, historic USGS topographic maps, and reports from our office files. We also have completed a recent reconnaissance of the site and vicinity, along with a review of available GIS layers published by the City through the Community GIS Portal.

DISCUSSION

Based on our review of the referenced reports along with communication with the Project Planner, we understand that the applicant proposes to construct a new multi-story (4 or 5 story) structure with parking. In the Base Project description, the parking area will extend three stories below the ground surface. The Proposed Project description includes approximately 7-foot deep elevator parking shafts/pits below the ground surface. The subject site is not located within a seismic hazard zone as mapped by the California Geological Survey. The lot is zoned as a commercial lot in the south area of Berkeley, and it neighbors a restricted two-family residential zone to the east. The lot has been identified as an Environmental Management Area and we understand that subsurface excavation and dewatering will be subject to review by the Toxic's Management Division, as applicable. The lot is currently vacant, covered with surficial decomposed asphalt.

The purpose of this geotechnical peer review is to, at a minimum, address the following concerns: 1) the existence/non-existence of a creek on the site, 2) the constructability of floors of underground parking with parking lifts/pit on the site (Base Project), and 3) the constructability of a floor of above-ground parking with parking lifts/pit on the site (Proposed Project). Our geotechnical peer review does not include evaluation of detailed construction level plans; however, we have reviewed planning documents available on the City's project website.

SITE CONDITIONS AND GEOTECHNICAL EVALUATIONS

A Geotechnical Consultant (Amso Consulting Engineers) has advanced a subsurface exploration program at the site which included four subsurface borings to a maximum depth of 35 feet below the ground surface. Groundwater levels were measured at depths between 8.5 and 10 feet below the ground surface during the drilling program advanced in June of 2013. The Geotechnical Consultant reported to have encountered up to 5 feet of artificial fill underlain by native stiff silty clay to the maximum depth explored. The Geotechnical Consultant completed geotechnical laboratory testing including Atterberg limits testing on two samples of surficial soil that yielded plasticity indices of 14 and 11 percent.

The California Geological Survey (CGS) has mapped the historic high groundwater at depths of approximately 8 feet below the ground surface at the subject site. As previously mentioned, the site is not located within a liquefaction hazard zone of required investigation, nor is the site located in an earthquake induced landslide hazard or fault rupture hazard zone delineated by the CGS. The site is mapped within Qf deposits related to deposition of older alluvial fans across the bay margin.

Based on our review of published topographic and other maps it appears that the site is not currently characterized by the presence of an open creek. Derby Creek appears to have been a historic stream that has since been destroyed during development of the area. The site vicinity has been developed with residential and commercial improvements, and drainage in the site vicinity is currently characterized by sheet flow to the west primarily along paved surface streets with traffic furniture (i.e. speed bumps) where it is ultimately intercepted by City maintained storm drain systems and conduits. Derby Creek is not depicted as a culverted creek on pertinent maps provided by the City. Based on flood hazard assessments and mapping by FEMA, the subject site is within an area of minimal flood hazard.

CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the referenced reports, it appears that the proposed site development concepts are constrained from a geotechnical perspective by the relatively shallow groundwater table, undocumented and potentially compressible artificial fill, and strong ground shaking. A Geotechnical Consultant (Amso) has completed a geotechnical subsurface site investigation identifying subsurface soil and groundwater conditions that impact site construction. We understand that since completion of the original

Geotechnical Investigation, the applicant's consultant has retired and is no longer the Geotechnical Engineer of Record for the project.

We generally concur with the findings of The Sutton Group, Inc. regarding the concern of the purported existence of Derby Creek. Derby Creek appears to have been a historic creek that has since been artificially filled-in and destroyed as part of regional development. The remaining watershed is now primarily accommodated by City maintained drainage improvements. **Consequently, we conclude that Derby Creek does not currently exist as either an open or culverted creek in the vicinity of the site.** Also, the subject property is not included within City published lists of properties affected by culverted or open creeks. Based on our review of the referenced letter report by The Sutton Group, Inc., we find that it is unclear if they or another firm intends to take responsibility as the Geotechnical Engineer of Record as the project continues.

In regards to concern number two, the geotechnical feasibility of the approximately 23-foot deep excavation and pit construction is dependent on the hydrogeologic conditions and planned construction methods to be used. The proposed design measures to be utilized for construction of the subsurface stories will likely be impacted by shallow groundwater and have not yet been developed or peer reviewed. Dewatering and shoring (e.g. tie-backs or internal bracing) for subsurface structures are relatively common practices in the Bay Area; however, we note that certain design measures, monitoring, and hydrogeologic practices are prudent to ensure successful construction and mitigation of potential adverse effects to neighboring improvements. These measures may be economically prohibitive to the project budget. We note that the Base Project would require the groundwater table to be temporarily lowered below the proposed base of the excavation. Extensive and continued dewatering may result in settlement of the immediate surrounding areas impacted by a lowered water table. We understand that a hydrogeologic study, to determine the potential impacts that a cone of depression from the dewatering may impose on nearby improvements, has not been completed. Also, the treatment and discharge of the pumped water would need to be evaluated by and conform with the requirements of the Regional Water Quality Control Board and Toxic's Management District, as applicable. Monitoring of structures and improvements in the surrounding area to evaluate potential distress would be prudent for Base Project construction. **We conclude that construction of the Base Project, given completion of the above noted prudent measures and considerations, is geotechnically feasible.**

In regards to concern number three, the Proposed Project which includes **parking shafts/pits to be excavated to depths above or approximately equivalent to the historic high groundwater table are geotechnically feasible** provided adequate shoring and construction measures are imposed to maintain stability of the excavations. Subsurface excavations extending greater than 5 feet below the ground surface will require shoring per CalOSHA requirements. If excavations are advanced along the property boundary, additional shoring measures should be considered to protect existing roadways and structures. Minor dewatering may be necessary depending on the ultimate depth of temporary excavations. Excavations and dewatering measures should be reviewed by the appropriate City and regional agencies as necessary to conform with local regulations.

In conclusion, it appears that Derby Creek does not currently exist in the vicinity of the site, and the Base and Proposed Projects are geotechnically feasible given that appropriate measures and considerations are implemented prior to, during and after construction. Construction of the Base Project subsurface stories would likely cost significantly more to construct compared to those included as part of the Proposed Project. In addition to the concerns addressed above, we recommend the following be clarified or considered prior to respective planning and building permit approvals:

1. **Clarifications and Supplemental Geotechnical Considerations** – The Project Geotechnical Consultant should provide a statement in writing confirming that they assume responsibility as the Geotechnical Engineer of Record and either fully accept the results of the referenced Geotechnical Investigations or will complete a separate Geotechnical Investigation for the project.

   Assuming that both the currently proposed Base and Proposed Projects should be addressed as part of the geotechnical investigation, the Project Geotechnical Consultant should: 1) provide updated seismic design criteria consistent with the currently adopted code, 2) discuss the anticipated hydrostatic uplift forces that will impact the Base and Proposed Project, 3) discuss dewatering that will be necessary for construction of the Base and Proposed Project, 4) provide recommendations for monitoring of potential distress to neighboring roadway improvements and structures due to dewatering and adjacent excavations associated with either the Base Project or Proposed Project. These monitoring recommendations should include thresholds of horizontal and vertical movement that would result in stoppage of work and commencement of additional shoring, as well as recommendations to evaluate the spatial dewatering impacts to the area including locations for monitoring wells and draw down wells for the Base and Proposed Projects.

   The Project Geotechnical Consultant should discuss whether it is appropriate to contract with a certified hydrogeologist (CHG) registered with the state of California to develop the necessary site investigation required to establish appropriate dewatering and monitoring measures for the Base and Proposed Projects.

   Documentation to address the above should be provided by the Project Geotechnical Consultant to the City for review and approval by the appropriate City staff, or equivalent, prior to approval of subject planning permits.

2. **Geotechnical Plan Review** - The Project Geotechnical Consultant should review and approve all geotechnical aspects of the final project building and grading plans (i.e., site preparation and grading, site surface and subsurface drainage improvements, and design parameters for foundations and basement retaining walls) to ensure that their recommendations have been properly incorporated.

   The results of the plan review should be summarized by the Geotechnical Consultant in a letter with appropriate laboratory testing results and evaluations and submitted to the City Engineer for review and approval prior to issuance of building permits.

3. **Geotechnical Construction Inspections** - The Project Geotechnical Consultant should inspect, test (as needed), and approve all geotechnical aspects of the project construction. The inspections should include, but not necessarily be limited to: site preparation and grading, site surface and subsurface drainage improvements, and excavations for foundations prior to the placement of steel and concrete. Temporary shoring measures should be reviewed and approved by the Project Geotechnical Consultant.

   The results of these inspections and the as-built conditions of the project should be described by the Geotechnical Consultant in a letter and submitted to the City Engineer for review prior to final (granting of occupancy) project approval.

LIMITATIONS

This geotechnical peer review has been performed to provide technical advice to assist the City with its discretionary permit decisions. Our services have been limited to review of the documents previously identified. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.
CITY GEOTECHNICAL CONSULTANT

Ted Sayre
Engineering Geologist
CEG 1795

David T. Schrier
Principal Geotechnical Engineer
GE 2334

DTS:CS:TS

AMSO CONSULTING ENGINEERS
SOILS, FOUNDATIONS & ENVIRONMENTAL ENGINEERING

ACE

731 SYCAMORE AVENUE, HAYWARD, CALIFORNIA 94544
Phone (510) 690-0714, Fax: (510) 690-0721, email: basil@amsoconsulting.com

July 15, 2013
Project 3599

Ms. Liz Beaubois, P.M.
Axis Development Group
580 California Street, 16th Floor
San Francisco, California 94104

Subject: Geotechnical Investigation For
2701 Shattuck Avenue Building
Berkeley, California

Dear Ms. Beaubois:

This report presents the results of our geotechnical investigation of the property located at 2701 Shattuck Avenue in Berkeley, California.

We understand that Axis Development Group is proposing to construct a five story residential building on this property. The first level will have an at-grade parking, lobby and a restaurant. Within the garage space, two underground pits will be constructed for car matrix systems. Four stories of residential units will be constructed over a concrete podium covering the first level. Access to the parking will be from Derby Street.

We were provided with an electronic copy of a Site Plan that shows the location of the proposed buildings. This plan was used to prepare our site plan (Figure 2) that shows the location of our exploration holes that were made as part of this geotechnical investigation.

SCOPE OF WORK

We performed the following work for this geotechnical investigation.

1. Reviewed geologic and geotechnical information in our files pertinent to the site and the surrounding area.

2. Explored, sampled and classified subsurface soils by means of four small diameter exploration borings. At the end of drilling all holes were backfilled with soil/cement mixture.

3. Performed laboratory test on bulk samples and selected soil samples obtained from the exploration holes to determine their pertinent index and engineering characteristics.

4. Reviewed and analyzed of the information collected above.

5. Developed site seismic characteristics in accordance with Section 1613 of the California Building Code.

6. Prepared this report presenting our findings, conclusions, and geotechnical recommendations.

FINDINGS

Surface Conditions

The site is located in the City of Berkeley at the southeast corner of Shattuck Avenue and Derby Street. The site is almost level with a ground elevation of about 160 feet above Mean Sea Level, based on the U.S.G.S Topographic Maps. The site for the proposed housing development is bound by residential buildings east and by a paved parking for a commercial building on the south.

At the time of our subsurface exploration in June 2013, the site was occupied by a commercial building along the south portion of the site. The rest of the site was covered with paved parking.

Site Geology and Subsurface Conditions

Figure 4 shows a portion of a published geologic map of the area. This map shows the site to be underlain by alluvial deposits. This was confirmed by our subsurface exploration drill holes. Subsurface conditions at the site were explored by means of four small diameter exploration borings. These exploration holes were advanced to between 20 and 35 feet below existing ground surface. Within the depths of our exploration, the native soils at the site consist of alluvial deposits of sand, silt and clay.

Based on the results of our exploration borings, surface soils at the site consist of fill soils comprising of silty clay (CL) and contains varying amounts of sand and gravels. This clayey fill soils is weak and of low plasticity and contains organics. This fill soil is about 5 feet thick.

Below this layer of fill, the site is underlain by a layer of stiff silty clay (CL) of low plasticity and low to moderate potential for expansion and extends to the maximum depth of our exploration.

At the time of our subsurface exploration in June 0f 2013, ground water was encountered at a depth of between 8 ½ and 10 feet below existing ground surface.

The descriptions given above pertain only to the subsurface conditions found at the site at the time of our subsurface exploration performed in June of 2013. Subsurface conditions, particularly ground water levels and the consistency of the near-surface soils, will vary with the seasons.

Detailed descriptions of the materials encountered in the borings and cone penetration tests are given on the appended boring logs together with the results of some of the laboratory tests performed on selected samples obtained from the drill holes.

Seismic Considerations

This site is located within the seismically active San Francisco Bay region but outside any of the Alquist-Priolo Earthquake Fault Zones. The following faults are closest to the site.

Fault	Distance to Fault		Maximum Moment
	Miles	Kilometers	Magnitude
HAYWARD (Total Length)	1.2	1.9	7.1
SAN ANDREAS (1906)	17	28	7.9
CALAVERAS (No.of Calaveras	13	20	6.8
CONCORD - GREEN VALLEY	15	24	6.9
RODGERS CREEK	16	26	7
SAN GREGORIO	20	32	7.3
GREENVILLE	19	31	6.9

Seismic hazards can be divided into two general categories, hazards due to ground rupture and hazards due to ground shaking. Since no active faults are known to cross this property, the risk of earthquake-induced ground rupture occurring across the project site appears to be remote. Based on historic records and on the known general seismicity of the San Francisco Bay region, we consider it probable that during the next 50 years the site will be shaken by at least one earthquake of Richter Magnitude 6.5 or greater, and by numerous earthquakes of lesser Magnitude, all having epicentral locations within about 20 miles of the site.

Should a major earthquake occur with an epicentral location close to the site, ground shaking at the site will undoubtedly be severe, as it will for other property in the general area. Even under the influence of severe ground shaking, the soils that underlie the area proposed for development are unlikely to liquefy.

Seismic Design Parameters

The following general site seismic parameters may be used for design in accordance with the California Building Code.

Site Class: **D** (Stiff Soil Profile)

Mapped Acceleration Parameters: $\mathbf{S_s}$ (for short periods) = 1.89g
$\mathbf{S_1}$ (for 1-second period) = 0.72g

Site Coefficient: $\mathbf{F_a}$ (for short periods) = 1.0
$\mathbf{F_v}$ (for 1-second period) = 1.5

Adjusted Maximum Considered EQ Spectral Response Acceleration Parameters:

$$\mathbf{S_{MS} = F_a * S_s} = 1.89g$$
$$\mathbf{S_{M1} = F_v * S_1} = 1.08g$$

Design Spectral Response Acceleration Parameters:

$$\mathbf{S_{DS} = 2/3 * S_{MS}} = 1.26g$$
$$\mathbf{S_{D1} = 2/3 * S_{M1}} = 0.72g$$

Seismic Design Category: **D**

We should point out that the structural seismic design is not intended to eliminate damage to a structure. The goal of the design system is to minimize the loss of human life. It is unlikely that any structure can be designed to withstand the forces of a great earthquake without any damage at all.

Potential Geologic and Geotechnical Hazards

There are several potential geologic and geotechnical hazards that can affect any given site. They are discussed below, along with any required mitigation measures.

Ground Rupture:	Since no faults are believed to cross the site, it is our opinion that this is not a significant hazard to this site. No mitigation is required.
Ground Shaking:	This hazard is common to all properties in California. Mitigate by proper structural design and by following the recommendations presented in this report.
Lurching and Lateral Spreading:	Such seismically generated movements are induced in areas with weak soils near open cuts or slopes. Such conditions do not exist on this site. No mitigation is required.
Liquefaction:	The soils that underlie this site are unlikely to liquefy.
Landsliding:	The site and vicinity are flat. Landsliding is not a potential hazard to this site. No mitigation is required.
Compressible Soils:	The surface layer of fill soil is weak and compressible and has the potential of settlement under the influence of the building loads. To minimize the potential of building settlement, we recommenda that this layer of weak and organic soils be subexcavated and recompacted.
Expansive Soils:	Surface soils at this site is of low plasticity and low potential for expansion.
Erosion:	The site soils are moderately erodable. Mitigate by controlling the discharge of concentrated water, both during and after construction.

CONCLUSIONS AND RECOMMENDATIONS

The most geotechnical concern about this site is the presence of undocumented fill soil. The thickness of this fill is about 5 feet. If left untreated, this weak fill will settle under the influence of the building loads. To minimize the potential effect of this fill soil on the proposed development, we recommend that it should be it should be sub-excavated and if suitable for reuse as structural fill be re-compacted as will be described in this report.

The site is suitable for the proposed construction of the housing project provided that the recommendations presented in this report are followed during the design and construction phases.

The following recommendations, which are presented as guidelines to be used by project planners and designers, have been prepared assuming **AMSO CONSULTING ENGINEERS** will be commissioned to observe and test during site grading and foundation construction. This additional opportunity to inspect the project site will allow us to compare subsurface conditions exposed during construction with those that were observed during this investigation.

Site Preparation Grading and Compaction

- All demolition debris, building foundations, utility lines including electric, water, sanitary sewers and storm drains designated for abandonment on the Project Plans, should be dug out and removed. All debris and materials arising from demolition and removal operations should be wasted off-site.

- Existing fill soils within areas of the site to be built on or paved should be sub-excavated. The depth and horizontal limits of these excavations should be determined in the field by the Soils Engineer at the time of excavation. For planning purposes, however, it may be assumed that these excavations will extend to an **average** depth of about 5 feet below existing ground surface. **Where possible**, these excavations should extend 5 feet horizontally beyond proposed building lines.

- Soil surfaces exposed by excavations should be scarified to a depth of 10 inches, conditioned with water (or allowed to dry, as necessary) to produce a soil water content of about 3 percent above the optimum value and then compacted to 90 percent relative compaction based on ASTM Test D1557-91.

- Structural fill may then be placed up to design grades in the proposed building and pavement areas. Structural fill using on-site **inorganic soil**, or approved import, should be placed in layers, each not exceeding 8 inches thick (before compaction), conditioned with water (or allowed to dry, as necessary) to produce a soil water content of about 3 percent above the optimum value, and then compacted to at least 90 percent relative compaction based of ASTM Test D1557-91. The upper 8 inches of pavement subgrades should be compacted to about 95 percent relative compaction based on ASTM Test D1557-91.

- On-site soils proposed for use as structural fill should be inorganic, free from deleterious materials, and should contain no more than 15% by weight of rocks larger than 3 inches (largest dimension) and no rocks larger than 6 inches. The suitability of existing soil for reuse as a structural fill should be determined by a member of our staff at the time of grading. We expect that most of the existing soil will be suitable for reuse as structural fill.
- If import soil is required for use as structural fill, it should be inorganic, should have a low expansion potential (with a plasticity index of 15 percent or less) and should be free from clods or rocks larger than 4 inches in largest dimension. Prior to delivery to the site, proposed import should be tested in our laboratory to verify its suitability for use as structural fills and, if found to be suitable, further tested to estimate the water content and density at which it should be placed.

Building Foundations

The proposed buildings may be supported on conventional shallow foundations bearing on competent in-place native soil or on compacted structural fill placed as described in the Site Preparation, Grading and Compaction section of the geotechnical investigation report.

Continuous, reinforced concrete foundations may be designed to impose pressures on foundation soils up to 2500 pounds per square foot from dead plus normal live loading. Continuous foundations should be at least 12 inches wide and should be embedded at least 18 inches below rough pad grade or adjacent finished grade, whichever is lower.

Interior isolated foundations, such as may support column loads, may be designed to impose pressures on foundation soils up to 3000 pounds per square foot from dead plus normal live loading. Interior foundations should be embedded at least 18 inches below rough pad grade.

Any building foundation located close to the car matrix system pits should be embedded at least 18 inches below a 1½:1 (horizontal to vertical) imaginary line that extends up from the bottom of the underground car matrix system retaining wall foundation.

The allowable foundation pressures given previously may be increased by one-third when considering additional short-term wind or seismic loading.

Based upon our experience with similar buildings constructed on similar foundation soils, we expect the total long-term static settlement of the building to be approximately 1(±) inch. Using the design values presented above, and assuming a minimum embeddment of both continuous and isolated footings, we would expect the post-construction differential settlement of a relatively uniformly loaded structure to be no more than about 3/4 of the total settlement.

During foundation construction, care should be taken to minimize evaporation of water from foundation and floor subgrades. Scheduling the construction sequence to minimize the time interval between foundation excavation and concrete placement is important. Concrete should be placed only in foundation excavations that have been kept moist, are free from drying cracks and contain no loose or soft soil or debris.

Car Matrix System Pit Walls

The following may be used in the design calculations for reinforced concrete retaining walls that will be used for the underground car matrix system pits.

1. The average bulk density of material placed on the backfill side of the wall will be 120 pcf.

2. The vertical plane extending down from the ground surface to the bottom of the heel of the wall will be subject to pressure that increases linearly with depth as follows.

Condition	Design Pressure
Active, drained	50 pcf
At-rest, drained	70 pcf
At-rest, un-drained	90 pcf

   Active pressures should only be used for walls that are not restrained to move. Restrained walls should be designed for at-rest pressure.

3. The effects of earthquakes may be simulated by applying a horizontal line load surcharge to the stem of the wall at a rate of 15 H^2 lb/horizontal foot of wall, where H is the height of the surface of the backfill above the base of the wall. This surcharge should be applied at a height of 0.6H above the base of the wall.

4. A coefficient of "friction" of 0.3 may be used to calculate the ultimate resistance to horizontal sliding of the wall base over the ground beneath the base.

5. An equivalent fluid pressure of 350 psf/ft may be used to calculate the ultimate passive resistance to lateral movement of the ground in front of the toe of the wall and in front of any "key" beneath the toe or stem of the wall.

6. The car matrix system pit slab should be designed as a mat foundation bearing on competent native soil/bedrock or on compacted structural fill placed as described in the previous section. For mat design, a coefficient of subgrade reaction of 300 kips per square foot per foot may be used. The structural mat foundation may be designed to impose pressure on foundation soils up to 1500 pounds per square foot from dead plus normal live load.

7. If the bottom of the proposed car matrix system pit will be below the ground water table, which was measured at about 8½ feet below existing ground surface at the time of our subsurface exploration, then it should be designed for potential hydrostatic uplift pressure.

A zone of drainage material at least 18 inches wide should be placed on the backfill side of walls designed for drained condition. This zone should extend up the back of the wall to about 18 inches down from the proposed ground surface above. The upper 18 inches or so of material above the drainage material should consist of native, clayey soil.

The drainage material and the clayey soil cap should be placed in layers about 6 inches thick and moderately compacted by hand-operated equipment to eliminate voids and to minimize post-construction settlement. Heavy compaction should not be applied; otherwise, the design pressure on the wall may be exceeded.

The drainage material should consist of either Class 2 Permeable Material complying with Section 68 of the CALTRANS Standard Specifications, latest edition, or 3/4 to 1½ inch clean, durable coarse aggregate. If the coarse aggregate is chosen as the drainage material, it should be separated from all adjacent soil by Mirafi 700X or a similar filter fabric approved by the project Soil Engineer.

In areas where the basement wall will be constructed along the property line and no space is available for the drainage blanket described above, a drainage membrane (such as Miradrain 6000 or equivalent) may be used.

Any water that may accumulate in the drainage material should be collected and discharged by a 4-inch-diameter, perforated pipe placed "holes down" near the bottom of the drainage material. The perforated pipe should have holes no larger that 1/4-inch diameter.

Concrete Slabs-On-Grade

Concrete floor slabs should be constructed on compacted soil subgrades prepared as described in the section on Site Preparation, Grading and Compaction.

If dampness of floors is not objectionable, concrete slabs may be constructed directly on the water-conditioned and compacted soil subgrade.

To minimize floor dampness, however, the following general guidelines may be used to minimize moisture-related problems in concrete floor slabs-on-grade that will be covered with moisture-sensitive floor coverings, adhesives, and coatings.

1. Install a section of capillary break material at least five inches thick. The capillary break should be a free-draining material, such as 3/8" pea gravel or a permeable aggregate complying with CALTRANS Standard Specifications, Section 68, Class 1, Type A or Type B.

2. Cover the capillary break material with a high quality membrane vapor barrier. The membrane should be at least 10-mil thick.

3. To minimize the potential of accidental damage to the membrane vapor barrier and the potential of concrete slab curling, a protective cushion of sand or 3/8" pea gravel at least two inches thick should be placed between the membrane vapor barrier and the floor slab.

4. At the owner's option, the layer of protective sand mentioned above may be omitted provided that a 15 mil or thicker membrane vapor barrier (such as Stego Wrap) is used and that additional attention is given to the design of reinforcement so that potential curling stresses within the slab are addressed.

5. Consider using concrete having a low water/cement ratio to accelerate slab drying time. Use of fly ash may help reduce soluble alkali content in the slab. Water should not be added to the concrete after initial batching.

6. Water vapor emission levels and pH should be measured as required by the flooring material manufacturer prior to floor installation. Measurements and calculations should be performed in accordance with ASTM F1868-98 and F710-98.

The guidelines presented above are based on information obtained from various published sources including the American Concrete Institute (ACI) and Portland Cement Association (PCA). These guidelines are only intended to present information that can be utilized to minimize the potential of long term impact from slab moisture infiltration. The application of these procedures does not affect the geotechnical aspect of foundation performance.

Portland Cement Concrete Pavement (For Garage)

For garage slabs and traffic areas, a concrete pavement section, where traffic includes occasional light trucks, should consist of at least 5 inches of Portland cement concrete pavement on top of at least 6 inches of **Class 2 aggregate base** material placed and compacted as described in the "Site Preparation, Grading and Compaction" section of the report. Concrete pavements should be reinforced with at least No. 4 reinforcing bars placed at 12 inches on-center in both directions.

For design of Portland Cement concrete pavement section, a modulus of subgrade reaction of k= 150 kips per square foot per foot should be used for the on-site compacted soils. Concrete for vehicle pavements should have a modulus of rupture of at least 550 pounds per square foot.

The garage concrete slab does not need to be underlain by a capillary break section as described in the section for "Concrete Slabs-on-Grade" of this report.

Utility Trenches

The attention of contractors, particularly the underground contractor, should be drawn to the requirements of California Code of Regulations, Title 8, Construction Code Section 1540 regarding Safety Orders for "Excavations, Trenches, Earthwork".

For purposes of this section of the report, bedding is defined as material placed in a trench up to 1 foot above a utility pipe and backfill is defined as all material placed in the trench above the bedding.

Unless concrete bedding is required around utility pipes, free-draining sand should be used as bedding. Sand proposed for use in bedding should be tested in our laboratory to verify its suitability and to measure its compaction characteristics. Sand bedding should be compacted by mechanical means to achieve at least 90 percent compaction density based on ASTM Tests D1557-91.

Approved, on-site, inorganic soil, or imported material may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs and vehicle pavements. In these areas, backfill should be conditioned with water (or allowed to dry) to produce a soil-water content of about 5 percent above the optimum value and placed in horizontal layers not exceeding 6 inches in thickness (before compaction). Each layer should be compacted to 87-90 percent relative compaction based of ASTM Test D1557-91. The upper 8 inches of pavement subgrades should be compacted to about 90 percent relative compaction based on ASTM Test D1557-91.

Where any trench crosses the perimeter foundation line of any building, the trench should be completely plugged and sealed with compacted clay soil for a horizontal distance of at least 2 feet on either side of the foundation.

Surface Drainage

Surface drainage gradients should be planned to prevent ponding and to promote drainage of surface water away from building foundations, slabs, edges of pavements and sidewalks, and towards suitable collection and discharge facilities.

Water seepage or the spread of extensive root systems into the soil subgrades of foundations, slabs, or pavements, could cause differential movements and consequent distress in these structural elements. This potential risk should be given due consideration in the design and construction of landscaping.

Drainage ditches and bio-swales should be located at least 5 feet away from building foundations, slabs, edges of pavements and sidewalks, and towards suitable collection and discharge facilities. Unpaved drainage swales and ditches should have a gradient of about 2 percent. If drainage swales and ditches are located less than 5 feet from pavements, then the curbs should be embedded at least 6 inches below pavement subgrade elevation.

If detention system are used to collect and discharge surface water, they should be located at least 10 feet away from building foundations, slabs, edges of pavements and sidewalks. Furthermore, the bottom of the detention system should be located above an imaginary line extending at a slope of 1½ to 1 (horizontal to vertical) from the bottom of nearby building foundation.

Follow-up Geotechnical Services

Our recommendations are based on the assumption that **AMSO CONSULTING ENGINEERS** will be commissioned to perform the following services.

1. Review final grading and foundation plans prior to construction.
2. Observe and advise during clearing and stripping of the site.
3. Observe, test and advise during grading and placement of structural fill.
4. Test proposed capillary break material that will be used beneath concrete slabs-on-grade and advise on suitability.
5. Observe and advise during foundation and slab construction.
6. Observe, test and advise during utility trench backfilling.
7. Observe, test and advise during construction of pavements.

LIMITATIONS

The recommendations contained in this report are based on certain plans, information and data that have been provided to us. Any change in those plans, information and data will render our recommendations invalid unless we are commissioned to review the change and to make any necessary modifications and/or additions to our recommendations.

Subsurface exploration of any site is necessarily confined to selected locations. Conditions may, and often do, vary between and around such locations. Should conditions different from those encountered in our explorations come to light during project development, additional exploration, testing and analysis may be necessary; changes in project design and construction may also be necessary.

Our recommendations have been made in accordance with the principles and practices generally employed by the geotechnical engineering profession. This is in lieu of all other warranties, expressed or implied.

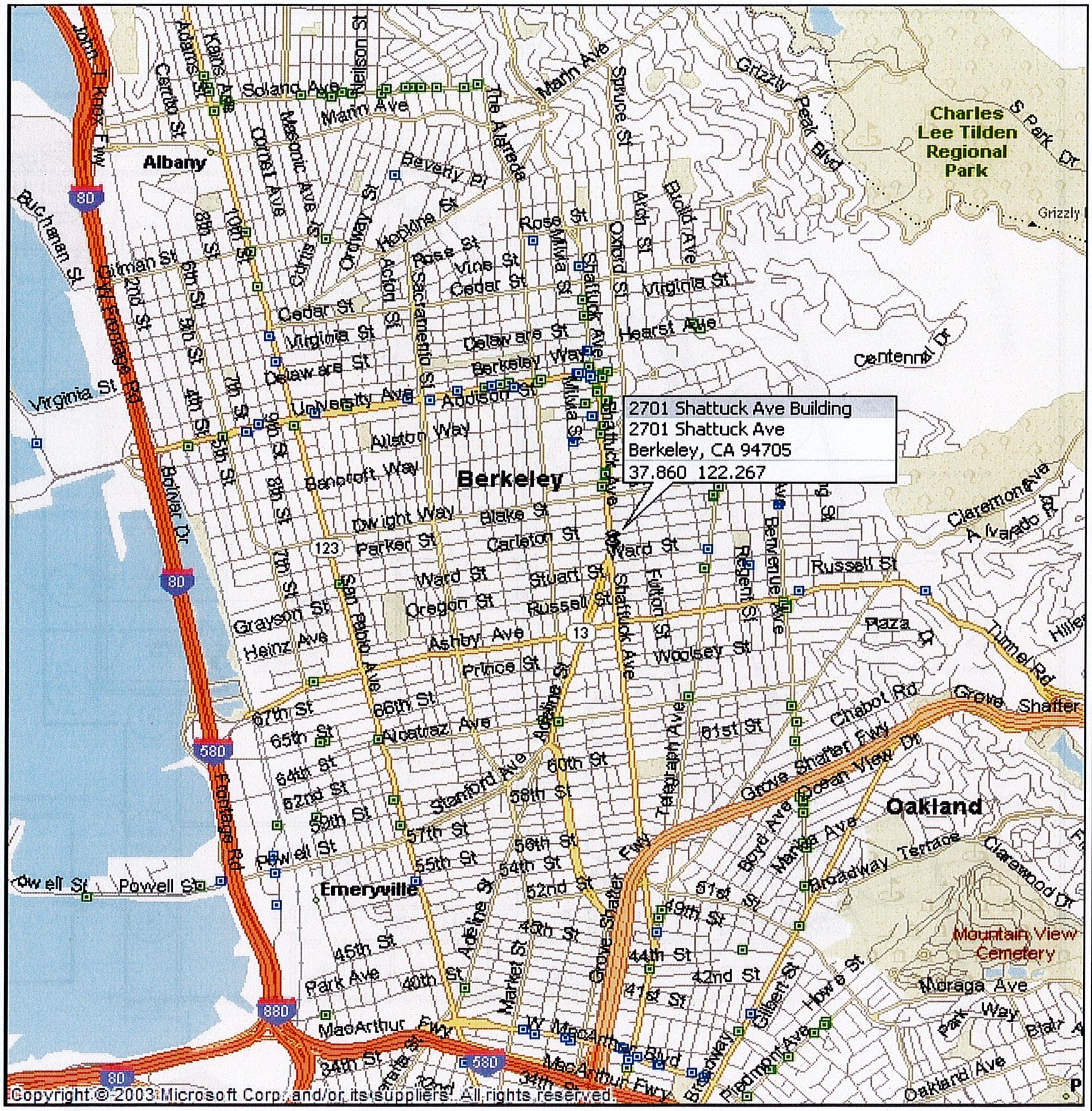
All earthwork and associated construction should be observed by our field representative, and tested where necessary, to compare the generalized site conditions assumed in this report with those found at the site at the time of construction, and to verify that construction complies with the intent of our recommendations.

Report prepared by:

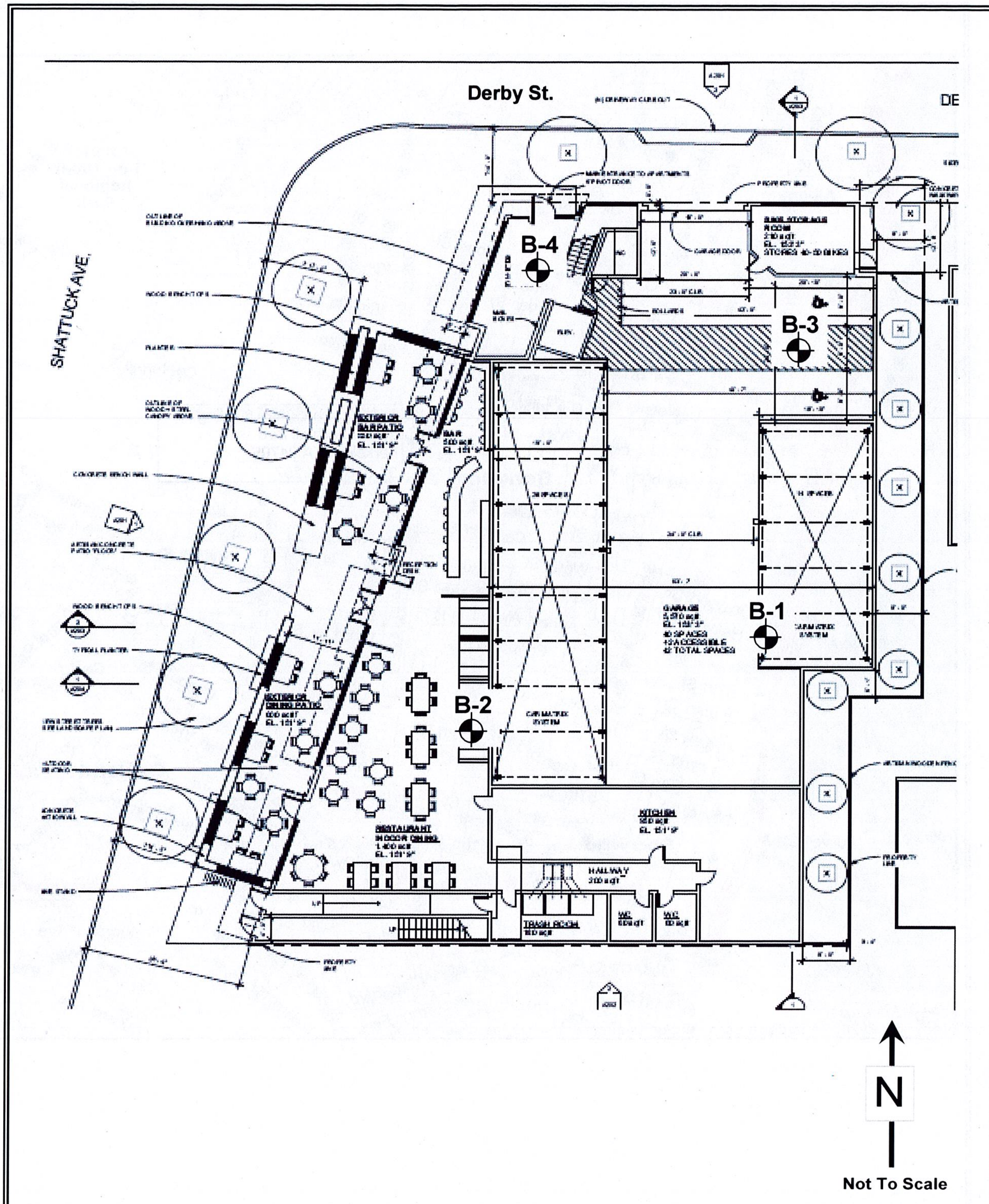
AMSO CONSULTING ENGINEERS

REGISTERED PROFESSIONAL ENGINEER
BASIL A. AMSO
No. 49998
Exp. 06/30/2015
CIVIL
STATE OF CALIFORNIA

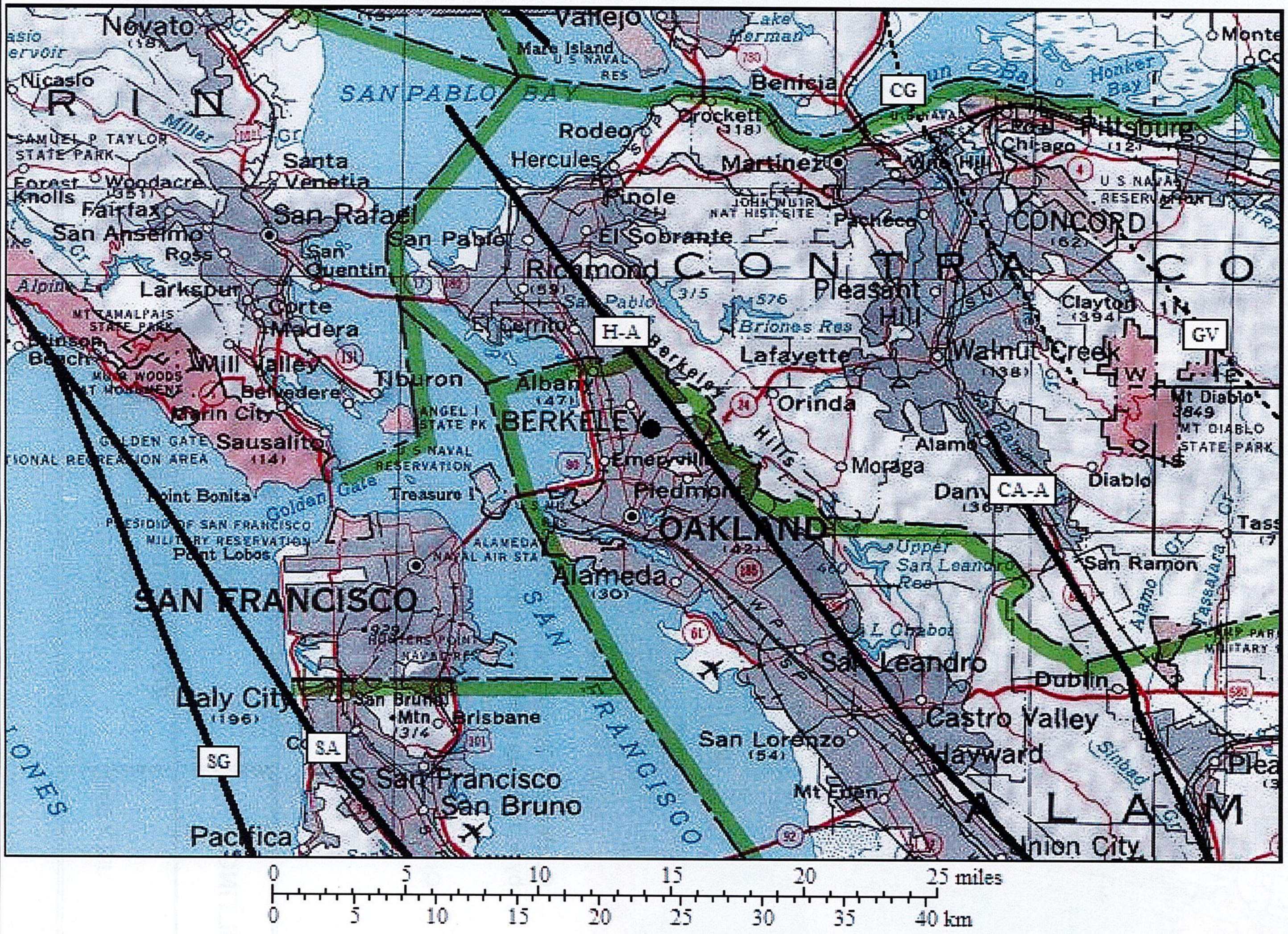
Basil A. Amso
CE 49998



AMSO CONSULTING ENGINEERS	**VICINITY MAP**	**FIGURE 1**
JULY 2013	**2701 SHATTUCK AVENUE BUILDING 2701 SHATTUCK AVENUE BERKELEY, CALIFORNIA**	**PROJECT 3599**



AMSO CONSULTING ENGINEERS	SITE PLAN AND PROPOSED LOCATION OF EXPLORATION BORINGS	FIGURE 2
JULY 2013	2701 SHATTUCK AVENUE BUILDING 2701 SHATTUCK AVENUE BERKELEY, CALIFORNIA	PROJECT 3599



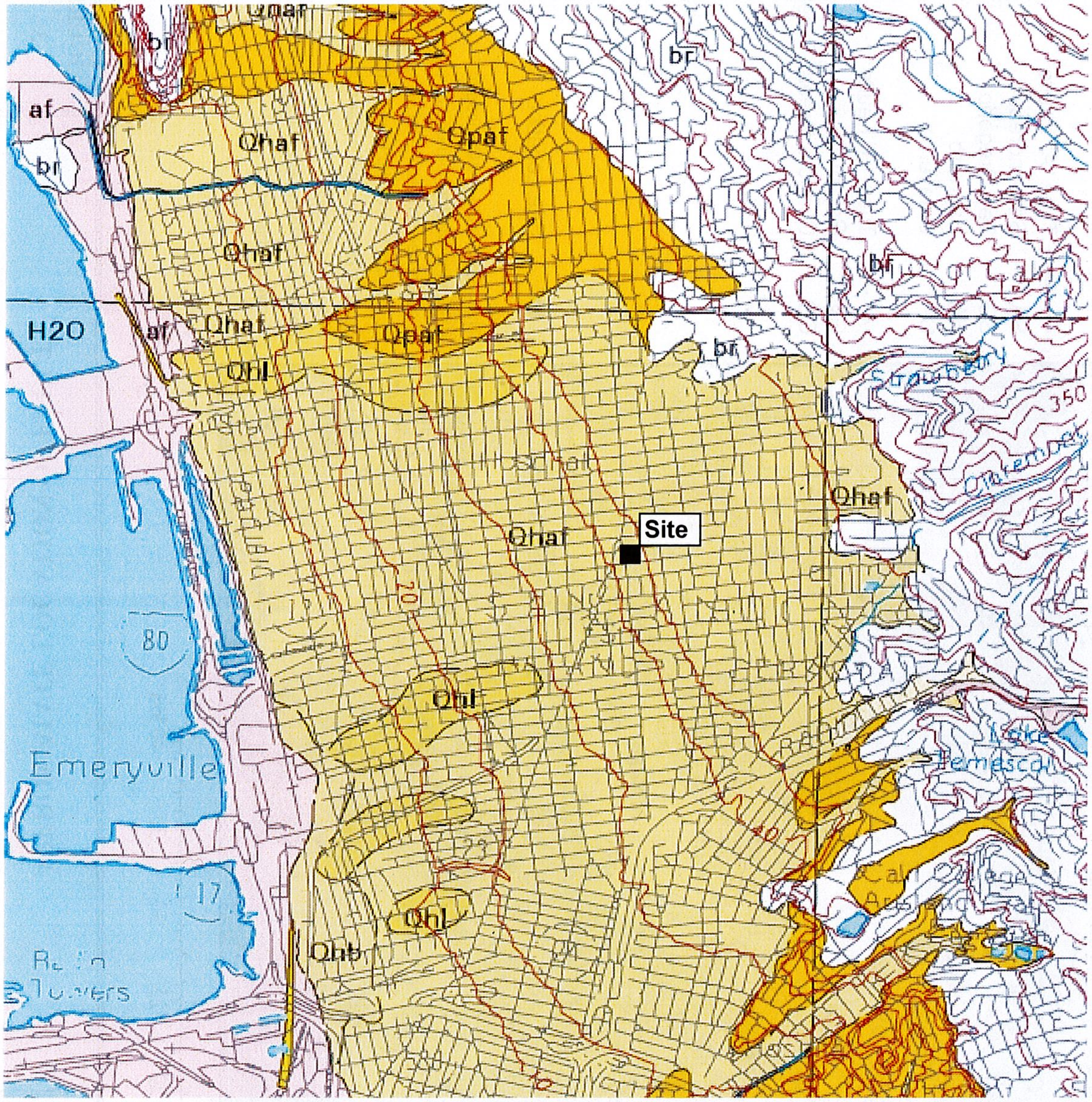
LEGEND

———	Type "A" Faults
SA	San Andreas
SG	San Gregorio
HA	Hayward
CA	Calaveras
········	Type "B" Faults
GV	Greenville
CG	Concord Green Valeey
●	Site Location

This map should not be used to determine whether or not a given property lies on a fault line. Its only purpose is to give the reader of this report a feel of aprox. distances to Types A & B fault.

Faults other than Types A & B are not shown on this map.

AMSO CONSULTING ENGINEERS	APPROXIMATE LOCATION OF FAULTS	FIGURE 3
JULY 2013	2701 SHATTUCK AVENUE BUILDING 2701 SHATTUCK AVENUE BERKELEY, CALIFORNIA	PROJECT 3599



MAP SOURCE: Quaternary Geology of Alameda County, Open File 97-97 by E.J. Helley and R.W. Graymer, 1997

LEGEND Qhaf: Alluvial Fan Deposit (Holocene)

AMSO CONSULTING ENGINEERS	GEOLOGIC MAP	FIGURE 4
JULY 2013	2701 SHATTUCK AVENUE BUILDING 2701 SHATTUCK AVENUE BERKELEY, CALIFORNIA	PROJECT 3599

APPENDIX A

Key to Exploration Logs and Boring Logs

KEY TO EXPLORATORY BORING LOGS

SOIL CLASSIFICATIONS

PRIMARY DIVISIONS			GROUP1 SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS More than half of material is larger than No. 200 sieve size	**GRAVELS** More than half coarse fraction is larger than No.4 sieve	Clean Gravels (less than5% fines*)	**GW**	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravel with fines*	**GM**	Silty gravels, gravel-sand-silt mixtures, non-plastio fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastio fines
	SANDS More than half coarse fraction is smaller than No.4 sieve	Clean Sands (less than 5%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*	**SM**	Silty sands, silt-sand mixtures, non-plastio fines
			SC	Clayey sand, sand-clay mixtures, plastio fines
FINE GRAINED SOILS More than half of material is smaller than No. 200 sieve size	**SILTS AND CLAYS** Liquid limit is less than 35		**ML**	Inorganic silts, clayey silts, rock flour, silty very fine sands
			CL	Inorganic clays of low plasticity, gravelly clay of low plasticity
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit is between35 and 50		**MI**	Inorganic silts, clayey silts and silty fine sand with intermediate plasticity
			CI	Inorganic clays, gravely clays, sandy clays and silty clays of intermediate plasticity
			OI	Inorganic clays and silty clays of intermediate plasticity
	SILTS AND CLAYS Liquid limit is greater than 50		**MH**	Inorganic silts, clayey silts, elastic silts, micaceous or diatomaceous silty or fine sandy soil
			CH	Inorganic clays of high plasticity
			OH	Organic clays and silts of high plasticity
HIGHLY ORGANIC SOILS			**Pt**	Peat, meadow mat, highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE	**200**	**40**	**10**	**4**	CLEAR SQUARE SIEVE OPENINGS **¾"**	**3"**	**12"**
Silts and Clays	Fine	Medium	Coarse	Fine	Coarse	Cobbles	Boulders
	SAND			**GRAVEL**			

RELATIVE DENSITY

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT*
VERY LOOSE	0 – 4
LOOSE	4 – 10
MEDIUM DENSE	10 – 30
DENSE	30 – 50
VERY DENSE	OVER 50

CONSISTENCY

CLAYS AND PLASTIC SILTS	UNCONFINED SHEAR STRENGTH (PSF)	BLOWS/FOOT*
VERY SOFT	0 – 250	0 – 2
SOFT	250-500	2 – 4
FIRM	500-1000	4 – 8
STIFF	1000-2000	8 – 16
VERY STIFF	2 000– 4000	16 – 32
HARD	>4000	OVER 32

SYMBOLS

▽	Initial Ground Water Level
▼	Final Ground Water Level
*	Standard Penetration Sampler
X	Modified California Sampler
D	Dames & Moore Sampler

NOTES

*BLOWS per FOOT – Resistance to advance the soil sampler in number of blows of a 140-pound hammer falling 30 inches to drive a split spoon sampler.

Stratification lines on the logs represent the approximate boundary between soil types, and the transition may be gradual.

Modified California Sampler – 2 ½ O.D. (1 7/8 Inch I.D.) sampler

Standard Penetration Sampler – 2 inch O.D. (1 3/8 Inch I.D.) split spoon sampler (ASTM D1586).

Dames & Moore Sampler – 3 inch O.D. (2.5 inch I.D.) sampler

BORING LOG

No. B 1

PROJECT **2701 Shattuck Avenue** DATE 06/26/2013 LOGGED BY BAA

DRILL RIG Continuous Flight Auger HOLE DIA. 4" SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL 18 Ft FINAL 9.5 Ft HOLE ELEVATION

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement Section												
		1										
Silty Clay; brown, damp, medium stiff;	CL/											
Fill	CH	2	x				34		20			
			x	14	1			20		100	10	3965
		3										
		4										
Silty Clay; gray, moist, medium stiff;	CL											
organic clay		5	x	9	0.7			21		101	10	1305
Silty Clay, brown, damp, stiff	CL											
		6										
		7										
		8										
Silty Sandy Clay; light brown, damp, stiff	CL	9										
to hard; with pieces of angular rock												
		10	x	23	3			18		107	10	5120
		11										
		12										
		13										
		14										
morw sandy		15	x	35	3			19		102	9	2480
		16										
		17										
		18										
		19										
		20	x	34	2.5			21		105	6	2305

BORING LOG

No. B 1

PROJECT **2701 Shattuck Avenue**	DATE 06/26/2013	LOGGED BY BAA	
DRILL RIG Continuous Flight Auger	HOLE DIA. 4"	SAMPLER X - Modified California; * - S.P.T	
GROUND WATER DEPTH INITIAL 18 Ft	FINAL 9.5 Ft	HOLE ELEVATION	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Very Sandy Silty Clay; light brown, damp, very stiff to hard; with fragments of angular gravel and crushed rock	CL											
		21										
		22										
		23										
		24										
		25	x	55	3			29		93	10	1950
		26										
		27										
		28										
		29										
		30	x	17	1.5			28		99		
		31										
		32										
		33										
		34										
		35	x	25	3.5			27		101		
Bottom of hole at 35 feet		36										
		37										
		38										
		39										
		40										

BORING LOG

No. B 2

PROJECT **2701 Shattuck Avenue** DATE 06/26/2013 LOGGED BY BAA

DRILL RIG Continuous Flight Auger HOLE DIA. 4" SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL 19 ft FINAL 9 ft HOLE ELEVATION

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement section												
		1										
Silty Clay; dark gray, to black, damp, stiff; **Fill**	CL/ CH	2										
			x	8	1.5			21		96	8	1985
		3										
		4										
soft, organic		5	x	5	0.7			22		97	10	1200
		6										
Silty Clay; light grayish brown, damp, stiff ti very stiff	CL	7										
		8										
		9				▼						
		10	x	23	3			18		98	10	3170
		11										
Silty Sandy Clay; light brown, damp, very stiff to hard	CL	12										
		13										
		14										
		15	x	33	3			24		100	10	3585
		16										
		17										
		18										
		19				▽						
		20	x	30	3			25		98	10	3390

BORING LOG

No. B 2

PROJECT **2701 Shattuck Avenue** DATE 06/26/2013 LOGGED BY BAA

DRILL RIG Continuous Flight Auger HOLE DIA. 4" SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL 19 ft FINAL 9 ft HOLE ELEVATION

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Sandy Silty Clay; light brown, damp	CL											
very stiff		21										
		22										
		23										
		24										
		25	x	28	3.5			25		98	10	3390
Bottom of hole at 25 feet												
		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

BORING LOG

No. B 3

PROJECT **2701 Shattuck Avenue** DATE 06/26/2013 LOGGED BY BAA

DRILL RIG Continuous Flight Auger HOLE DIA. 4" SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL 19 Ft FINAL 9 Ft HOLE ELEVATION

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavement section												
		1										
Silty Clay; dark gray to black, damp, medium stiff to soft	CL/ CH	2	x	11	1.2		30	21	19	96	10	1295
		3										
		4										
bloack, oranic		5	x	10	0.7			23		98	10	2640
Sandy Silty Clay; brown, damp, stiff	CL	6										
		7										
		8										
		9				▼						
becomes stiff		10	x	21	2			18		105	9	2910
		11										
		12										
		13										
		14										
		15	x	42	3.2			21		90		
		16										
		17										
		18										
		19				▽						
Bottom of hole at 2o feet		20	x	32	3			20		103	10	5450

BORING LOG

No. B 4

PROJECT **2701 Shattuck Avenue** DATE 06/26/2013 LOGGED BY BAA

DRILL RIG Continuous Flight Auger HOLE DIA. 4" SAMPLER X - Modified California; * - S.P.T

GROUND WATER DEPTH INITIAL 18 Ft FINAL 8.5 Ft HOLE ELEVATION

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT (%)	WATER CONTENT (%)	PLASTIC LIMIT (%)	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
Pavenet Section		1										
Silty Clay; dark brown to dark gray, damp soft	CL/ CH	2	x	7	0.5			21		96	8	755
		3										
brown		4										
		5	x	11	1.1			22		97	10	1050
Silty Clay; dark gray, damp, medium stiff to stiff	CL	6										
		7										
		8										
		9				▼						
		10	x	26	3			25		101	10	3045
Sandy Silty Clay; brown, damp, stiff with pieces of ruhed rock	CL	11										
		12										
		13										
		14										
		15	x	33	3.5			24		100	10	4120
		16										
		17										
		18				▽						
		19										
Bottom of hole at 20 feet		20	x	29	3							

APPENDIX B

Laboratory Test Results

PLASTICITY INDEX

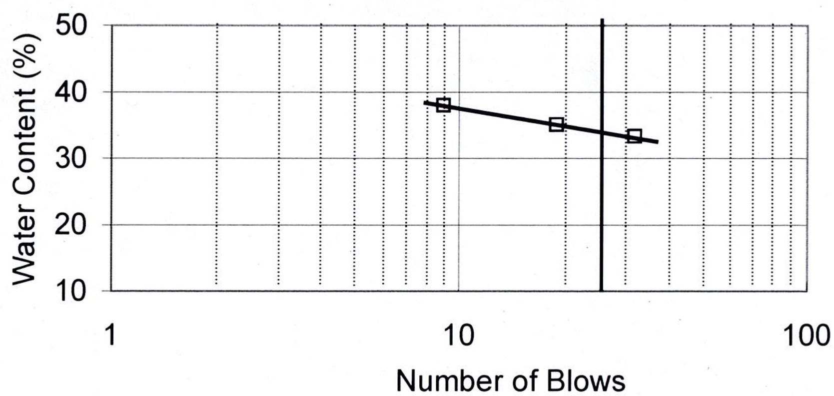
TEST DESIGNATION: ASTM D4318 OR CAL 204

Project Name:	**2701 Shattuck Avenue**	Project No.:	3599
Sample No.:	B1 @ 2.5 FT	Lab No.:	
Location		Test Date:	06/28/2013
Description:	Silty Sandy Clay	Tested By:	MT

TEST DATA

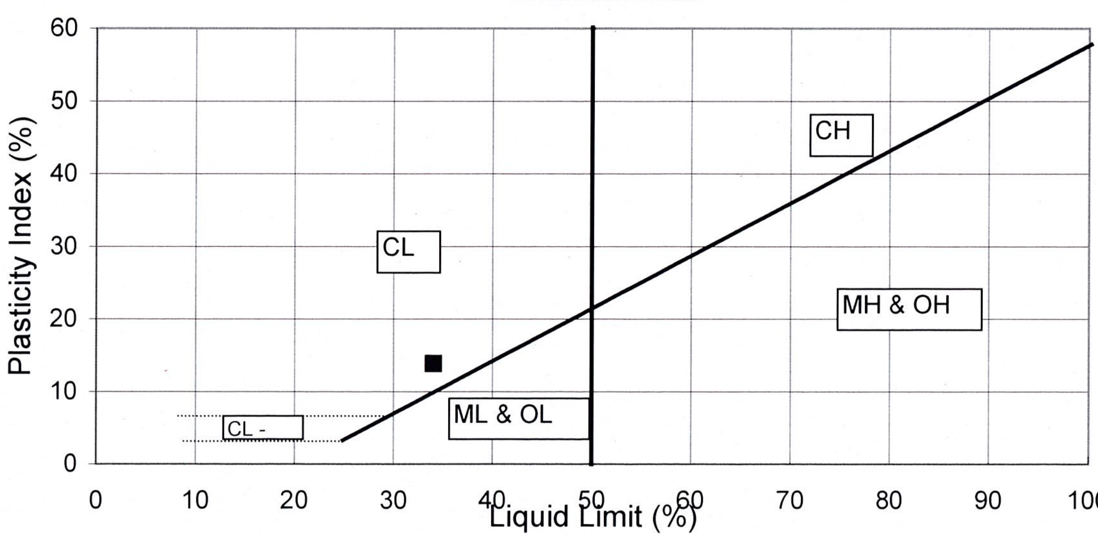
	Liquid Limit				Plastic limit			Water Content
Number of Blows	9	19	32					
Tare Number	T	O	4		5			
Tare + Wet Wt (gm)	50.13	48.04	49.02		154.50			
Tare + Dry Wt (gm)	40.64	39.63	41.38		134.00			
Tare Wt (gm)	15.67	15.67	18.48		32.20			
Wt of Water (gm)	9.49	8.41	7.64		20.50			
Soil Dry Wt (gm)	24.97	23.96	22.90		101.80			
Water Content (%)	38.01	35.10	33.36		20.14			
			Average		20.14			

Liquid Limit Test



LL	PL	PI
34	20	14

Plasticity Chart



PLASTICITY INDEX

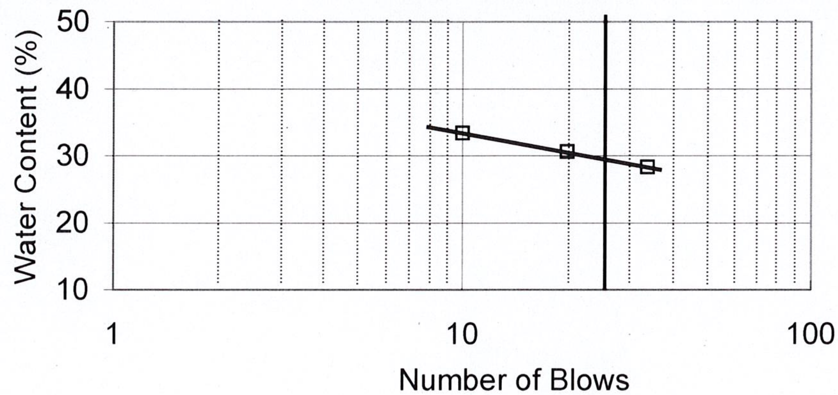
TEST DESIGNATION: ASTM D4318 OR CAL 204

Project Name:	**2701 Shattuck Avenue**	Project No.:	3599
Sample No.:	B3 @ 2 FT	Lab No.:	
Location		Test Date:	06/28/2013
Description:	Silty Sandy Clay	Tested By:	MT

TEST DATA

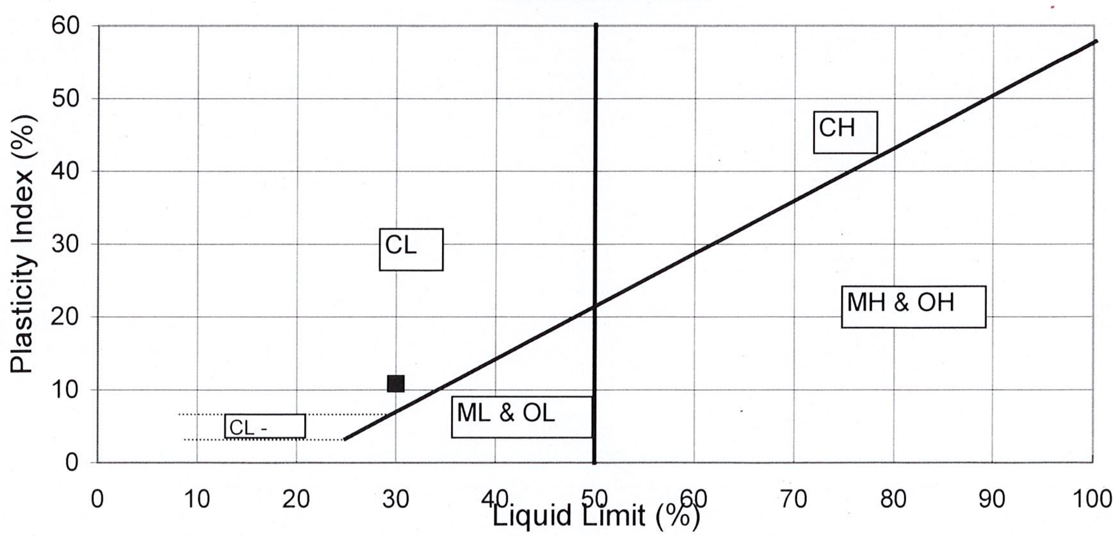
	Liquid Limit				**Plastic limit**			**Water Content**
Number of Blows	10	20	34					
Tare Number	F	11	M		5			
Tare + Wet Wt (gm)	47.35	50.63	47.51		153.45			
Tare + Dry Wt (gm)	39.43	43.15	40.47		134.10			
Tare Wt (gm)	15.71	18.74	15.64		33.34			
Wt of Water (gm)	7.92	7.48	7.04		19.35			
Soil Dry Wt (gm)	23.72	24.41	24.83		100.76			
Water Content (%)	33.39	30.64	28.35		19.20			
	Average				19.20			

Liquid Limit Test

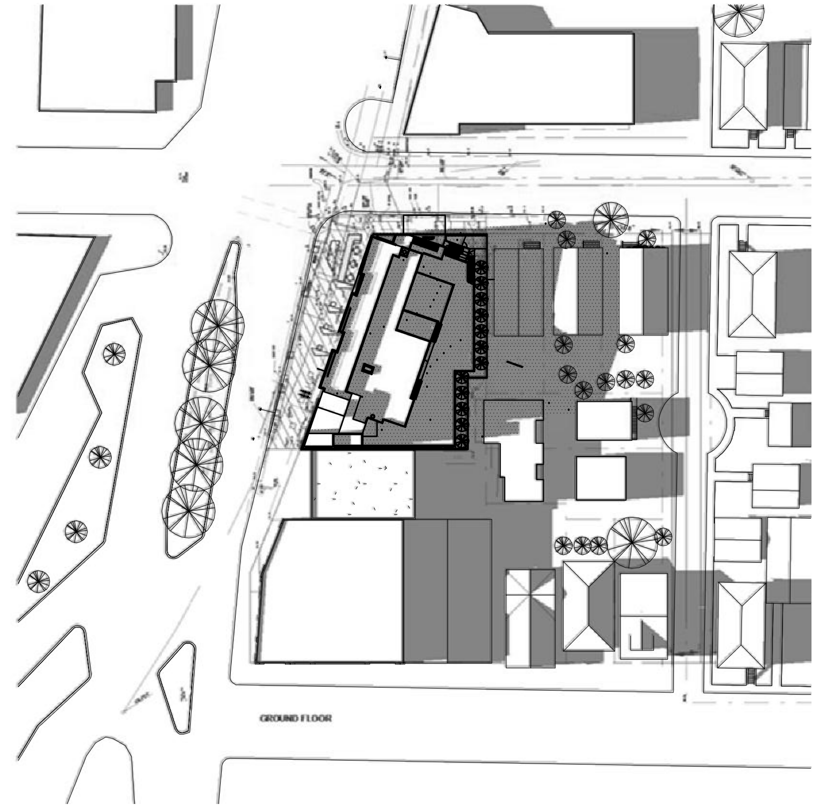


LL	PL	PI
30	19	11

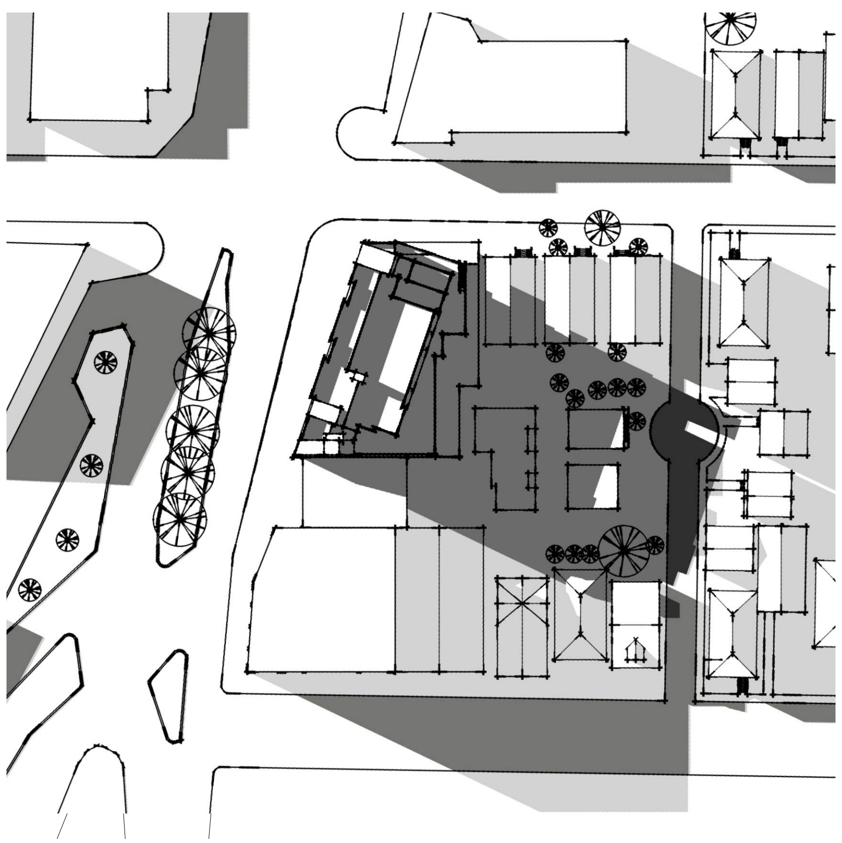
Plasticity Chart

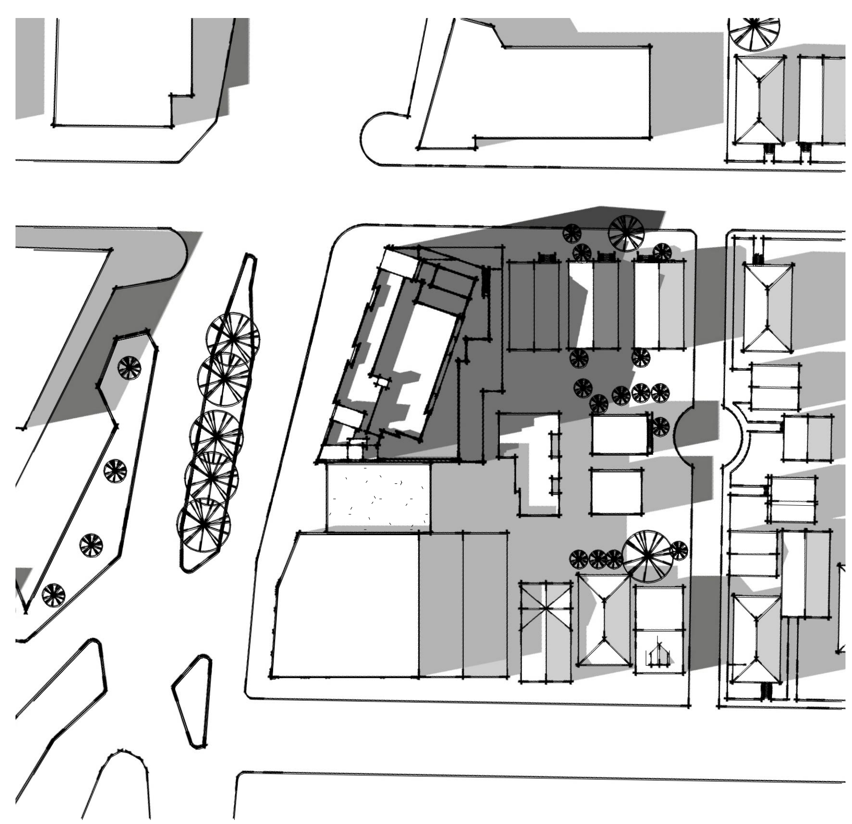


Original Plan



PLAN - B





JUNE 21 AT 6:00 PM ASSESSMENT:
The Sun does not cast shadow on Derby till 6pm. For the neighbors on Derby street, there was minimum shadows in the Original Plan. In Plan B, there is almost NO shadow on the second and third houses adjacent to site. For rest of the neighborhood, there is no substantial difference.

SEPTEMBER / MARCH 21 AT 4:00 PM
ASSESSMENT: **In Spring and Fall, there is light shadow impact. THERE IS** NO SUBSTANTIAL DIFFERENCE BETWEE**TWO PLANS.**

DECEMBER 21 AT 4:00 PM ASSESSMENT: **In Winter, Shadow duration is one hour till sunset at 5PM. For the neighbors on Derby street, in Plan B, there is 50% improvement for the third houses adjacent to site. For rest of the neighborhoods, there is no substantial difference between two plans.**

revisions	by

HDO
architects, planners
2950 camino diablo
suite 110
walnut creek, ca
94597
(925) 256-6042

2701 SHATTUCK AVENUE
BERKELEY, CALIFORNIA
FOR:
2701 SHATTUCK BERKELEY, LLC

drawing
SHADOW STUDY SUMMARY

drawn
checked
RH
date
9/25/19
scale
NO SCALE
job no
1802
sheet