Bay City Geology, Inc.

GEOTECHNICAL
ENGINEERING INVESTIGATION

PROPOSED NEW TWO-STORY
SINGLE FAMILY RESIDENCE WITH BASEMENT

Tract: G1LLETTES REGENT SQUARE, Lot: 19, Block: F
APN: 4279-001-019

247 20th Street
Santa Monica, California 90402

for

BBS Development
11990 San Vicente Blvd., Suite 100
Los Angeles, CA 90049

Project 1735

October 27, 2017

Bay City Geology

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BBS Development
Scott Moore
11990 San Vicente Blvd., Suite 100
Los Angeles, CA 90049

Subject: **GEOTECHNICAL ENGINEERING INVESTIGATION**
Proposed New Single Family Residence with Basement
247 20th Street
Santa Monica, California

Dear Mr. Moore:

Bay City Geology, Inc. is pleased to submit this geotechnical engineering report to provide recommendations for the proposed new residence.

Based on this investigation, it is our opinion that the proposed construction is feasible from a geotechnical engineering standpoint provided the recommendations contained herein are incorporated into the project design plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development. When final plans for the site development become available, or if the proposed construction is revised, the plans should be forwarded to this office for review and comment.

The scope of this investigation is limited to the project area as depicted on the Plot Map herein. This report is not a comprehensive evaluation of the entire property and may not contain sufficient information for other than the intended use. Prior to use by others, Bay City Geology, Inc. should be consulted to determine if additional work is required. If the project is delayed more than one year, this office should be contacted to verify current site conditions and prepare an update report.

We appreciate the opportunity of serving you on this project. If you have any questions pertaining to our report, or if we can be of further service, please do not hesitate to contact us. This report may not be copied. If you wish additional copies, you may order them from this office.

Respectfully submitted,
BAY CITY GEOLOGY, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
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INTRODUCTION

This report details the results of a limited Geotechnical Engineering Investigation on a portion of the subject property. The purpose of this investigation has been to ascertain the subsurface conditions pertaining to the proposed new two-story single family residence with basement. Review of the project included reconnaissance mapping, description of earth materials, determining soil structure, obtaining representative earth samples, performing laboratory testing, engineering analyses, and preparation of this report. Findings, conclusions and appropriate recommendations are included herein.

SCOPE

The scope of this investigation includes the following:

- Review of two (2) test pit explorations. Explorations were backfilled with the excavated materials.
- Preparation of the enclosed Plot Map (see Appendix I).
- Sampling of representative earth materials, laboratory testing and analyses (see Appendix II).
- Review of reference materials and available public reports at the City Santa Monica, Department of Building & Safety (see Appendix V).
- Presentation of findings, conclusions, and recommendations for the proposed project.

M & M & Co. prepared the topographic base map utilized in this investigation. Preliminary building plans were provided by the client and plotted onto the base map. It consists of one sheet plotted to a scale of one-inch equals sixteen feet.

The scope of this investigation is limited to the project area explored as depicted on the Plot Map. This report is not a comprehensive evaluation of the entire property. This report has not been prepared for use by other parties or for other purposes, and may not contain sufficient information for other than the intended use. Prior to use by others, Bay City Geology, Inc. should be consulted to determine if additional work is required. If the project is delayed more than one year, this office should be contacted to verify current site conditions and prepare an update report.

PROPOSED DEVELOPMENT

It is our understanding that the existing dwelling will be demolished and wasted from the site. The site will be developed with a new two-story single-family residence with partial basement. Grading will consist of conventional cut and retaining wall backfill methods. Final building plans have not been prepared and await the conclusions and recommendations of this investigation.

SITE DESCRIPTION

Location and Description

Access to the property is via 20th Street from San Vicente Boulevard. The property is essentially flat-lying and developed with an existing two-story single-family residence. The property is landscaped with, lawn areas, shrubs and trees. Details of the topography are depicted on the Location Map and Plot Map in Appendix I.

Drainage

Surface water at the site consists of direct precipitation onto the property. Much of this water drains as sheet flow to low-lying areas, offsite and/or to the street. The residence is partially provided with roof gutters and downspouts. Portions of the yard are serviced by an irrigation system. No area drains and/or subdrain outlet pipes were observed on the property.

Groundwater

No active surface groundwater seeps or springs were observed on the subject site. The subsurface exploration did not encounter groundwater to a depth of (20) feet. The historic high groundwater level was obtained from review of the California Division of Mines and Geology (CDMG) *Seismic Hazard Zone Report (SHZR 023) for the Beverly Hills 7.5-Minute Quadrangle* (1998, 2005), (Plate 4). Review of this report indicates that the historically highest groundwater level is on the order of (40) feet below grade. Seasonal fluctuations of groundwater levels may occur by varying amounts of rainfall, irrigation and recharge. Groundwater is not anticipated to pose a problem to the proposed project.

SUMMARY OF FINDINGS

Previous Works

The subject property was developed circa 1925. No geology and/or geotechnical reports were found on file at the City of Santa Monica covering original or subsequent construction at the site.

Stratigraphy

The site is underlain by non-marine sedimentary soils of Pleistocene time. The earth materials encountered on the subject property are briefly described below. Approximate depths and more detailed descriptions are given in the enclosed Exploration Logs (see Appendix I).

Terrace Deposits (Qt)

Quaternary terrace deposits are weathered bedrock material that have eroded from natural ascending slopes and accumulated in generally flat-lying areas followed by additional erosion and/or uplift of the local area. Terrace deposit soils onsite primarily consist of dense dark-brown to reddish-brown clayey sand (SC) with rock clasts that generally range between (1) and (2) inches in length.

Seismicity
Ground motion caused by an earthquake is likely to occur at the site during the lifetime of the development due to the proximity of several active and potentially active faults. Therefore, earthquake insurance with building code upgrades is suggested. The output from the United States Geological Survey computer program for the prediction of peak horizontal ground acceleration is provided in the Appendix III. Generally, on a regional scale, quantitative predictions of ground motion values are linked to peak acceleration and repeatable acceleration, which is a response to earthquake magnitudes relative to the fault distance from the subject property.

This seismic evaluation is designed to provide the client with current, rational, and believable seismic data that could affect the property during the lifetime of the proposed improvements. The minimum design acceleration for a project is listed in the current Building Code. It is recommended that the structural design of the proposed dwelling be based on current design acceleration practices of similar projects in the area.

The Safety Element of the City of Santa Monica General Plan established a “Hazard Management Zone” for the Santa Monica fault. The Hazard Management Zone includes all areas located between about (380) feet to nearly (500) feet north of the North branch and about (100) feet to nearly (600) feet south of the South Branch of the Santa Monica fault. The Hazard Management Zone map also indicates areas where researchers have mapped interpreted “Strong” and “Weak” geomorphic expressions of the Santa Monica fault. Leighton & Associates, Inc., March 30, 1994, published a detailed map of the Hazard Management Zone in the “Technical Background Report to the Safety Element of the City of Santa Monica General Plan”. A map showing the locations of the geomorphic expressions is shown on Plate 5.

The subject site is not located within the “Hazard Management Zone” (Plate 5). Although the State of California has not zoned the Santa Monica fault as an Earthquake Fault Zone in accordance with the Alquist-Priolo Earthquake Fault Zoning Act of 1972, the City is currently treating the fault as active. The site is located approximately (1,735) feet north of the North Branch Santa Monica Fault delineated by the City of Santa Monica. The potential for ground rupture at the site is considered moderate to high. Therefore, earthquake insurance with building code upgrades is suggested. A detailed subsurface analysis can be performed to determine the ground rupture potential on the subject site. A proposal for a detailed analysis will be prepared if requested.

If a segment of the Santa Monica fault were to extend below the proposed development, and if that segment were to rupture, large ground accelerations exceeding 1.0***g*** (where “***g***” is the acceleration due to Gravity) may be encountered at the site, which may be damaging to structures. Also, if the fault is located beneath the site, a rupture along the fault in the subsurface may propagate to the surface and result in ground rupture and surface displacement as the result of movement from the active fault. The study by Dolan *et al.* (2000) suggested that the Santa Monica fault has undergone at least six distinct surface ruptures over the past 50,000 years, with the most recent approximately 1,000 to 2,000 years ago. Such ruptures could produce displacements of (~1.1 to 2.0) meters, which would be damaging to structures.

Dolan *et al.* (2000) have evaluated the seismic potential of the Santa Monica fault system. Their report indicates that the Santa Monica fault is active and capable of producing damaging earthquakes to structures. This determination was based on paleoseismologic and geomorphologic data, generally from seismic trenches within the Veteran's Administration property in west Los Angeles.

Plate 5 depicts new locations of the Santa Monica fault that are not depicted on previous geologic maps such as, Hoots, (1932); Poland and Others, (1959), Geologic Survey Water-Supply Paper 1461; State of California Department of Water Resources Southern District Bulletin No. 104 (1961); and Dibblee (1991). The above referenced groundwater studies did not indicate a groundwater barrier in the location of the new Santa Monica fault. Based on the above data, it appears that additional geologic work is needed to determine the active location and potentially active sections of the fault. The Veteran's Administration section, in west Los Angeles, appears to be an active section of the Santa Monica fault.

The subject site is not located near a section of the Santa Monica fault (Plate 5) mapped by Leighton & Associates (1995) that has not been investigated in detail, and which is also a different section of the fault from the "Veteran's Section" (Plate 6) mapped by Dolan *et al.* (2000).

Based field reconnaissance by the engineering geologist, no significant geomorphic fault features or alignments of geomorphic fault features exist on or directly adjacent to the subject site. One method to determine if the upper ground surface has been ruptured by recent fault activity is to excavate and geologically log a seismic trench perpendicular to the direction of known existing faults. A proposal to perform seismic trenching shall be provided upon request.

There are several active and/or potentially active faults that could possibly affect the site within Los Angeles County. However, all of Southern California is in a seismically active region. Presently, the time, location, magnitude, amount of fault displacement, and duration of shaking of an earthquake cannot be accurately predicted. The most recent significantly damaging seismic event in the Los Angeles area was the January 17, 1994 magnitude 6.7 Northridge earthquake in the San Fernando Valley. The epicenter was about (1) mile south-southwest of Northridge at a focal depth of (12) miles beneath the ground surface. This was the first earthquake to strike directly under an urban area of the United States since the 1933 Long Beach earthquake. The earthquake occurred on a blind thrust fault and produced the strongest ground motions ever instrumentally recorded (~1.7*g*), in an urban setting in North America.

The high accelerations, both vertical and horizontal, lifted structures off of their foundations and/or shifted walls laterally. Damage was wide-spread: sections of major freeways collapsed, parking structures and office buildings collapsed, and numerous apartment buildings suffered irreparable damage. Significant damage occurred further away from the epicenter at Fillmore, Glendale, Santa Clarita, Santa Monica, Simi Valley, and in western and central Los Angeles. At least 33 deaths were attributed to the earthquake and over 8,700 injured. In addition, the earthquake caused an estimated $20 billion in damage, making it one of the costliest natural disasters in U.S. history. Since the earthquake occurred on an unmapped "blind thrust" fault that produced no surface rupture, the event initiated considerable scientific and geologic engineering investigations that significantly modified building codes and structural engineering.

The following is a summary of known active faults that have produced historic earthquakes and are capable of generating ground shaking at the subject property. Plate 7 depicts all known active and potentially active faults within the region. An active fault, as defined by the California Geologic Survey, Special Publication 42 (SP 42), is one which has “had surface displacement within Holocene time (about the last 11,000 years)”. A potentially active fault as defined by SP 42 is a fault in which “no known historical ground surface ruptures or earthquakes have occurred. These faults, however, show strong indications of geologically recent activity and evidence of surface displacement during Quaternary time (last 1.6 million years)".

Although there are probably some unrecognized active faults in southern California, nearly all the movement is on the well-defined active faults, any of which could cause an earthquake at any time. Location and timing of earthquakes cannot be predicted.

The San Andreas fault zone is the dominant active fault zone in California. It consists of numerous subparallel faults of varied lengths in a zone generally (0.3 to 1.5) kilometers wide in Southern California. The dip of the fault is near vertical, and the sense of motion is right-lateral. Historically, the 1857 Fort Tejon earthquake with an estimated magnitude of 7.9 ruptured the ground surface from the vicinity of Cholame to somewhere between the Cajon Pass and San Gorgonio Pass, a distance of approximately (200) miles to the southeast. The fault extends from the Gulf of California northward to the Cape Mendocino area where it continues along the ocean floor. It is approximately (750) miles in length. The San Andreas fault is estimated to produce earthquakes with magnitudes ranging from 6.8 to 8.0 with a recurrence interval of approximately 140 years. The San Andreas fault is long overdue for a major seismic event.

The San Fernando fault consists of five major en-echelon strands at least (10) miles in length. The San Fernando (Sylmar) earthquake of February 9, 1971, produced a magnitude of 6.6 along an east-west trending reverse fault with a northerly dip. The length of the surface rupture was about (9.5) miles and ground shaking lasted for approximately 60 seconds. The earthquake created a zone of discontinuous surface ruptures, named the San Fernando fault zone, which partly follows the boundary between the San Gabriel Mountains and the San Fernando - Tujunga Valleys and partly transects the northern portion of the San Fernando Valley. This latter zone of tectonic ruptures was associated with some of the heaviest property damage sustained in the region. Within the entire length of the surface ruptures, the maximum vertical offset measured on a single scarp was about (1) meter, the maximum lateral offset about (1) meter, and the maximum shortening (thrust component) about (0.9) meters. Severe ground fracturing and landslides were responsible for extensive damage in areas where faulting was not previously observed.

The Newport-Inglewood fault zone consists of several strands that extend from offshore by Laguna Beach to either merge with or be truncated by the Malibu-Santa Monica-Hollywood fault zone near Beverly Hills. The fault has a length of about (45) miles. It was the source of the "Long Beach Earthquake”, which occurred on March 10, 1933, with a magnitude of 6.3. Numerous small earthquakes have occurred in historic time along and near the fault zone. The fault zone is easily observed by an alignment of hills and mesas including Cheviot Hills, Baldwin Hills, Rosecrans Hills, Dominguez Hills, Signal Hill, Reservoir Hill, Alamitos Heights, Landing Hill, Bolsa Chica Mesa, and Newport Mesa.

The Whittier-Elsinore fault zone consists of several subparallel, overlapping and en-echelon fault strands in a zone up to (1.2) kilometers wide. It extends nearly (125) miles from the Mexican border to the northern edge of the San Fernando Valley. Recent seismicity includes the Whittier Narrows earthquake of October 1, 1987, with a magnitude of 5.9. Also, numerous close and scattered small earthquakes have occurred in historic time near and along the fault.

The Raymond fault is a combination fault with reverse and left-slip movement that acts as a groundwater barrier. The activity of the fault is attested to by the numerous geomorphic features found along its entire length of approximately (14) miles. Scattered small earthquakes have occurred north of the fault trace. It may be the source of the magnitude 5.0, December 3, 1988, Pasadena Earthquake.

In June 1995, two portions of the Malibu Coast fault zone were reclassified as active faults. On August 16, 2007, the fault zone near the east side of Malibu Bluff Park was removed from the State of California Earthquake Fault Zone map by the California Geologic Survey. The Malibu Coast fault consists of several subparallel strands in a zone as wide as (0.5) kilometers, with a length of at least (17) miles. It strikes approximately east-west and dips north about 45 to 80 degrees. Recent seismic activity includes the magnitude 5.2, January 1, 1979, and magnitude 5.0 January 18, 1989, Malibu Earthquakes. Both events caused minor injuries and minor damage to structures within Malibu and Santa Monica.

The Sierra Madre fault zone consists of five en-echelon fault strands in a zone approximately (0.5) miles wide, with a length of about (45) miles. The Sierra Madre fault system is considered capable of producing earthquakes with magnitudes ranging between 6.0 and 7.0. The most recent event was the June 28, 1991, Sierra Madre Earthquake. This earthquake occurred on the Clamshell - Sawpit Canyon fault, an offshoot of the Sierra Madre fault zone in the San Gabriel Mountains. Because of its depth and moderate size, it caused no surface rupture, though it triggered rockslides that blocked some mountain roads. Roughly $40 million in property damage occurred in the San Gabriel Valley. Unreinforced masonry buildings, were the hardest hit.

The largest and closest body of water to the subject project is the Pacific Ocean, which is approximately (1.75) miles southwest of the subject site. Properties along the coast area have a potential hazard from tsunami. Tsunamis are sea waves generally produced by earthquakes. A tsunami wave can travel for thousands of miles at high speeds, exceeding (400) miles per hour. Little data exists to evaluate the potential for a local tsunami generated off the coast of southern California.

Historically, two tsunamis have been generated off the coast of southern California. The 1812 Santa Barbara Earthquake was reported to generate (10) to (12) feet high sea waves at Gaviota. The 1927 Point Arguello Earthquake produced sea waves to (6) feet. The 1964 Alaskan Earthquake produced sea waves of less than (4) feet in the Los Angeles Harbor.

The lower threshold seismic event for tsunami development is considered to be a magnitude 6.5 earthquake. Offshore faults and the Santa Monica faults appear capable of producing a magnitude 6.5 earthquake and conceivably producing a sea wave. However, the sea floor topography in the Santa Monica Bay and Santa Barbara Channel, and the Channel Islands minimize the risk of a large tsunami to the coast.

The subject site is not located within a Tsunami Hazard Zone delineated by the State of California. Due to the distance from the Pacific Ocean of (1.75) miles, and the elevation of the site at approximately (40) feet above sea level, the potential for tsunami inundation is very low.

Seiches are waves with low-energy that form in bodies of water such as reservoir, bays, rivers and lakes. The proposed project is considerably far away from any of the noted water body types. Thus, the considerable distance from any significant water bodies precludes the potential for seiche and seich-related adverse conditions at the subject site.

The potential for lurching, surface manifestations, and disruption to existing topography from seismic shaking can occur almost anywhere in Southern California. Proper maintenance of properties can mitigate some of the potential for these types of manifestations, but the potential cannot be completely eliminated.

Liquefaction

Liquefaction is a process by which sediments below the water table temporarily lose strength and behave as a viscous liquid rather than a solid. The types of sediments most susceptible are clay-free deposits of sand and silts; gravel only occasionally liquefies. The actions in the soil which produce liquefaction are as follows: seismic waves, primarily shear waves, passing through saturated granular layers, distort the granular structure, and cause loosely packed groups of particles to collapse. The pore-water pressure between grains increases, if drainage cannot occur. If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular soil layer temporarily behaves as a viscous liquid rather than a solid.

In the liquefied condition, soil may deform with little shear resistance; deformations large enough to cause damage to buildings and other structures are called ground failures. The ease with which a soil can be liquefied depends primarily on the looseness of the material, the depth, thickness, and areal extent of the liquefied layer, the ground slope, and the distribution of loads applied by buildings and other structures.

The State of California has prepared Seismic Hazard Zone Reports and Maps to regionally map areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacement. The maps may not identify all areas that have potential for liquefaction, strong ground shaking, or other earthquake-related geologic hazards.

The subject site is not located within a Liquefaction Hazard Zone as designated on the CDMG *Seismic Hazard Zone Map of the Beverly Hills Quadrangle* (1999) (Plate 3). The historic high groundwater level was obtained from review of the CDMG *Seismic Hazard Zone Report* (*SHZR 023*) *for the Beverly Hills 7.5-Minute Quadrangle* (1998, 2005), (Plate 4). Review of this report indicates that the historically highest groundwater level is on the order of (40) feet below grade.

Groundwater was not encountered within the explorations to a depth of (20) feet below existing grade. Also the subject site is observed to be underlain by dense and competent, Pleistocene age terrace deposit soils (Qt). Due to the considerable depth to the historic groundwater table, along with the age, density, and consistency of the earth materials encountered onsite, it is the opinion of this firm that the potential for liquefaction and liquefaction settlement onsite is very low.

Additionally, the age, density, and consistency of the Pleistocene age terrace deposit soils (Qt) underlying the site preclude the potential for seismically induced dry-sand settlement. Therefore, it is the opinion of this firm that the potential for seismically induced dry-sand settlement is very low.

Also, the subject site is located within a region of Santa Monica which is generally is flat-lying with regional gradients of approximately >50 – >100:1 (horizontal to vertical). The subject site is located considerably far away from any free-face slopes. Therefore, it is the opinion of this firm that the potential for lateral spreading to occur at the subject site is very low.

Landslides

The State of California has prepared Seismic Hazard Evaluation reports to regionally map areas of potential increased risk of permanent ground displacement based on historic occurrence of landslide movement, local topographic expression, and geological and geotechnical subsurface conditions. The maps may not identify all areas that have potential for earthquake-induced landsliding, strong ground shaking, or other earthquake-related geologic hazards. The subject site is not located within an earthquake-induced landslide hazard zone on the State of California Seismic Hazard Map (Plate 3).

CONCLUSIONS

1. Based on the results of this investigation and a thorough review of the proposed development, as discussed, the project is suitable for the intended use providing the following recommendations are incorporated into the design and subsequent construction of the project. Also, the development must be performed in an acceptable manner conforming to building code requirements of the controlling governing agency.

2. Based on the State of California Seismic Hazard Maps, the subject site is not located within a liquefaction-induced hazard zone (Plate 3). Due to the age, density, and consistency of the earth materials encountered onsite, it is the opinion of this firm that the potential for liquefaction onsite is very low.

3. Based on the State of California Seismic Hazard Maps, the subject site is not located within an earthquake-induced landslide hazard zone (Plate 3).

4. The SITE CLASS based on California Building Code is D – Stiff Soil. Additional seismic design values are listed in Appendix III.

5. Based upon field observations, laboratory testing and analysis, the Quaternary terrace deposit soils (Qt) found in the test pits should possess sufficient strength to support the recommended new engineered, compacted fill pad for support of the new residence, and provide adequate foundation support for the proposed new basement.

RECOMMENDATIONS

Specific Recommendations

1. The planned demolition of the existing residence is anticipated to disturb the upper terrace deposit soils (Qt) underlying the proposed development area to an approximate depth of (1 to 2+/-) feet below existing grade. Grading and earthwork should be utilized to create a new uniform building pad area for support of the proposed new residence. The proposed new residence shall be supported on foundations bearing entirely into the recommended new properly placed engineered, compacted fill materials (CAf).

   Grading and earthwork to create a new uniform building pad area should include removals of any disturbed surficial earth materials to a minimum depth of (5) feet below grade, and expose competent terrace deposit soils (Qt). The removal excavations should extend laterally at least (3) feet beyond the planned new building-line, and extend to a depth of at least (3) feet below the proposed new foundations. All excavated on-site earth materials should be replaced as engineered, compacted fill materials (CAf).

2. The proposed development also includes the construction of a new basement under a portion of the proposed new residence. The portion of the proposed new residence to be constructed over the new basement and the new basement level retaining walls should be supported on foundations bearing entirely into the dense, competent terrace deposit soils (Qt) underlying the proposed development area at the proposed basement subgrade elevation.

3. Potentially expansive soils were observed to underlie the project area. All new foundations, slabs-on-grade, and pool shells shall be designed for a MEDIUM expansive soil condition.

4. All grading and earthwork performed for construction of the proposed development shall be performed as outlined in the Grading and Earthwork and Retaining Wall Backfilling sections of Appendix IV – General Recommendations.

5. The soils chemistry results should be incorporated into the design of the proposed development.

6. The property owner shall maintain the site as outlined in the Drainage and Maintenance section of Appendix IV – General Recommendations.

Foundation Design

The proposed new two-story residence shall be supported on conventional foundations bearing entirely into the new properly, placed engineered, compacted fill materials (CAf) recommended to be placed in the proposed development area. Additionally, the portion of the proposed new residence to be constructed over the new basement and the new basement level retaining walls shall be supported on conventional foundations bearing entirely into the dense, competent terrace deposit soils (Qt) underlying the proposed development area at the new basement subgrade elevation. Also, all new foundations shall be designed for a MEDIUM expansive soil condition.

Conventional Foundations

The proposed development may be supported by conventional foundations bearing entirely into the recommended bearing material(s). Conventional foundations should consist of either continuous footings, or pad footings and grade beams (or other suitable structural members).

The minimum conventional foundation design recommendations are given as follows:

❖ *Allowable Bearing Pressures:*
  ➢ Strip Footings — 1,500 pounds per square foot
  ➢ Column Footings — 2,000 pounds per square foot
  ➢ Maximum Allowable — 2,000 pounds per square foot

  ➢ Modulus of Subgrade Reaction — 72 kips per cubic foot

❖ *Allowable Bearing Pressure Increases:*
  ➢ For Additional Footing Width — Not Recommended
  ➢ For Additional Footing Depth — Not Recommended

❖ *Minimum Footing Widths:*
  ➢ Strip Footings — 12 inches
  ➢ Column Footings — 30 inches (square)

❖ *Minimum Footing Embedment Depths:*
  ➢ Strip Footings — 24 inches
  ➢ Column Footings — 24 inches

  ➢ All foundation embedment depths shall be measured into the recommended bearing material, below the lowest adjacent grade.

❖ *Lateral Resistance Parameters:*
  ➢ Coefficient of Friction — 0.26
  ➢ Passive Earth Resistance (acting as a fluid) — 300 pounds per square foot, per foot
  ➢ Maximum Passive Earth Pressure — 3,000 pounds per square foot

  ➢ Lateral loads may be resisted by friction acting at the base of the footings and/or by passive resistance within the recommended bearing material.

The foundation bearing values provided above are for the total of dead and frequently applied live loads and include a Factor-of-Safety of at least (3). These bearing values may be increased by a factor of (1/3) for temporary loads, such as, wind and seismic forces. The bearing values given above are net bearing values; the weight of concrete below grade may be neglected.

When combining passive earth pressure and friction for lateral resistance, the passive earth pressure component should be reduced by one-third. The coefficient of friction should be applied to dead load forces only.

All continuous footings shall be reinforced with a minimum of (4) #(5) bars, two placed near the top and two near the bottom. Reinforcing recommendations are minimums and may be revised by the project structural engineer.

All footing excavation depths will be measured from the lowest adjacent grade of recommended bearing material(s). Footing depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth into the recommended bearing materials will not be acceptable to this office.

Foundation Settlement

The terrace soils (Qt) underlying the proposed development area should be anticipated to experience settlement over the design-life of the proposed development.

Static Settlement

Static settlement of the proposed new development should be anticipated. The differential static settlement potentials of the terrace deposit soils (Qt) underlying the proposed development area are anticipated to be approximately (½ to ¾) of an inch between walls or piers, within (20) feet, or less, of each other, and under similar loading conditions. A total static settlement potential of about (1) inch should be anticipated. Static settlement potentials were determined using the recommended allowable foundation loads:

Hydroconsolidation

Hydroconsolidation is settlement of soils that collapse when they become saturated. Hydroconsolidation potential is greatest at the subject site for the upper soils due to the potential of saturation from irrigation and rainfall. The amount of hydroconsolidation settlement of the upper soils can be reduced by proper maintenance of the subject site. Plumbing lines should maintained in leak free condition, site drainage should be maintained as outlined in the Drainage and Maintenance section above, and landscape watering should be kept to a minimum to reduce infiltration of moisture to the deeper soils. Hydroconsolidation can occur in deeper soils due to elevated groundwater levels. The depth to historic groundwater is greater than (40) feet at the subject site. Based upon the depth to the historic groundwater, hydroconsolidation of the deeper soils should not pose any significant hazard at the subject site.

Expansive Soils

Potentially expansive soils were encountered at the subject property. All new foundations, slabs-on-grade, and new hardscape may be designed for a MEDIUM expansive soil condition, in accordance with §1808.6 of the 2013CBC.

Expansive soils can be a problem, as variation in moisture content will cause a volume change in the soil. Expansive soils heave when moisture is introduced and contract as they dry. During inclement weather and/or excessive landscape watering, moisture infiltrates the soil and causes the soil to heave (expansion). When drying occurs the soils will shrink (contraction).

Repeated cycles of expansion and contraction of soils can cause pavement, concrete slabs on grade and foundations to crack. This movement can also result in misalignment of doors and windows. To reduce the effect of expansive soils, foundation systems are usually deepened and/or provided with additional reinforcement design by the structural engineer.

Planning of yard improvements should take into consideration maintaining uniform moisture conditions around structures. Soils should be kept moist, but water should not be allowed to pond. These designs are intended to reduce, but will not eliminate, deflection and cracking and do not guarantee or warrant that cracking will not occur.

Site Drainage

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage should be collected and transferred to the street, or an approved drainage facility in non-erosive drainage devices. The proposed development should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within (5) feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters located within wall backfill and/or near foundations should be sealed to prevent moisture intrusion into the subgrade earth materials.

Retaining Walls

Interior basement retaining walls with a maximum height of (10) feet, may be anticipated for the proposed development. All retaining walls shall conform to the requirements of 2013 CBC §1610, including Table 1610.1.

Retaining walls may be designed as follows:

Retaining Wall Design Recommendations

Retaining Wall	Retained Material & Soil Type	Retained Height (feet)‡	Backslope Gradient (MAX.)	Design Lateral Soil Loads (psf/ft)		Restrained Design Earth Pressure (psf)*	Seismically Induced Lateral Load (psf/ft)†
				Active Pressure	At-Rest Pressure		
Basement Walls	Terrace Deposit Soils	10	LEVEL	60	76	47.5 * H	28

* - "H" is defined as the height of retained soil.
† - The seismically induced earth pressure should be applied as an <u>inverted</u> triangular pressure distribution.
‡ - Exterior walls up to (20) feet in height shall be designed as two "stacked" (10)-foot walls, structurally connected.
N/A – The specific design recommendations are not applicable (N/A) to the proposed development.

Exterior retaining walls, e.g. yard retaining walls, should be designed as cantilevered retaining walls. Cantilevered retaining walls shall be designed utilizing the active pressure lateral load with a triangular earth pressure distribution. Cantilevered retaining walls should also be allowed to deflect (0.01H) to (0.02H), where "H" is the retained height of the wall.

Interior retaining walls, e.g. basement retaining walls, should be designed as restrained retaining walls. Restrained retaining walls (walls for which horizontal movement is restricted at the top), shall be designed utilizing the at-rest pressure lateral load. Also, restrained walls shall be designed using a trapezoidal pressure distribution with the restrained design earth pressure.

All retaining walls, planned to be greater than (6) feet in height, should be designed to withstand a seismically induced lateral load. The seismically induced lateral load may be assumed to act as a fluid with the equivalent fluid weight indicated above, and should utilize an inverted triangular earth pressure distribution.

In addition to the lateral loads given above, all retaining walls should be designed to resist the surcharge loading imposed by any existing/proposed structures and/or by adjacent traffic loads. Wall surcharge loads may be determined utilizing the attached figures 11 and 12 obtained from the Naval Facilities Engineering Command, Design Manual 7.2 (1986).

Retaining wall foundations may be designed per the recommendations provided above, in the Foundation Design section of this report. The retaining wall design lateral loads are provided with the assumption that the wall has been outfitted with a permanent drainage system; and has been backfilled as outlined in the Retaining Wall Backfilling section in Appendix IV – General Recommendations.

Retaining walls should be provided with a permanent drainage system to eliminate the build-up of excess hydrostatic pressure behind the wall. Alternatively, retaining walls may be designed to withstand a hydrostatic loading surcharge in addition to the lateral loads given above. The hydrostatic loading surcharge shall be taken as an equivalent fluid weight of (62.4) pounds per square foot, per foot of wall height. The hydrostatic loading surcharge shall be applied to the full height of the retaining wall.

All retaining walls that are not designed to withstand hydrostatic loading should be provided with a subdrain or weepholes covered with a minimum of (12) inches of (¾) inch crushed gravel. Basement, or partial basement, retaining walls should be provided with a subdrain system which either includes a sump pump, or outlets to an area drain with sump pump. The retaining wall subdrainage system may outlet to an area drain to remove drainage to the street, or another appropriate location approved by the project civil engineer and the reviewing agency.

All required retaining wall backfill should be compacted to at least (90) percent of the Proctor maximum dry density, per the latest edition of ASTM D 1557. Proper compaction of the backfill is recommended to provide lateral support to adjacent properties, structures, and improvements.

Even with proper compaction of the required backfill, settlement of the backfill may occur because of the significant depth of the backfill. Accordingly, utility lines, footings, or falsework should be planned and designed to accommodate such potential settlements. All grading and drainage requirements listed in the Retaining Wall Backfilling section of Appendix IV – General Recommendations shall apply.

Water and moisture affecting retaining walls is a common post-construction complaint. Poorly applied or omitted waterproofing can lead to standing water inside proposed structures or efflorescence on the wall.

We recommend that all retaining walls be waterproofed. Waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer. Bay City Geology, Inc. does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary. The actual waterproofing design shall be provided by the architect, structural engineer or contractor with experience in waterproofing.

Temporary Excavations

Excavations associated with the recommended site grading, and for construction of the new proposed basement retaining walls should be anticipated for the proposed development. Grading over-excavations are anticipated to be approximately (5+/-) feet in depth below existing grade. Also, excavations for the proposed new basement level are anticipated to be up to (12) feet in depth below existing grade. Conventional excavation equipment may be used to make these excavations. Excavations should be anticipated to expose competent terrace deposit soils (Qt). All temporary excavations should be observed and verified by the project soils engineer and/or project engineering geologist during construction so that modifications can be made if variations in the earth materials occur

All excavations should be stabilized within (30) days of initial excavation. If this time is exceeded, the project soils engineer must be notified, and modifications, such as shoring or slope trimming may be required. Water should not be allowed to pond on top of, or at the toe of, the excavations, nor to flow toward them. All excavations should be protected from inclement weather. Excavations should be kept moist, not saturated, to reduce the potential for raveling and sloughing during construction. No vehicular surcharge should be allowed within (3) feet of any excavation.

Grading Over-excavations

Over-excavations necessary to construct the new engineered, compacted fill material (CAf) building pad are anticipated to be (5) feet in height and located within (5) feet of the property boundaries. Grading over-excavations are anticipated to remove lateral support from adjacent structures, properties, and/or public rights-of-way, and should be performed utilizing the A-B-C Slot Cutting Method. This method employs the use of the earth as a buttress and allows the excavation to proceed in phases. The initial excavation is made at a slope of 1H:1V (horizontal to vertical).

Slots are cut, using the A-B-C method, in which all slots are of the same width. The initial slot "A" is cut (8) feet in width, leaving the "B" and "C" slots to buttress the excavation. The "A" slot is then backfilled with the engineered, compacted fill materials. The same procedure is used for the "B" slots; then the "C" slots.

Basement Excavations
Compound excavations are anticipated to be necessary to construct the proposed new basement level. The planned temporary compound excavations are anticipated to be a total of (12) feet in depth and located within (12) feet of the adjacent property boundaries. The planned compound basement excavations should consist of a (5)-foot vertical at the toe of the cut with a 1H:1V (horizontal to vertical) gradient slope above the vertical for a height of (7) feet.

The proposed new basement excavations should be anticipated to remove lateral support from adjacent structures, properties, and/or public rights-of-way. The enclosed temporary excavation calculations demonstrate that the terrace deposit soils (Qt) are suitable for vertical excavations up to (5) feet in height, with a 1H:1V (horizontal to vertical) slope above.

<u>Slabs-on-Grade & Hardscape</u>
All conventional slabs-on-grade and hardscape shall be cast over competent terrace deposit soils (Qt), or properly placed, competent engineered, compacted fill materials. Grading and earthwork for subgrade preparation and slab support should include the following:

- Any existing loose and/or spilled fill and loose native soils within the footprint of any proposed slab areas should be over-excavated and removed to expose competent <u>terrace deposit soils (Qt)</u>. The minimum over-excavation for placement of new engineered, compacted fill materials shall be (24) inches.

- The bottom of the over-excavation should be observed by the project soils engineer and/or engineering geologist.

- The bottom of the over-excavation should be scarified about (6) inches, moisture conditioned, and compacted.

- Engineered fill should be placed in loose lifts of about (4) to (6) inches in thickness and compacted.

- Engineered fill should be moisture controlled to be within (3) percent of the optimum moisture content.

- Engineered fill should be compacted to a minimum of (90) percent of the Modified Proctor Maximum Dry Density, per the latest edition of ASTM D 1557.

- Engineered fill should be tested for compaction with a minimum frequency of at every (2) vertical feet or (500) cubic yards of fill placed, whichever is MORE restrictive.

- Engineered fill should be surface tested for compaction at the proposed subgrade elevation.

- The project soils engineer and/or engineering geologist should be contacted to provide periodic observation of the grading operation and perform compaction testing of the engineered compacted fill placed.

- The project soils engineer and/or engineering geologist should prepare a final report detailing the grading and earthwork performed, placement and testing of the engineered compacted fill, and providing the as-built condition of the project area with respect to engineering geology.

Foundation excavation spoils should either be removed from the slab areas or compacted into place by mechanical means and tested for compaction.

Slabs-on-grade should be reinforced with a minimum of (#5) reinforcing bars, placed at (12) inches on center each way. For interior slabs and/or slabs where moisture control is required, a vapor retarder with a minimum thickness of 15-mil should placed below the concrete slab. The vapor retarder should conform to ASTM E1745 Class A with water vapor transmission rate <0.01 perms and should be installed in accordance with ASTM E1643. The structural engineer should provide design considerations such as reinforcement to offset potential increase in curling stresses in the slab.

Slabs, walkways, and decking are likely to crack as a result of shrinkage and curing processes of concrete. Typical concrete shrinkage can result in cracks and gaps along control joints and where slabs connect with structures. Slabs should be provided with proper control joints in an effort to control the location of the cracking. The gaps will require periodic caulking to limit infiltration of moisture.

Provisions for cracks should be incorporated into the design and construction of the foundation system, slabs and proposed floor coverings. Concrete slabs should have sufficient control joints spaced at a maximum of approximately (8) feet. Slabs-on-grade should be quartered or saw cut slabs to mitigate cracking and be isolated from the stem wall footing. Exterior slabs planned adjacent to descending slopes or planter areas should be provided with a thickened edge. The thickened edge should be a minimum of (12) inches wide and (24) inches deep and two #(4) bars.

Movement of slabs adjacent to structures can be mitigated by doweling slabs to perimeter footings. Doweling should consist of (#4) bars bent around exterior footing reinforcement. Dowels should be extended at least (2) feet into planned exterior slabs. Doweling should be spaced consistent with the reinforcement schedule for the slab. With doweling, (⅜) inch minimum thickness expansion joint material should be provided. Where expansion joint material is provided, it should be held down about (⅜) inch below the surface. The expansion joints should be finished with a color matched, flowing, flexible sealer (e.g., pool deck compound) sanded to add mortar-like texture. As an option to doweling, an architectural separation could be provided between the main structures and abutting appurtenant improvements.

These recommendations are considered as minimums unless superseded by the project structural engineer.

The on-site earth materials exhibit a MEDIUM expansion potential. Thus in accordance with 2013 CBC §1808.6.4, prior to pouring interior (living area) slabs, the slab sub-grade earth materials should be pre-saturated to a minimum moisture content of (130) percent of the optimum moisture content, per the latest edition of ASTM D 1557. Pre-saturation of the slab sub-grade earth materials shall extend to a minimum depth of (24) inches below grade.

For exterior areas, new hardscape, (e.g.: walkway areas, pool areas, and driveway areas), may consist of flexible paving, including: A/C pavement and/or flexible and permeable paving stones. New flexible paving may be cast over newly placed engineered, compacted fill materials. New flexible paving should be designed for an expansive soil condition. Grading and earthwork, as outlined above and with a (24) inch over-excavation, should be utilized for preparation of subgrade soils for support of new flexible paving.

Additionally, the property owner should be aware that removal of all existing fill materials and/or native soil materials in the area of new flexible paving is not required. However, pavement constructed over native soil/existing fill materials will most likely have a shorter design life and increased maintenance costs Also, if necessary, a 'Request for Modification' of the Building Code to allow placement of new engineered, compacted fill over competent native soil and/or existing fill materials should be submitted along with this geotechnical report.

Concrete Mix Design

Our experience indicates that the earth materials at the site contain negligible levels of sulfates. As such, a concrete mix design including: Type II Portland cement is recommended for the project. Also, we recommend that a low permeability concrete be utilized at the site to limit moisture transmission through slabs and foundations. For this purpose, the water/cement ratio to be used at the site should be limited to between (0.45) and (0.50). Limited use (subject to approval of mix designs) of a water reducing agent may be included to increase workability. The concrete should be properly cured to minimize risk of shrinkage cracking. The code dictates at least (7) days of moist curing. Two to three weeks is preferred to minimize cracking. One-inch hard rock mixes should be provided. Pea gravel mixes are specifically not recommended but could be utilized for relatively non-critical improvements (e.g., flatwork) and other improvements provided the mix designs consider limiting shrinkage.

Contractors/other designers should take care in all aspects of designing mixes, detailing, placing, finishing, and curing concrete. The mix designers and contractor are advised to consider all available steps to reduce cracking. The use of shrinkage compensating cement or fiber reinforcing should be considered. Mix designs proposed by the contractor should be considered subject to review by the project engineer.

REVIEWS

Plan Review and Plan Notes

The final grading, building, and/or structural plans shall be reviewed and approved by the consultants to ensure that all recommendations are incorporated into the design or shown as notes on the plan.

The final plans should reflect the following:

1. This Soils Engineering Investigation by Bay City Geology, Inc. is a part of the plans.

2. Plans must be reviewed and signed by the soils engineer and geologist.

3. The project soils engineer and/or geologist must review all grading.

4. The project soils engineer and/or geologist shall review all foundation excavations prior to placing steel and concrete.

Construction Review

Reviews will be required to verify all geologic and geotechnical work. It is required that all footing excavations, seepage pits, and grading be reviewed by this office prior to placing steel and concrete. This office should be notified at least **two working days** in advance of any field reviews so that staff personnel may be made available.

The property owner should take an active role in project safety by assigning responsibility and authority to individuals qualified in appropriate construction safety principles and practices. Generally, site safety should be assigned to the general contractor or construction manager that is in control of the site and has the required expertise, which includes but not limited to construction means, methods and safety precautions.

LIMITATIONS

General

Findings, conclusions and recommendations contained in this report are based upon the surface mapping, subsurface exploration, data analyses, and specific information as described and past experience. Earth materials and conditions immediately adjacent to, or beneath those observed may have different characteristics, such as, earth type, physical properties and strength. Therefore, no representations are made as to the nature, quality, or extent of latent earth materials. Site conditions can and do change from those that were first envisioned. During construction, if subsurface conditions differ from those encountered in the described exploration, this office should be advised immediately so that appropriate action can be taken.

Findings, conclusions and recommendations presented herein are based on experience and background. Therefore, findings, conclusions and recommendations are professional opinions and are not meant to indicate a control of nature.

Potentially expansive soils were encountered on the subject property. Design for foundations, slabs on grade, and retaining walls have been provided to mitigate this soil condition. These designs do not guarantee or warrant that cracking will not occur.

This limited report provides information regarding the geologic findings on the subject property. It is not designed to provide a guarantee that the site will be free of hazards in the future, such as, landslides, slippage, differential settlement, debris flows, seepage, concentrated drainage or flooding. Hillside and flatland properties are subject to different hazards. It may not be possible to eliminate all hazards, but homeowners must maintain their property and improve deficiencies.

CONSTRUCTION NOTICE

Construction can be difficult. Recommendations contained herein are based upon surface reconnaissance and subsurface explorations deemed suitable by your consultants.

It is this Corporation's aim to advise you through this report of the general site conditions, suitability for construction, and overall stability. It must be understood that the opinions are based upon testing, analysis, and interpretation thereof.

Quantities for foundation concrete and steel may be estimated, based on the findings given in this report. However, you must be aware that depths and magnitudes will most likely vary between the explorations given in the report.

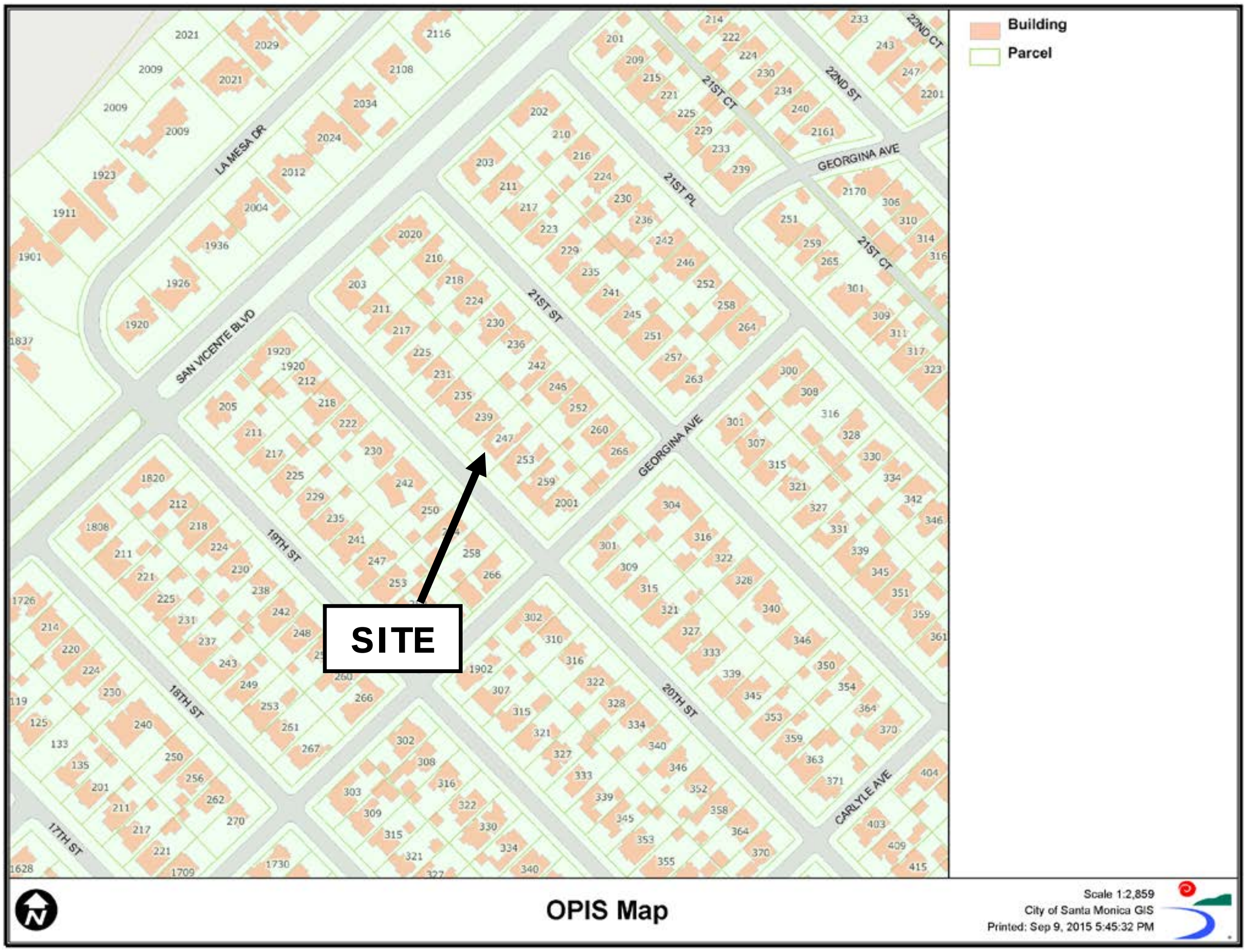
APPENDIX I

Location Maps

Plot Map

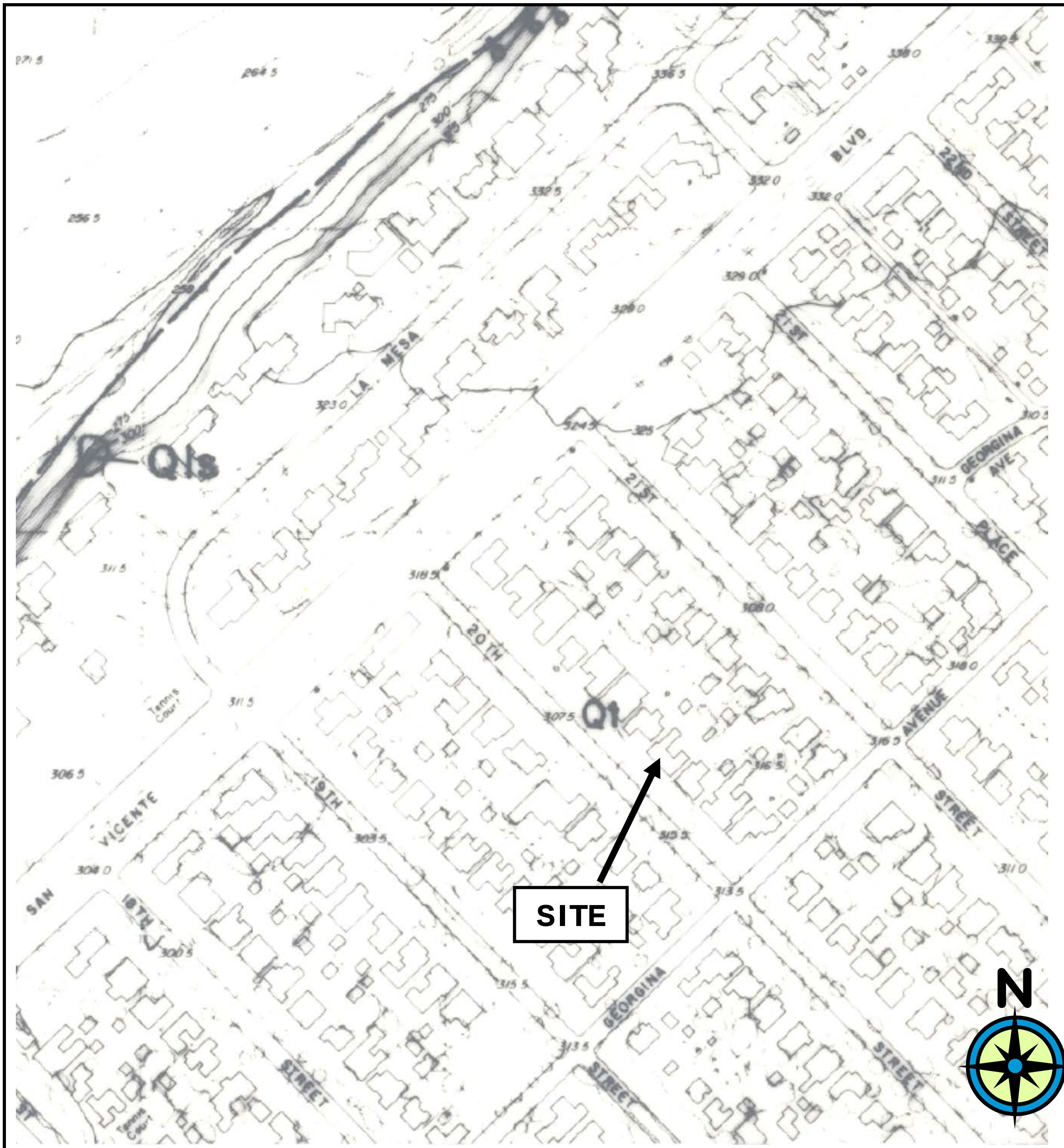
Field Exploration
Exploration Logs 1 and 2

LOCATION



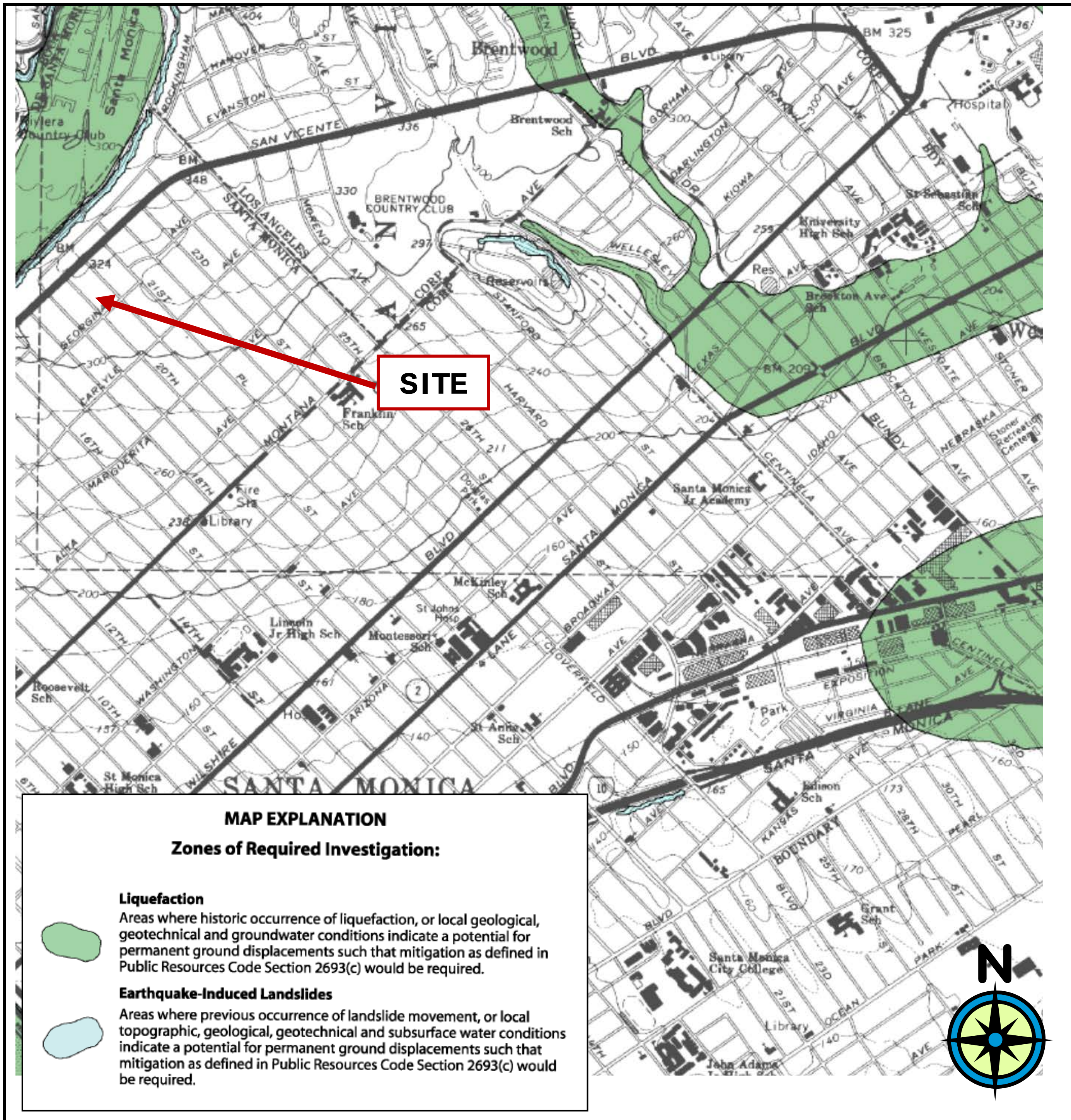
Reference:	Santa Monica GIS	Scale: as shown
Project Address:	247 20th Street Santa Monica, California	Plate 1

REGIONAL GEOLOGY MAP



Reference:	City of Los Angeles Geologic Maps No. 303	Approximate Scale: 1”=500’
Project Address:	247 20th Street Santa Monica, California	Plate 2

SEISMIC HAZARD MAP



Reference:	State of California Seismic Hazard Map of the Beverly Hills Quadrangle	1”=1600’
Project Address:	247 20th Street Santa Monica, California	Plate 3

HISTORIC HIGH GROUNDWATER

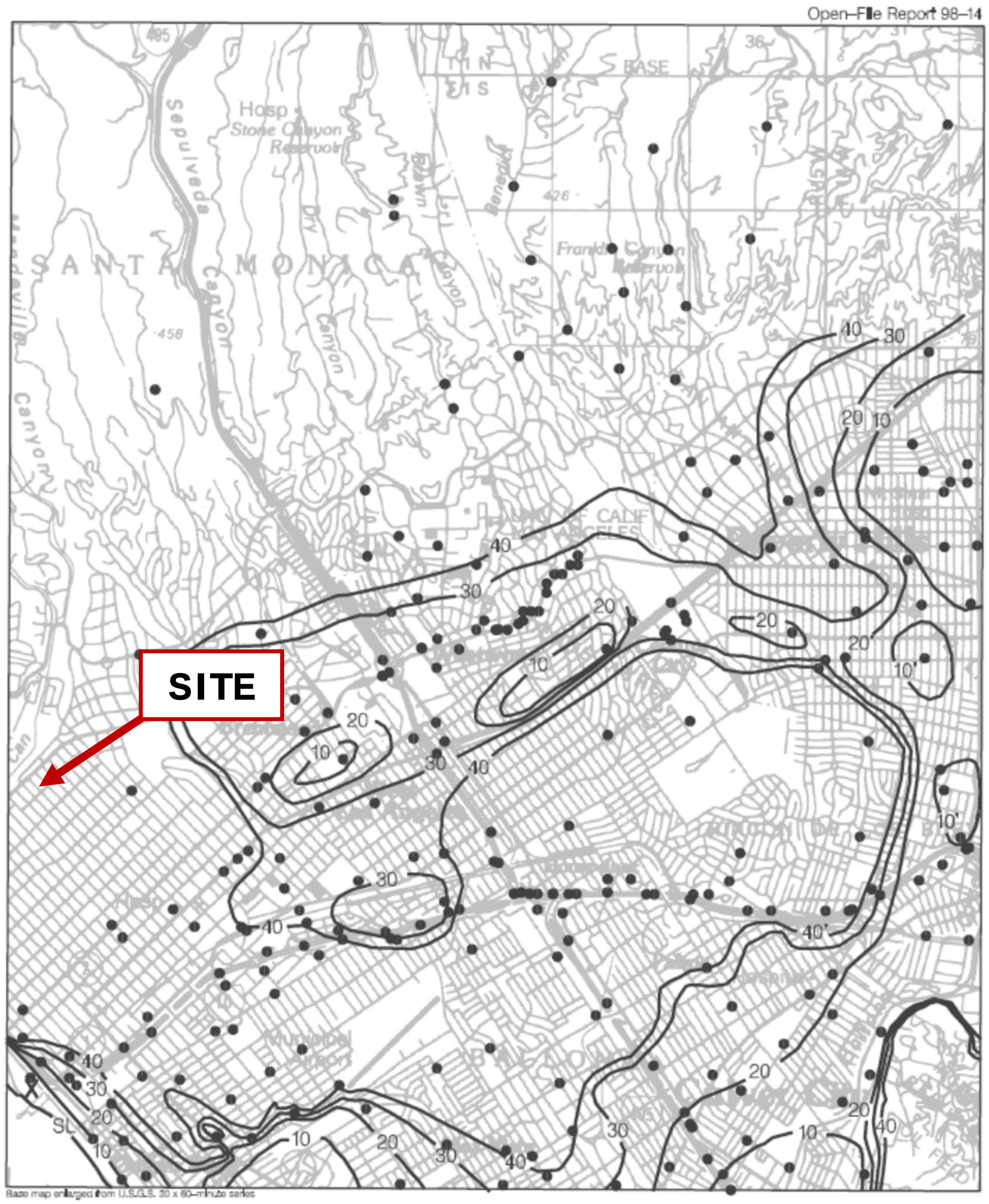
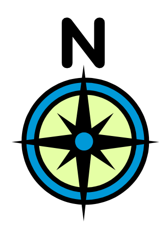


Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Beverly Hills Quadrangle.

● Borehole Site — 30 — Depth to ground water in feet

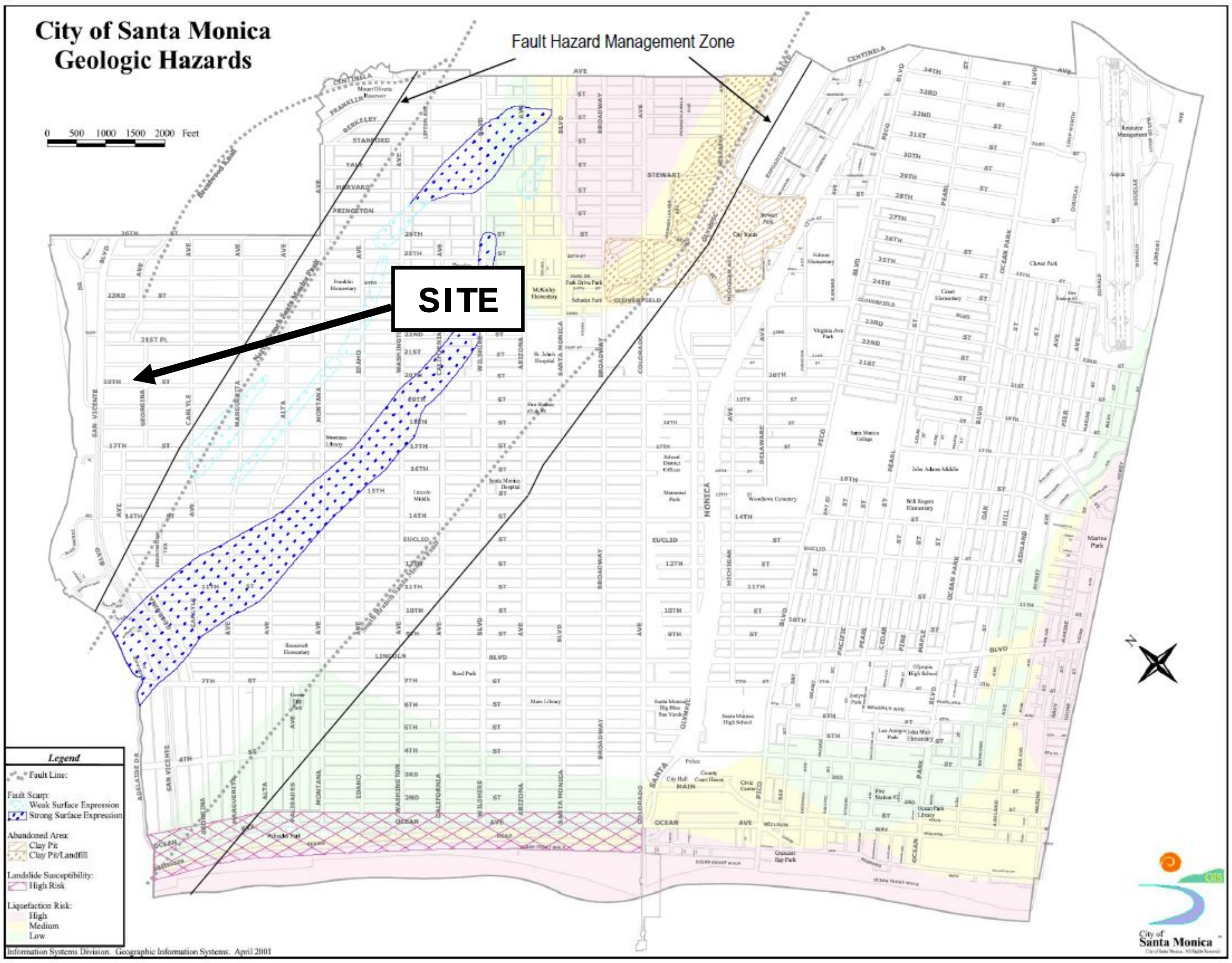
X Site of historical earthquake–generated liquefaction. See "Areas of Past Liquefaction" discussion in text.

ONE MILE
SCALE



Reference:	State of California Seismic Hazard Report of the Beverly Hills Quadrangle
Project Address:	247 20th Street Santa Monica, California — Plate 4

SANTA MONICA GEOLOGIC HAZARDS



1. A discussion of fault rupture hazards is required for all projects located within the Fault Hazard Management Zone. (Section 3.3.1). The Fault Hazard Management Zone extends 380 to nearly 500 feet north of the north branch and 100 to nearly 600 feet south of the south branch of the Santa Monica fault.
2. The above map is taken from the "*Technical Background Report to the Safety Element of the City of Santa Monica General Plan*", by Leighton & Associates, Inc., March 30, 1994.
3. The more recent state maps depicting the Hazard Zones associated with liquefaction and landslides supercede those shown above and are used in the application of these geotechnical guidelines.

Reference:	Guidelines for Geotechnical Reports, City of Santa Monica, Department of Building and Safety	Scale: As Shown
Project Address:	247 20th Street Santa Monica, California	Plate 5

SANTA MONICA FAULT MAP

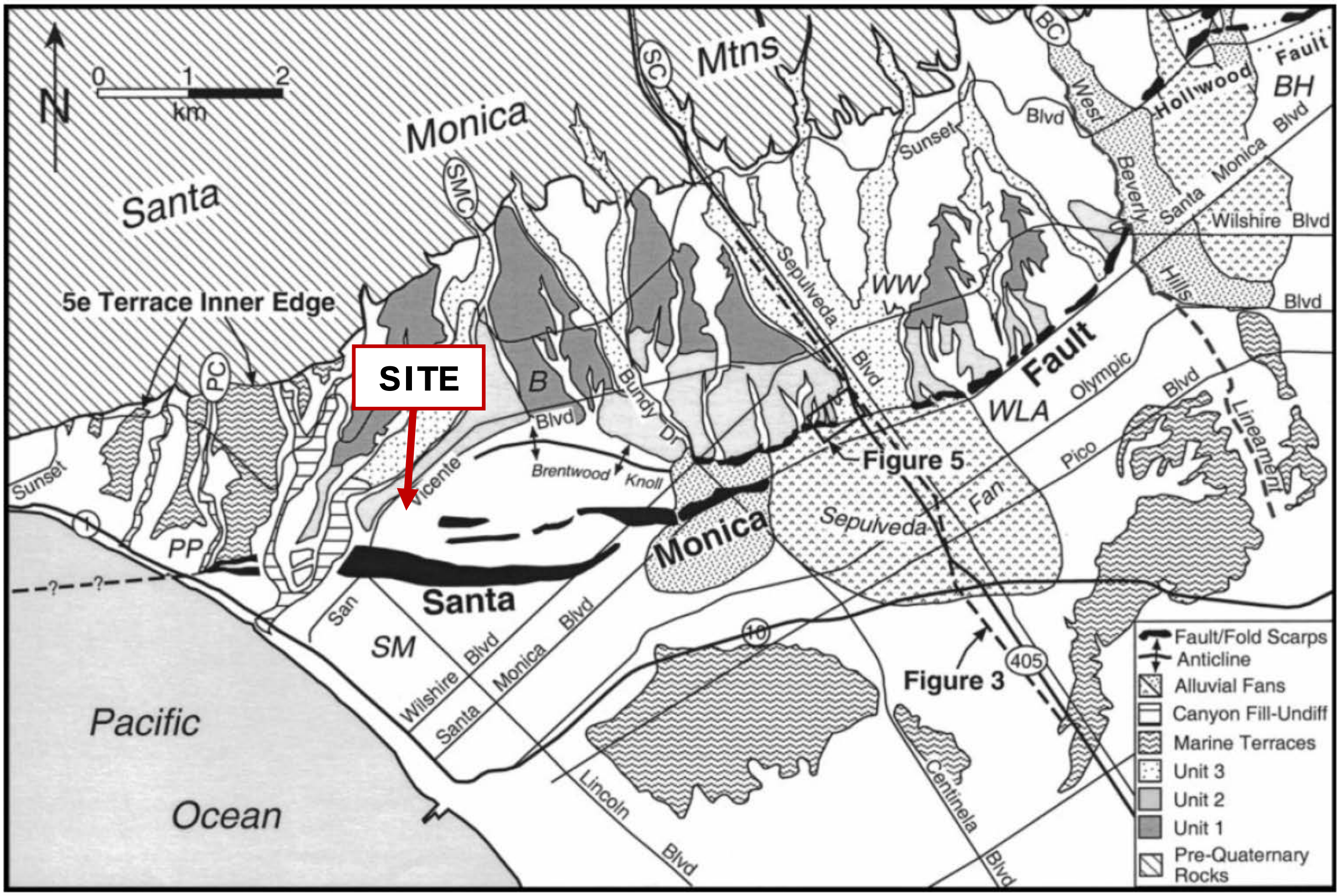
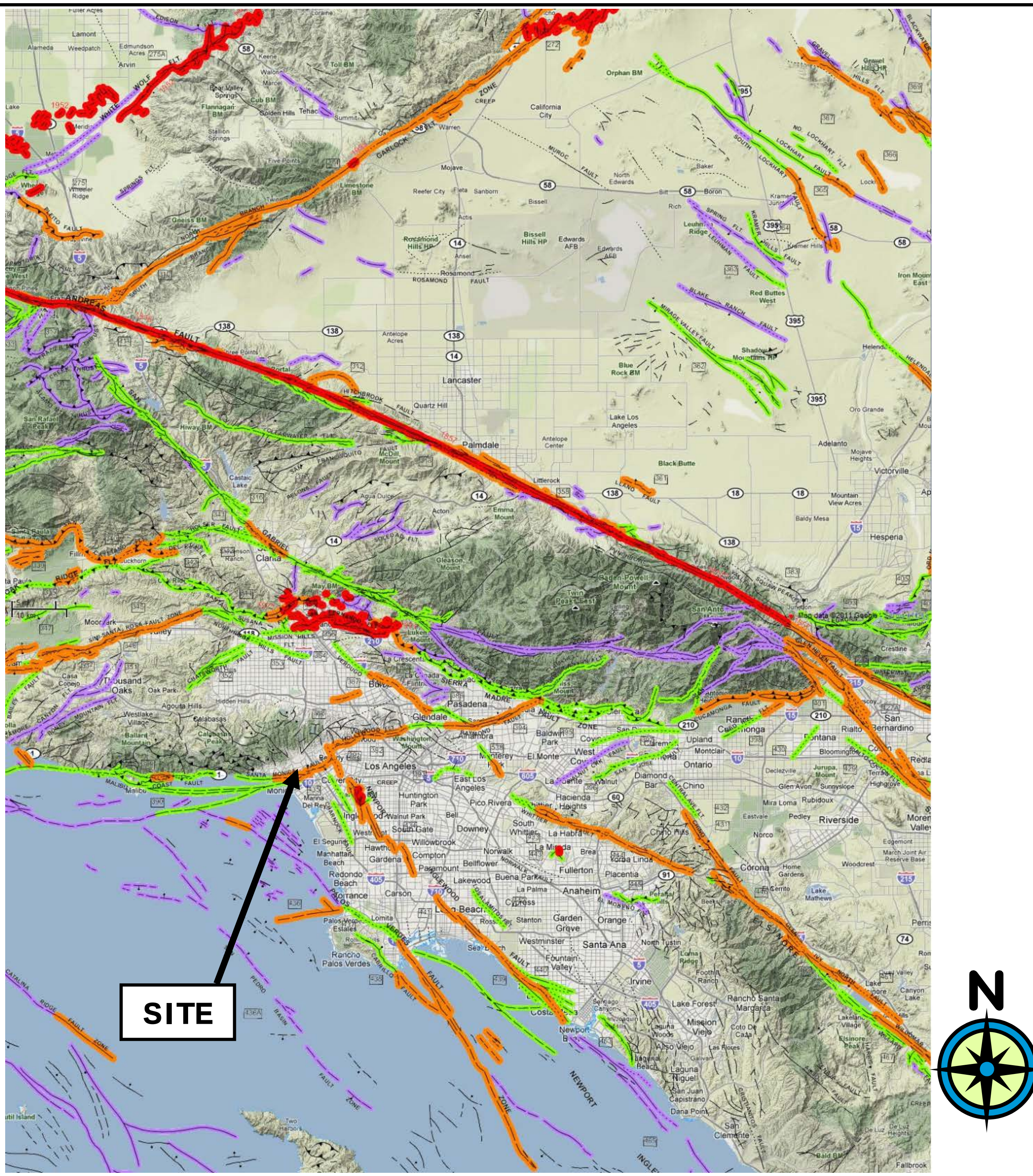


Figure 2. Tectonic geomorphologic map of the Santa Monica fault zone and environs based on interpretation of 1926 vintage U.S. Geological Survey 6 ft topographic maps (Sawtelle, Topanga Canyon, and Hollywood quadrangles) and field mapping. Note location of trench site just west of Freeway I-405. B—Brentwood; BH—Beverly Hills; PC—Potrero Canyon; PP—Pacific Palisades; SM—Santa Monica; WLA—west Los Angeles; WW—Westwood.

Reference:	Geological Society of America Bulletin, (Dolan, 2000)	Scale: 1” = 2 km
Project Address:	247 20th Street Santa Monica, California	Plate 6

REGIONAL FAULT MAP



Reference:	A Portion of the California Geologic Survey Fault Activity Map of California (2010) 1"=30km
Address:	247 20th Street Santa Monica, California Plate 7

FAULT ACTIVITY MAP OF CALIFORNIA EXPLANATION

EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

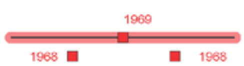
(c) displaced survey lines.

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along fault break.

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.



Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

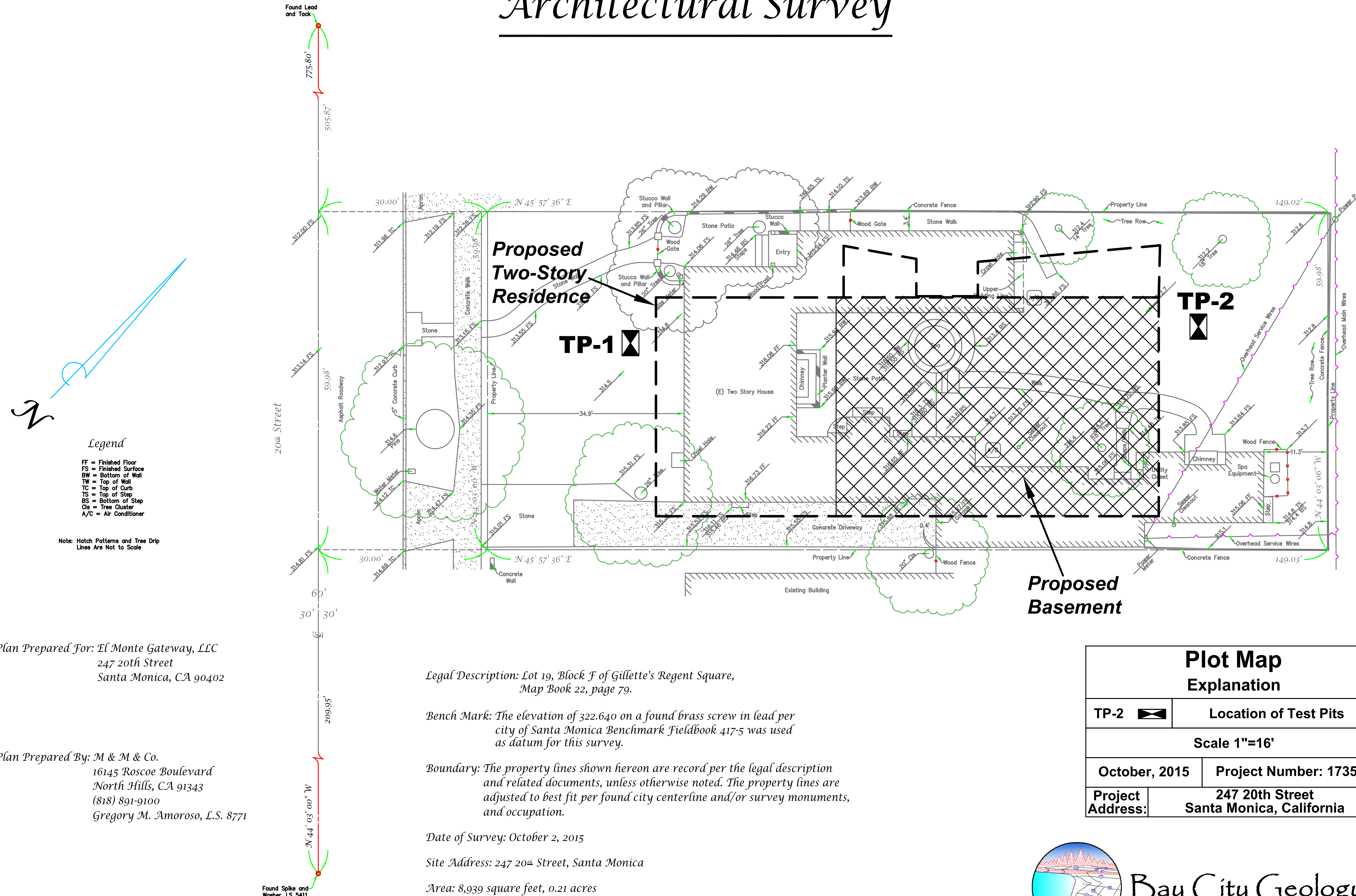
Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

Architectural Survey



Plan Prepared For: El Monte Gateway, LLC
247 20th Street
Santa Monica, CA 90402

Plan Prepared By: M & M & Co.
16145 Roscoe Boulevard
North Hills, CA 91343
(818) 891-9100
Gregory M. Amoroso, L.S. 8771

Legal Description: Lot 19, Block F of Gillette's Regent Square, Map Book 22, page 79.

Bench Mark: The elevation of 322.640 on a found brass screw in lead per city of Santa Monica Benchmark Fieldbook 417-5 was used as datum for this survey.

Boundary: The property lines shown hereon are record per the legal description and related documents, unless otherwise noted. The property lines are adjusted to best fit per found city centerline and/or survey monuments, and occupation.

Date of Survey: October 2, 2015

Site Address: 247 20th Street, Santa Monica

Area: 8,939 square feet, 0.21 acres

Plot Map Explanation	
TP-2	Location of Test Pits
Scale 1"=16'	
October, 2015	Project Number: 1735-1
Project Address:	247 20th Street Santa Monica, California

Bay City Geology, Inc.

Field Exploration

A field exploration of the site was conducted in September 2017. The soils and geologic conditions were mapped by a representative of this office (refer to Plot Map & Exploration Logs). Subsurface exploration was performed by manually trenching into the underlying earth materials. Explorations were excavated to a maximum depth of (20) feet. The Plot Map in Appendix I depicts locations of the subsurface explorations. The explorations were logged by the engineering geologist using both visual and tactile methods.

Representative samples of the on-site earth materials were obtained from the explorations. Hand samples taken from test pits and/or hand-auger explorations were obtained using a (6) inch long, brass ring lined, steel barrel hand-sampler; driven with a slide-safety hammer. The soil is retained in the brass rings of (2½) inches in diameter and (1) inch in height. The samples are transported in moisture tight containers. Additional bedrock sample locations are indicated on the Exploration Logs.

Project Address: *247 20th Street*
Date Logged: *09/14/15*
Project Number: *1735*
Logged By: *J. Miller*

EXPLORATION: TP-1

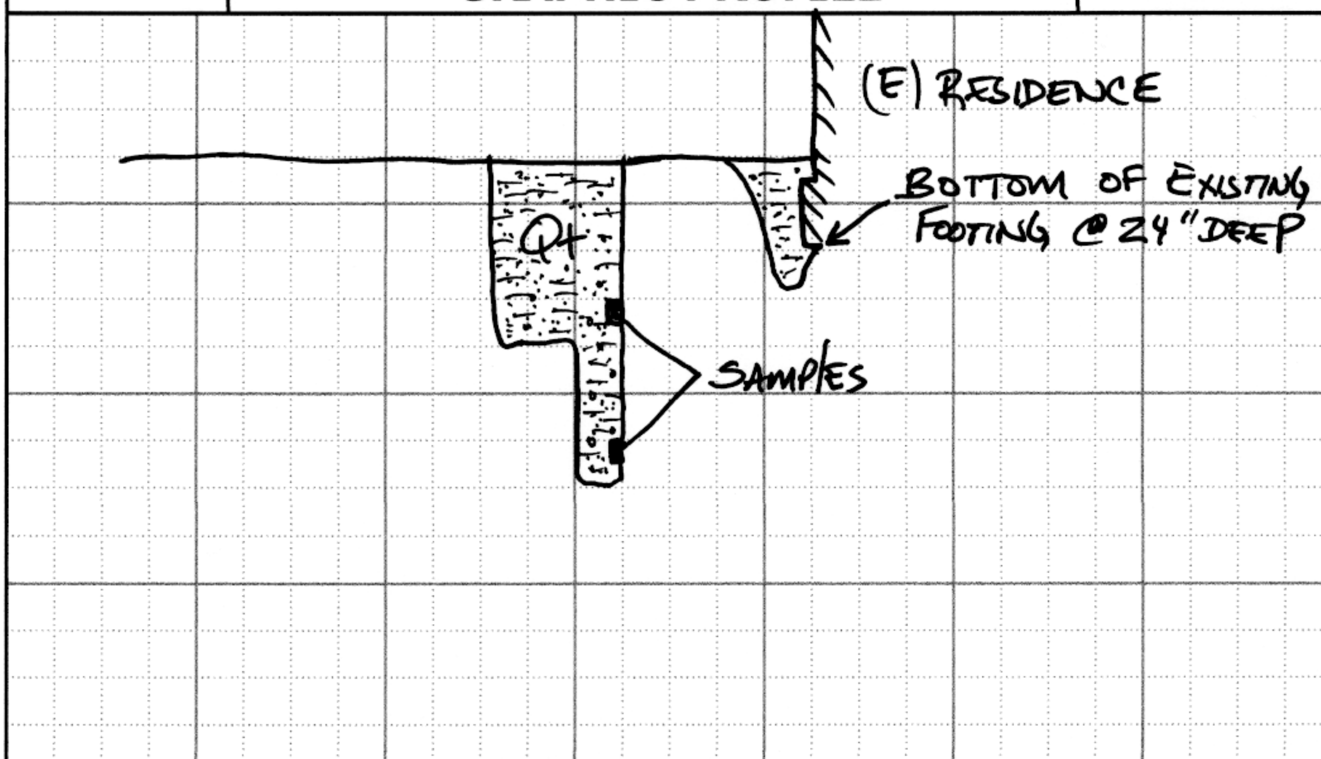
DESCRIPTION

0.0 - 7.0' TERRACE DEPOSITS; Qt, sandy clay, dark-brown, most, firm, rock clasts up to 1-inch in length.

@3.0' clayey sand, reddish-brown, slightly moist, dense, rock clasts up to 2-inches in length.

GRAPHIC PROFILE

Scale: 1"=4'



Project Address: *247 20th Street*

Date Logged: *09/14/15*

Project Number: *1735*

Logged By: *J. Miller*

EXPLORATION: TP-2

DESCRIPTION

0.0 - 20.0' TERRACE DEPOSITS; Qt, sandy clay, dark-brown, most, firm, rock clasts up to 1-inch in length.

@3.0' clayey sand, reddish-brown, slightly moist, dense, rock clasts up to 2-inches in length.

@6.0' abundant slate clasts, very dense.

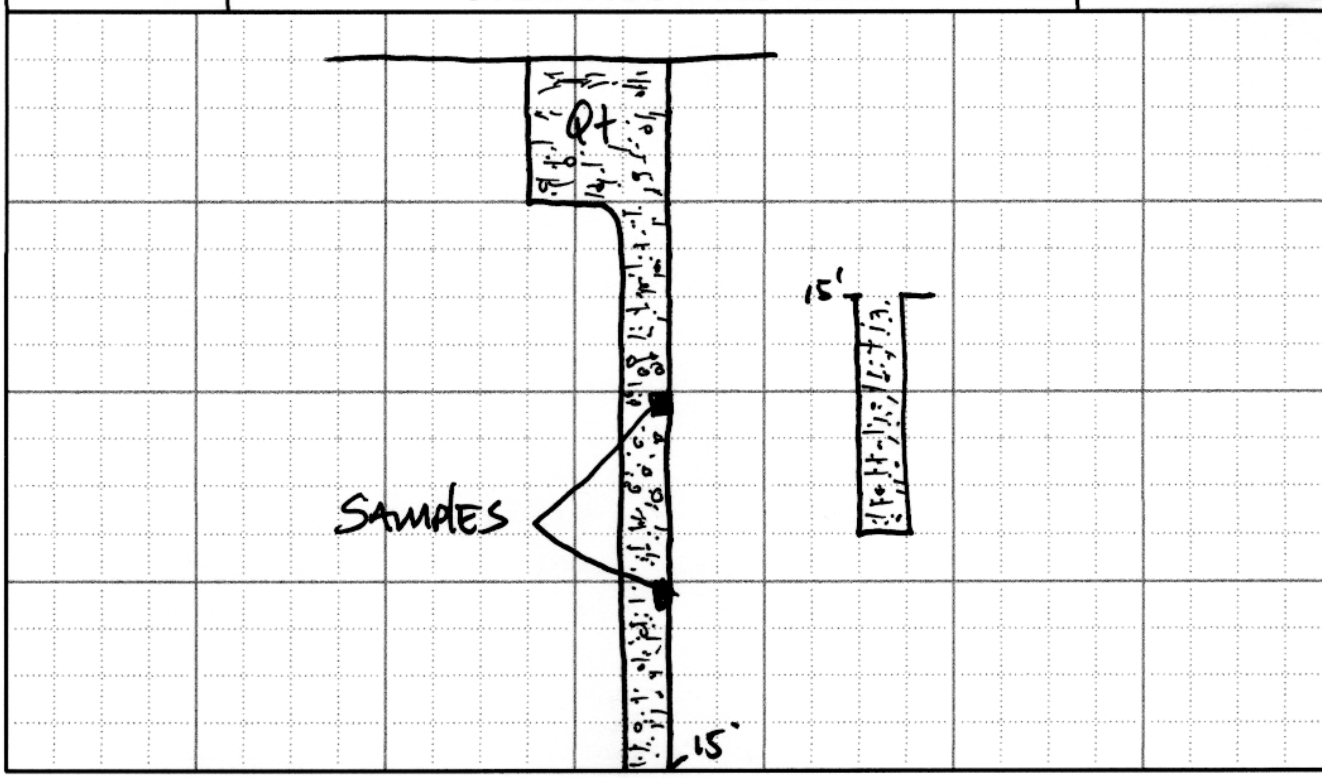
@8.0' silty sand, decrease in slate clasts.

@13.0' increase in slate clasts, sandy silt to silty sand.

(No recovery below 13.0' feet)

GRAPHIC PROFILE

Scale: 1"=4'



APPENDIX II - LABORATORY TESTING

Laboratory testing was performed on representative samples obtained during our field exploration. Samples were tested for the purpose of estimating material properties for use in subsequent engineering evaluations. Testing included in-place moisture and density, shear strength testing, consolidation testing, and soil chemical testing.

Laboratory testing was performed on samples obtained as outlined in Appendix I. All samples were sent to the laboratory for examination, testing, and classification using the Unified Soil Classification System (USCS). All soils laboratory testing performed complies with current ASTM standards.

Moisture and Density Tests

The dry unit weight and moisture content of the undisturbed samples were determined in accordance with ASTM D 2216 and ASTM D 2937. Classification of the obtained soil samples were performed utilizing the USCS in accordance with ASTM D 2487 and ASTM D 2488.

Shear Tests

Direct single-shear tests were performed in accordance with ASTM D 3080 procedures using a strain-controlled direct shear machine. The desired normal load is applied to the specimen and allowed to come to equilibrium. The rate of deflection on the sample is approximately (0.005) inches per minute. The samples are tested at higher and/or lower normal loads in order to determine the angle of internal friction and the cohesion.

Consolidation

Consolidation tests in accordance with ASTM D 2435 procedures were performed on samples contained within the one inch brass rings of the Modified California ring sampler. Consolidation tests are used to predict the soil settlement behavior under a specific loading progression. Porous stones are placed in contact with top and bottom of the samples to permit to allow the addition or release of water. Loads are applied in several increments and the results are recorded at selected time intervals. Samples are tested at field and increased moisture content. The results are plotted on the Consolidation Test Curve and the load at which the water is added as noted on the drawing.

Expansion Index Tests

Expansion characteristics of the soil were determined in accordance with ASTM D 4829 procedures. The sample is remolded and compacted into an expansion mold with a degree of saturation between 40-60%. A vertical confining pressure of (144) pounds per square foot is applied to the sample. The sample is inundated with distilled water. The deformation is recorded after (24) hours.

Sample: TP-1, Bulk Terrace Deposit Soils (SC) **EI_{50}** = 75 MEDIUM

Summary of Corrosion Testing Results

Sample: TP-1 @ 3ft., Bulk Terrace Deposit Soils (SC)

CALTRANS 643 Soil pH	CALTRANS 422 Chloride Content (ppm)	CALTRANS 417 Sulfate Content (wt %)	CALTRANS 643 Minimum Resistivity (Ohm-cm)
6.25	200	0.001	2,600

UNIT WEIGHT & WATER CONTENT OF UNDISTURBED SAMPLES

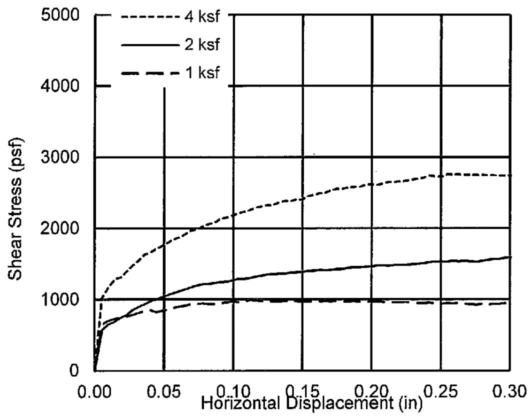
JOB NUMBER:	*1735*	***DATE:***	***10/2/2015***
PROJECT:	*BCG-20th St*	***SAMPLED BY:***	***Jonathan***
TESTED BY:	*RT*	***RING TYPE* (1 OR 2):***	***1***

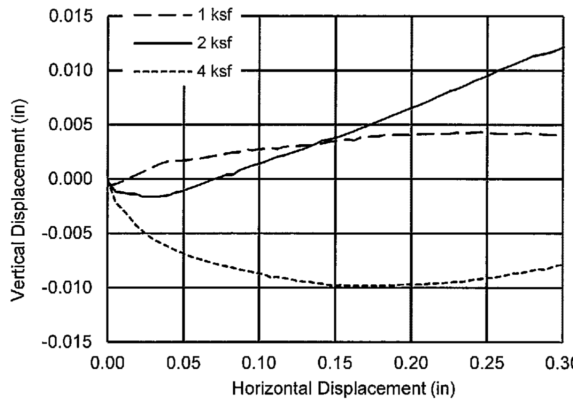
* RING TYPE 1: SMALL 2.42" ϕ , 45 GRAMS
RING TYPE 2: LARGE 2.62" ϕ , 75 GRAMS

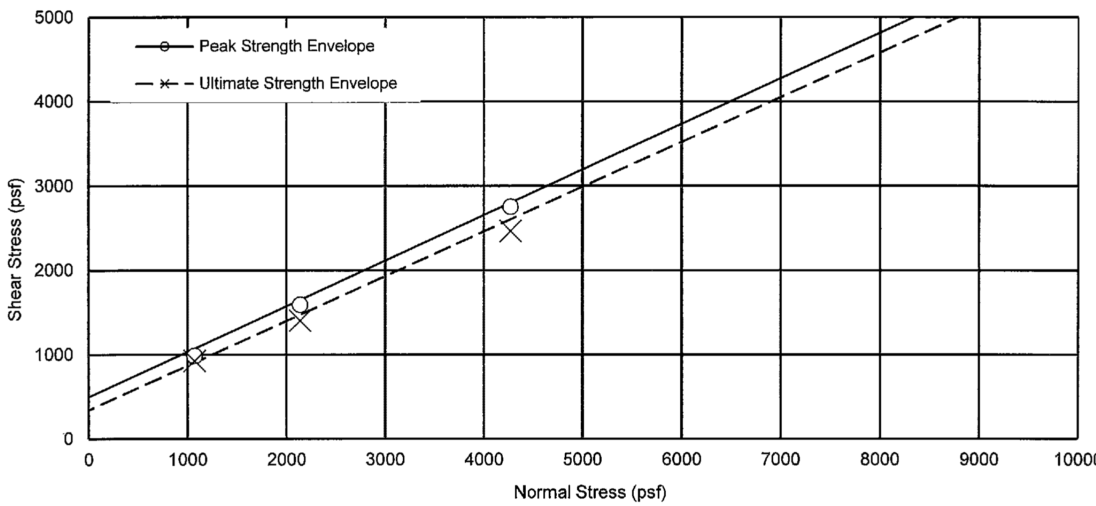
UNIT WEIGHT DATA (ALL WEIGHTS IN GRAMS)

BORING / TEST PIT NO.	***TP1***			***TP2***				
SAMPLE DEPTH (FT)	***3***	***6***		***7***	***11***	***15***	***20***	
NUMBER OF RINGS	***5***	***4***		***5***	***4***	***4***	***6***	
WEIGHT OF RINGS & WET SAMPLE	***1005.80***	***807.90***		***890.20***	***795.00***	***770.40***	***1106.50***	
WEIGHT OF 1-RING & WET SAMPLE	201.16	201.98		178.04	198.75	192.60	184.42	
WEIGHT OF 1-RING	45.00	45.00		45.00	45.00	45.00	45.00	
WEIGHT OF WET SOIL	156.16	156.98		133.04	153.75	147.60	139.42	
TARE NO.								
TARE WEIGHT	***46.70***	***38.30***		***32.50***	***46.30***	***45.20***	***44.20***	
WET WEIGHT SOIL & TARE	***235.90***	***230.80***		***162.10***	***238.30***	***237.60***	***192.50***	
DRY WEIGHT SOIL & TARE	***218.30***	***212.00***		***151.10***	***226.40***	***232.20***	***178.80***	
WEIGHT OF WATER	17.60	18.80		11.00	11.90	5.40	13.70	
DRY WEIGHT SOIL	171.60	173.70		118.60	180.10	187.00	134.60	
BORING / TEST PIT NO.	TP1			TP2				
SAMPLE DEPTH (FT)	3	6		7	11	15	20	
WET DENSITY (PCF)	129.3	130.0		110.2	127.3	122.2	115.5	
WATER CONTENT (%)	10.3	10.8		9.3	6.6	2.9	10.2	
DRY DENSITY (PCF)	117.3	117.3		100.8	119.4	118.8	104.8	
REMARKS								

	Sample ID :	1 ksf	2 ksf	4 ksf			
Initial	Water Content (%)	10.3	10.3	10.3			
	Dry Density (%)	116.7	116.2	115.8			
	Saturation (%)	62.7	61.8	61.1			
Final	Water Content (%)	18.0	18.2	18.0			
	Dry Density (pcf)	115.7	114.9	114.4			
	Saturation (%)	106.5	105.4	102.8			
Normal Stress (psf)		1067	2134	4269			
Peak Shear Stress (psf)		986	1591	2754			
Ultimate Shear Stress (psf)		925	1403	2466			







Peak Cohesion, c' (psf):	498	Ultimate Cohesion, c (psf):	342	DIRECT SHEAR TEST (ASTM D 3080)
Peak Friction, ϕ' (deg):	28.4	Ultimate Friction, Φ' (deg):	27.9	

SAMPLE TYPE: Undisturbed
DESCRIPTION: Clayey SAND

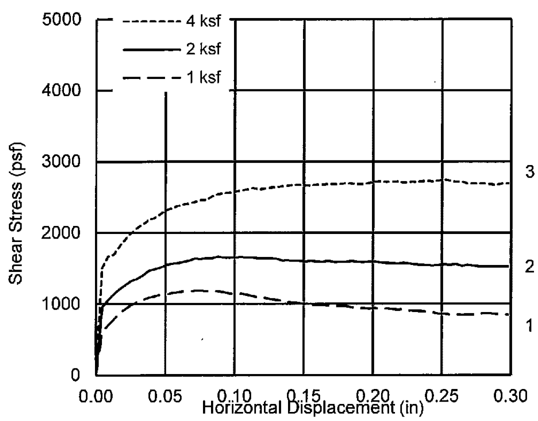
LL:
PL: USCS:
PI: GEOLOGY:

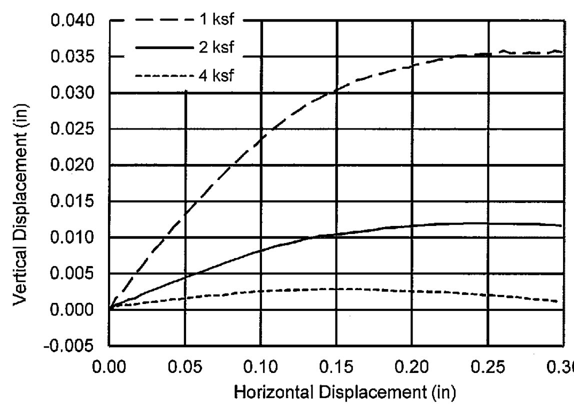
CLIENT: Bay City Geology
PROJECT NAME:
LOCATION: 20th St

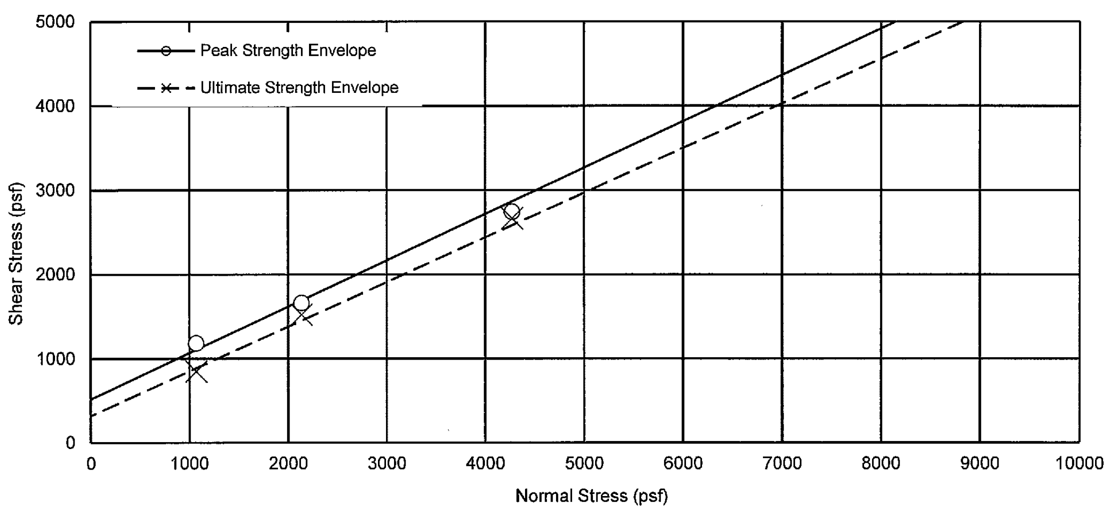
SAMPLE LOCATION: TP1 @ 3'

<u>*NOTE</u>: Sample was observed pre- and post-shear testing, no rock fragments (>0.24) inches present.

		Sample ID :	1 ksf	2 ksf	4 ksf			
Initial		Water Content (%)	10.8	10.8	10.8			
		Dry Density (%)	116.6	116.1	115.7			
		Saturation (%)	65.5	64.6	63.9			
Final		Water Content (%)	18.3	18.2	18.1			
		Dry Density (pcf)	115.8	115.1	114.3			
		Saturation (%)	108.6	106.0	103.1			
Normal Stress (psf)			1067	2134	4269			
Peak Shear Stress (psf)			1184	1662	2745			
Ultimate Shear Stress (psf)			847	1524	2665			







Peak Cohesion, c' (psf):	520	Ultimate Cohesion, c (psf):	321	DIRECT SHEAR TEST (ASTM D 3080)
Peak Friction, ϕ' (deg):	28.8	Ultimate Friction, Φ' (deg):	27.9	

SAMPLE TYPE: Undisturbed
DESCRIPTION: Clayey SAND

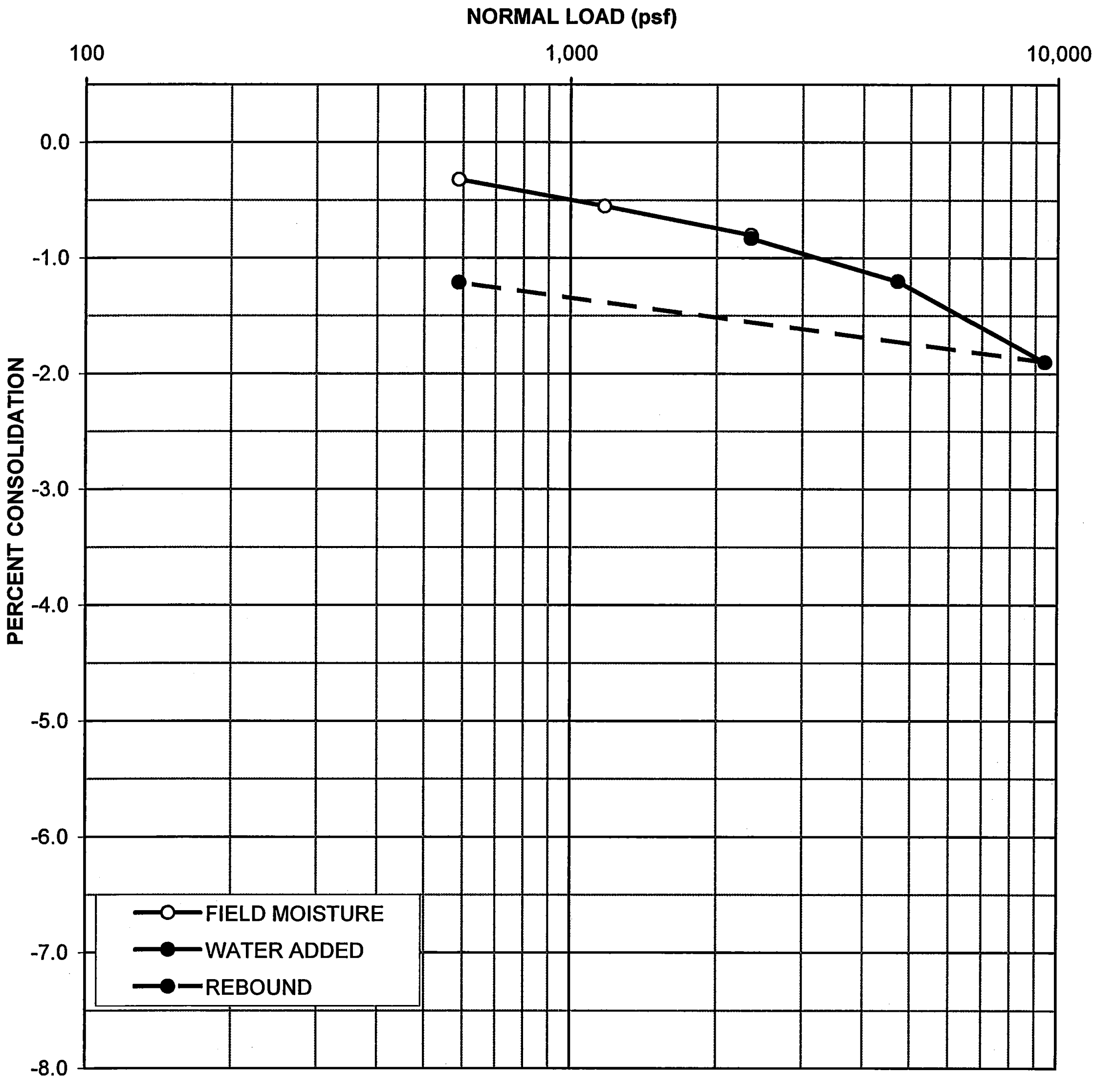
LL:
PL:
PI:

USCS:
GEOLOGY:

CLIENT: Bay City Geology
PROJECT NAME:
LOCATION: 20th St.

SAMPLE LOCATION: TP1 @ 6'

<u>*NOTE</u>: Sample was observed pre- and post-shear testing, no rock fragments (>0.24) inches present.



BORING NO. TP2 **DEPTH (FT)** 11.0

REPRESENTATIVE FOR Soil
SOIL TYPE AND DESCRIPTION

HYDROCONSOLIDATION (%) 0.03

APPENDIX III

ENGINEERING ANALYSIS

Settlement Calculations

Retaining Wall Calculations

Temporary Excavation Calculations

Seismic Design Considerations

Bay City Geology, Inc.

Project: 20th St (SM)/ BBS

File No.: 1735-1

Settlement Calculation - Column Footing

Description: 2.5' Square Footing

Gridline:

Soil Unit Weight	123.0	pcf	Column	Footing
Bearing Value	2000.0	psf	12.5	kips
Depth of Footing	2.5	feet		
Width of Footing	2.5	feet		

* Influence Values are based on Westergaard's Analyses

Depth Below Ground Surface (feet)	Average Depth Below Ground Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (a/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
3.0													
	4.0	1.5	1.7	38%	759	492	1251	TP 2 @ 11	0.55	0.22	0.33	2.5	0.10
5.0													
	6.0	3.5	0.7	13%	256	738	994	TP 2 @ 11	0.49	0.39	0.10	2.0	0.02
7.0													
	8.0	5.5	0.5	5%	95	984	1079	TP 2 @ 11	0.51	0.48	0.03	2.0	0.01
9.0													
	10.0	7.5	0.3	3%	60	1230	1290	TP 2 @ 11	0.56	0.53	0.03	2.0	0.01
11.0													
	12.0	9.5	0.3	1%	25	1476	1501	TP 2 @ 11	0.62	0.60	0.02	2.0	0.00
13.0													
	14.0	11.5	0.2	1%	25	1722	1747	TP 2 @ 11	0.68	0.66	0.02	2.0	0.00
15.0													
	16.0	13.5	0.2	1%	13	1968	1981	TP 2 @ 11	0.73	0.73	0.00	2.0	0.00
17.0													
	18.0	15.5	0.2	1%	13	2214	2227	TP 2 @ 11	0.78	0.78	0.00	2.0	0.00
19.0													
	20.0	17.5	0.1	1%	13	2460	2473	TP 2 @ 11	0.88	0.88	0.00	2.0	0.00
21.0													
	22.0	19.5	0.1	1%	13	2706	2719	TP 2 @ 11	0.91	0.91	0.00	2.0	0.00
23.0													
	24.0	21.5	0.1	1%	13	2952	2965	TP 2 @ 11	0.97	0.97	0.00	2.0	0.00
25.0													
	26.0	23.5	0.1	1%	13	3198	3211	TP 2 @ 11	1.00	1.00	0.00	2.0	0.00
27.0													

REFERENCE: Sowers, G. F. (1979) Introductory Soil Mechanics and Foundations: Geotechnical Engineering.
4th ed. Prentice Hall: New York, NY.

Total Settlement (in):	0.15
Differential Settlement (in):	0.10
-- to --	0.07

Bay City Geology, Inc.

Project: 20th St (SM)/ BBS

File No.: 1735-1

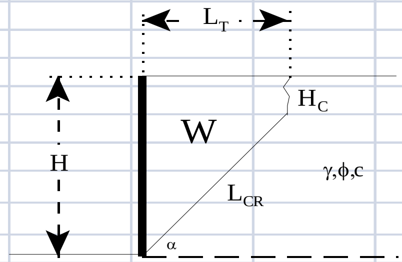
Description: Retaining Wall - Active Pressure Condition

[Assume: Retaining Terrace Deposit Soils (SC), Wall Height = 10 Feet Max.]

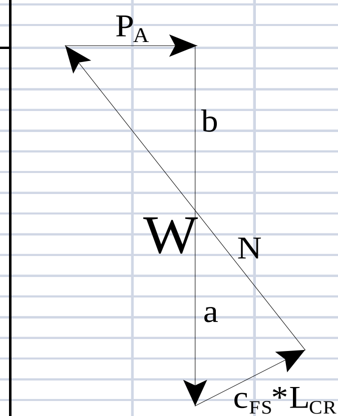
Retaining Wall Design with Level Backslope
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	138.6	pcf
Friction Angle of Retained Soils	(ϕ)	27.9	degrees
Cohesion of Retained Soils	(c)	312.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	19.4	degrees
	(c_{FS})	208.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.6	39	5438.7	7.6	3446.9	1991.8	952.5
46	4.6	38	5302.9	7.6	3319.8	1983.1	991.2
47	4.5	37	5162.4	7.5	3196.9	1965.5	1025.7
48	4.4	36	5018.5	7.5	3078.5	1940.0	1055.9
49	4.4	35	4872.3	7.5	2964.7	1907.7	1081.9
50	4.3	34	4724.6	7.4	2855.3	1869.2	1103.6
51	4.3	33	4575.8	7.3	2750.4	1825.4	1121.2
52	4.3	32	4426.5	7.3	2649.6	1776.9	1134.5
53	4.3	31	4277.2	7.2	2552.9	1724.2	1143.7
54	4.2	30	4127.9	7.1	2460.0	1667.9	1148.8
55	4.2	29	3979.0	7.0	2370.6	1608.4	1149.7
56	4.2	28	3830.5	6.9	2284.4	1546.1	1146.5
57	4.3	27	3682.7	6.8	2201.3	1481.4	1139.1
58	4.3	26	3535.5	6.7	2120.8	1414.6	1127.6
59	4.3	24	3388.9	6.6	2042.8	1346.0	1111.9
60	4.4	23	3242.9	6.5	1967.0	1275.9	1092.0
61	4.4	22	3097.6	6.4	1893.0	1204.6	1067.9
62	4.5	21	2952.8	6.3	1820.6	1132.2	1039.6
63	4.5	20	2808.5	6.1	1749.5	1059.0	1007.0
64	4.6	19	2664.5	6.0	1679.2	985.2	970.2
65	4.7	18	2520.7	5.9	1609.5	911.1	929.0
66	4.8	17	2376.9	5.7	1540.0	836.9	883.7
67	4.9	16	2233.1	5.5	1470.3	762.8	834.1
68	5.0	15	2088.8	5.4	1399.9	689.0	780.3
69	5.2	14	1944.0	5.2	1328.2	615.8	722.5
70	5.4	13	1798.3	4.9	1254.7	543.6	660.8



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 1149.73 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 23.0 pcf

Design Wall for an Equivalent Fluid Pressure: 60 pcf

Minimum Allowable Design Pressure, per Table 1610.1

Bay City Geology, Inc.

Project:	20th St (SM)/ BBS		
File No.:	1735-1		
Description:	Retaining Wall Design, At-Rest Pressure		
	[3 Feet MAX. height]		
Soil Weight	γ	138.6	pcf
Internal Friction Angle	ϕ	27.9	degrees
Cohesion	c	312	psf
Height of Retaining Wall	H	10	feet

Cantilever Retaining Wall Design based on At Rest Earth Pressure

$\sigma'_h = K_o \sigma'_v$

	$K_o = 1 - \sin\phi$	0.532	
	$\sigma'_v = \gamma H$	1386.0	psf
σ'_h =	737.4	psf	
EFP =	73.7	pcf	
P_o =	3687.2	lbs/ft	(based on a triangular distribution of pressure)

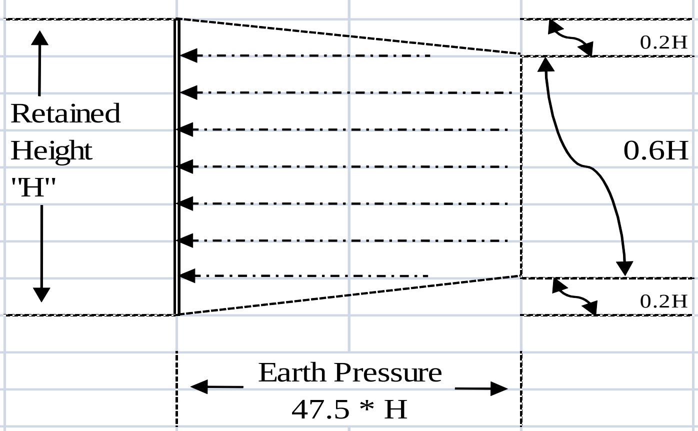
Design wall for an EFP of 76 pcf

Restrained Wall Design based on At Rest Earth Pressure

P_o =	3687.2	lbs/ft	
$\sigma'_{h, max}$ =	46.1	H	(based on a trapezoidal distribution of pressure)
$\sigma'_{h, max}$ =	368.7	psf	

Design restrained wall for 47.5 H

Trapazoidal Distribution of Earth Pressure



Bay City Geology, Inc.

Project: 20th St (SM) / BBS

File No.: 1738-1

Description: Retaining Wall- Seismicaly Induced Pressure

[Assume: Retaining Terrace Deposit Soil (SM) , Wall = 10 ft. Max.]

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Max. Height of Retaining Wall:	(H)	10.0	feet
Retained Soil Unit Weight:	(γ)	138.6	pcf
Horizontal Ground Acceleration:	($k_h = (1/2)*(2/3)*(PGA_M)$)	0.28	g

Seismic Increment (ΔP_{AE}):

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.75*k_h)$

ΔP_{AE} = 1477.8 lbs/ft

Force applied at 0.6H above the base of the wall
Transfer load to 2/3 of the height of the wall

$T*(2/3)*H = \Delta P_{AE}*0.6*H$

T = 1330.0 lbs/ft

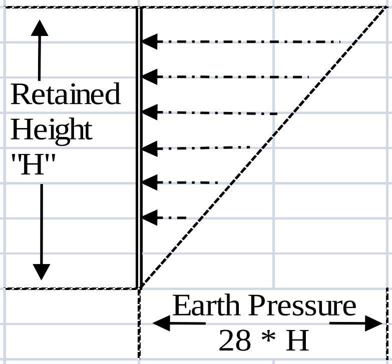
$EFP = 2*T/H^2$

EFP = 26.6 pcf

Design wall for an Seismic Lateral Soil EFP of 28 pcf

triangular distribution of pressure, inversely applied to the proposed retaining wall.

Triangular Inverse Distribution of Earth Pressure



Bay City Geology, Inc.

TEMPORARY EXCAVATION HEIGHT

File No. **1735-1**
Project: **20th St (SM)/ BBS**
ASSUME: 1H:1V (or flatter) Backslope
ASSUME: Basement Excavations

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST).
THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW.
ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Terrace Dep. Soils (SC)		WALL HEIGHT:	5	feet
SHEAR DIAGRAM:	TP - 1 @ 6 (ult.)		BACKSLOPE ANGLE:	45	degrees
COHESION:	312	psf	SURCHARGE:	0	pounds
PHI ANGLE:	27.9	degrees	SURCHARGE TYPE:	P	Point
DENSITY:	138.6	pcf	INITIAL FAILURE ANGLE:	10	degrees
SAFETY FACTOR:	1.25		FINAL FAILURE ANGLE:	80	degrees
WALL FRICTION:	0	degrees	INITIAL TENSION CRACK:	2	feet
CD (C/FS):	249.6	psf	FINAL TENSION CRACK:	20	feet
PHID = ATAN(TAN(PHI)/FS) =	23.0	degrees			

Run Calculation

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50	degrees
AREA OF TRIAL FAILURE WEDGE	9.6	square feet
TOTAL EXTERNAL SURCHARGE	0.0	pounds
WEIGHT OF TRIAL FAILURE WEDGE	1332.8	pounds
NUMBER OF TRIAL WEDGES ANALYZED	1349	trials
LENGTH OF FAILURE PLANE	3.1	feet
DEPTH OF TENSION CRACK	4.6	feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	2.0	feet
CALCULATED HORIZONTAL THRUST	**-122.5**	**pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-9.8**	**pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**5.0**	**feet**

CONCLUSIONS:

THE CALCULATION INDICATES THAT TEMPORARY COMPOUND EXCAVATIONS WITH A 5-FOOT VERTICAL AND A 1H:1V BACKSLOPE IN THE TERRACE DEPOSIT SOILS HAVE A NEGATIVE THRUST AND ARE TEMPORARILY STABLE.

Bay City Geology, Inc.	SLOT CUT CALCULATION
	File No.: **1735-1** Project: **20th St (SM)/ BBS** ASSUME: Grading Over-Excavations

CALCULATE THE FACTOR OF SAFETY OF SLOT CUT EXCAVATIONS. ASSUME COHESIVE AND FRICTIONAL RESISTANCE ALONG THE SIDES OF SLOTS AS WELL AS THE FAILURE SURFACE. THE HORIZONTAL PRESSURE ON THE SIDES OF THE SLOTS IS THE AT-REST PRESSURE (1-SIN(phi)).

CALCULATION PARAMETERS

EARTH MATERIAL:	Terrace Dep. Soils (SC)		EXCAVATION HEIGHT:	5	feet
SHEAR DIAGRAM:	TP - 1 @ 6 (ult.)		BACKSLOPE ANGLE:	0	degrees
COHESION:	312	psf	SURCHARGE:	0	pounds
PHI ANGLE:	27.9	degrees	SURCHARGE TYPE:	P	Point
DENSITY:	138.6	pcf	INITIAL FAILURE ANGLE:	10	degrees
SLOT BOUNDARY CONDITIONS			FINAL FAILURE ANGLE:	80	degrees
SLOT CUT WIDTH:	8	feet	INITIAL TENSION CRACK:	2	feet
COHESION:	156	psf	FINAL TENSION CRACK:	25	feet
PHI ANGLE:	13.95	degrees			

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50	degrees
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	2.0	feet
DEPTH OF TENSION CRACK	2.6	feet
TOTAL EXTERNAL SURCHARGE	0.0	pounds
VOLUME OF FAILURE WEDGE	60.9	ft^3
WEIGHT OF FAILURE WEDGE	8445.2	pounds
LENGTH OF FAILURE PLANE	3.1	feet
SURFACE AREA OF FAILURE PLANE	25	ft^2
SURFACE AREA OF SIDES OF SLOTS	7.6	ft^2
NUMBER OF TRIAL WEDGES ANALYZED	21164	trials
TOTAL RESISTING FORCE ALONG WEDGE BASE (FrB)	3863.8	pounds
TOTAL RESISTING FORCE ALONG WEDGE SIDES (FrS)	1230.9	pounds
RESULTANT HORIZONTAL COMPONENT OF FORCE	**-65.4**	**pounds**
CALCULATED FACTOR OF SAFETY	**2.01**	

CONCLUSIONS:

THE CALCULATION INDICATES THAT SLOT CUTS UP TO 8 FEET WIDE AND 5 FEET HIGH IN THE TERRACE DEPOSIT SOILS HAVE A SAFETY FACTOR GREATER THAN 1.25 AND ARE TEMPORARILY STABLE.

Seismic Design Considerations

Any new structures to be developed on the site and in the project area should be designed in accordance with the seismic design considerations contained in Section 1613 of the 2013 California Building Code and ASCE 7-10, the following parameters should be considered for design:

Subject Property Location: Latitude = 34.043122° Longitude = -118.497096° Mapped Spectral Response Acceleration Parameters: S_S = 2.172 ***g*** S_1 = 0.804***g*** Site Class: D - Stiff Soil Site Coefficients: F_a = 1.0 F_v = 1.5	Maximum Considered Earthquake Spectral Response Acceleration Parameters: S_{MS} = 2.172***g*** S_{M1} = 1.207***g*** Design Spectral Response Acceleration Parameters: S_{DS} = 1.448***g*** S_{D1} = 0.804***g*** Mapped Long Period Transition: T_L = 8 seconds Mapped Risk Coefficients (per ASCE 7-10 §21.2.1.1) C_{RS} = 0.931 C_{R1} = 0.933
Source: USGS U.S. Seismic Design Maps Website (2017) - http://earthquake.usgs.gov/designmaps/us/application.php	

Occupancy Group (per 2013 CBC §312.1) & Risk Category (per 2013 CBC Table 1604.5):

Group R-3 / Category II (Buildings and other structures except those listed in Risk Categories I, III, & IV)

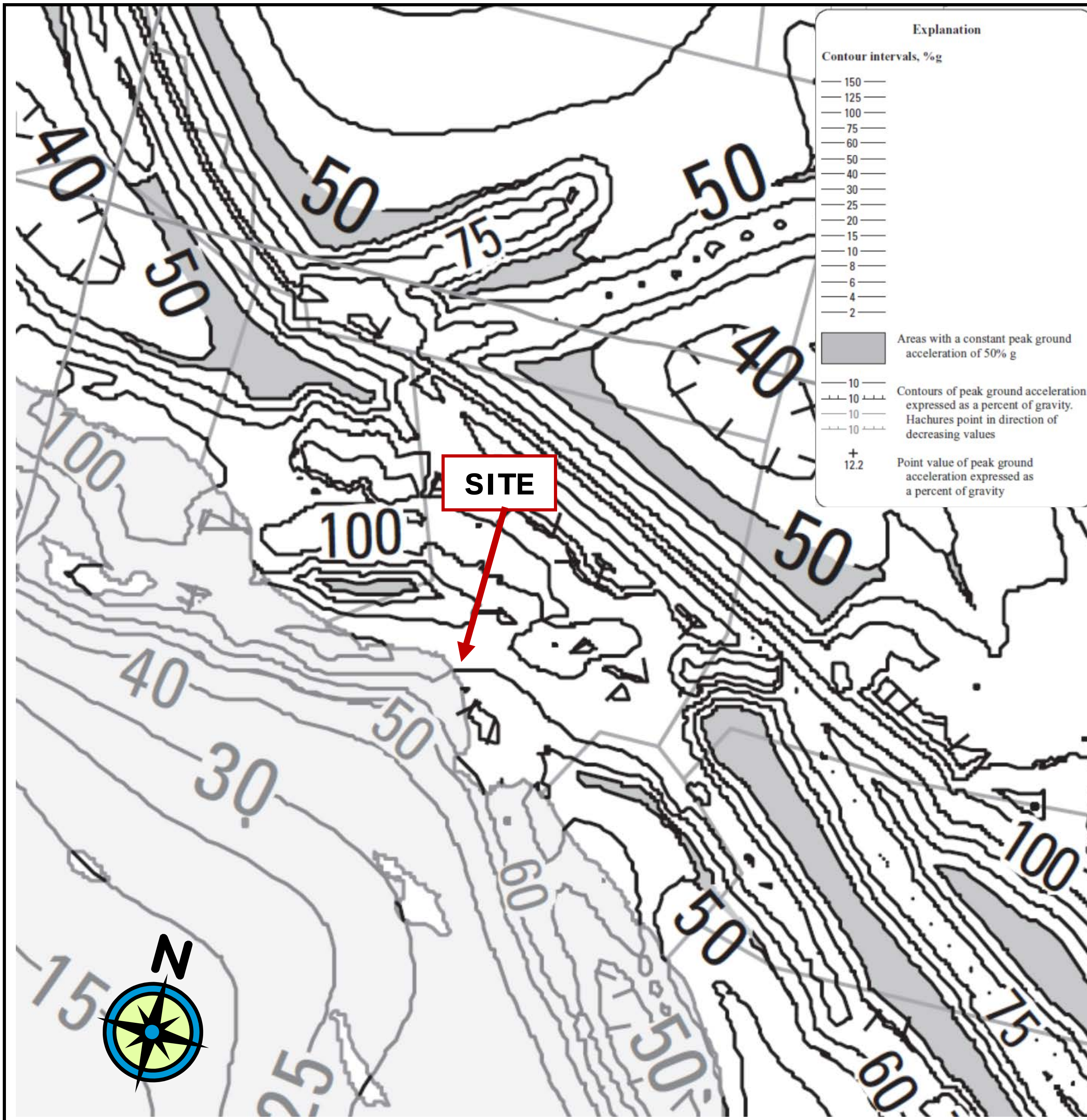
Seismic Design Category (per 2013 CBC §1613.5):

Category E

Peak Ground Acceleration (per 2013 CBC §1803.5.12 & ASCE 7-10 §11.8.3):

$PGA(MCE_G)$= 85.3% * ***g*** ; F_{PGA} = 1.0;
$PGA_M = F_{PGA} * PGA(MCE_G)$ = (1.0) * (0.853)***g*** = 0.853***g***
Note: "***g***" is the acceleration due to Gravity.

MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN



Reference:	Portion of Figure 22-7, ASCE Standards 7-10
Project Address:	247 20th Street Santa Monica, California — Plate 8

APPENDIX IV – GENERAL RECOMMENDATIONS

Drainage and Maintenance

Maintenance of the property and structures located within must be performed to minimize the chance of serious damage and/or instability to improvements. Most problems are associated with or triggered by water. Therefore, a comprehensive drainage system should be designed and incorporated into the final plans. In addition, pad areas should be maintained and planted in a way that will allow this drainage system to function as intended. The following are specific drainage, maintenance, and landscaping recommendations. Reductions in these recommendations will reduce their effectiveness and may lead to damage and/or instability to the improvements. It is the responsibility of the property owner to ensure that the residence and drainage devices are maintained in accordance with the following recommendations and the requirements of all applicable government agencies.

Drainage
Positive pad drainage should be incorporated into the final plans. The pad should slope away from the footings at a minimum five percent slope for a horizontal distance of five feet. In areas where there is insufficient space for the recommended five foot horizontal distance concrete or other impermeable surface should be provided for a minimum of three feet adjacent the structure. Pad drainage should be at a minimum of two percent slope where water flows over lawn or other planted areas. Drainage swales should be provided with area drains about every fifteen feet. Areas drains should be provided in the rear and side yards to collect drainage. All drainage from the pad should be directed so that water does not pond adjacent to the foundations or flow towards them. Roof gutters and downspouts are required for the proposed structures and should be connected into a buried area drain system. All drainage from the site should be collected and directed via non-erosive devices to a location approved by the building official. Area drains, subdrains, weep holes, roof gutters and downspouts should be inspected periodically to ensure that they are not clogged with debris or damaged. If they are clogged or damaged, they should be cleaned out or repaired.

Landscaping (Planting)
Planters placed immediately adjacent to the structures are not recommended. If planters are proposed immediately adjacent to structures, impervious above-grade or below-grade planter boxes with solid bottoms and drainage pipes away from the structure are suggested. All slopes should be maintained with a dense growth of plants, ground-covering vegetation, shrubs and trees that possess dense, deep root structures and require a minimum of irrigation. Plants surrounding the development should be of a variety that requires a minimum of watering. It is recommended that a landscape architect be consulted regarding planting adjacent to improvements. It will be the responsibility of the property owner to maintain the planting. Alterations of planting schemes should be reviewed by the landscape architect.

Irrigation
An adequate irrigation system is required to sustain landscaping. Over-watering resulting in runoff and/or ground saturation must be avoided. Irrigation systems must be adjusted to account for natural rainfall conditions. Any leaks or defective sprinklers must be repaired immediately. To mitigate erosion and saturation, automatic sprinkling systems must be adjusted for rainy seasons. A landscape architect should be consulted to determine the best times for landscape watering and the proper usage.

Pools/Plumbing
Leakage from a swimming pool or plumbing can produce a perched groundwater condition that may cause instability or damage to improvements. Therefore, all plumbing should be leak-free. Pools located adjacent to descending slopes should be provided with a pool subdrain system.

Grading and Earthwork

General Grading Guidelines

1. Prior to commencement of work, a pre-grading meeting shall be held. Participants at this meeting will consist of the contractor, the owner or his representative, and the soils engineer and/or engineering geologist. The purpose of the meeting is to avoid misunderstanding of the recommendations set forth in this report that might cause delays in the project.
2. Prior to placement of fill materials, all vegetation, rubbish, and other deleterious material should be disposed of off-site. The proposed structures should be staked out in the field by a surveyor. This staking should, as a minimum, include areas for over-excavation, toes of slopes, tops of cuts, setbacks, and easements. All staking shall be offset from the proposed grading area at least (5) feet.

   The proposed construction areas should be excavated down to competent bedrock (or other recommended competent earth material).
3. The excavated grade (or “bottom”), that is determined to be satisfactory for the support of the controlled fill materials, shall then be scarified to a depth of at least (6) inches and moistened as required. The bottom should be compacted to at least (90) percent relative compaction.
4. The controlled fill materials shall consist of earth materials approved by the project soils engineer and/or engineering geologist. These materials may be obtained from the on-site excavation areas, from any other approved source areas, and by blending soils from one or more sources. The controlled fill materials used shall be free from organic matter, vegetation, and other deleterious substances. Also, the controlled fill materials shall not contain rocks greater than (8) inches in diameter, nor of a quantity sufficient to make compaction difficult.
5. The approved controlled fill materials shall be placed in approximately level layers (“lifts”) about (4 to 6) inches thick, and moistened as required. Each layer shall be thoroughly mixed to attain uniformity of moisture in each layer.

   When the moisture content of the controlled fill materials is found to be (3) percent or more below the optimum moisture content, as specified by the soils engineer, water shall be added and thoroughly mixed in until the moisture content is brought up to the optimum moisture content, and <u>no more than</u> (3) percent above the optimum moisture content.

   When the moisture content of the controlled fill materials is greater than (3) percent above the optimum moisture content, as specified by the soils engineer, the fill material shall be either dried and aerated by scarifying, or it shall be blended with additional drier fill materials and thoroughly mixed until the moisture content is brought down to the optimum moisture content, and <u>no more than</u> (3) percent above the optimum moisture content.

   Each lift of controlled fill materials shall be compacted to a minimum of (90) percent relative compaction (as determined by the modified Proctor maximum dry density - ASTM D 1557), using approved compaction equipment. Where cohesionless soil having less than (15) percent finer than (0.005) millimeters is used for controlled fill materials, the controlled fill material shall be compacted to a minimum of (95) percent relative compaction.
6. Review of controlled fill material placement and compaction should be provided by the soils engineer (or his designee) during the progress of grading. Generally, density tests will be required at intervals not exceeding (2) feet of fill height/depth, or for every 500 cubic yards of controlled fill materials placed.
7. During periods when inclement weather is expected at the project site, all controlled fill materials that have been spread and are awaiting compaction shall be compacted before stopping work, either because of inclement weather or at the end of the work day. The upper surface of the controlled fill

area shall sloped/contoured to drain all precipitation to a single location; where water may be collected and removed from the controlled fill area.

Following inclement weather, work may resume only after the condition of the controlled fill area and materials have been review by the soils engineer, and he has given authorization to resume work. Loose fill materials not compacted prior to the rain shall be removed and aerated so that the moisture content of these controlled fill materials will be not less than and no more than (3) percent above the optimum moisture content.

Surface materials previously compacted before the inclement weather period, shall be scarified, brought to the proper moisture content, and re-compacted prior to placing additional controlled fill materials, if deemed necessary by the soils engineer

8. Review of geotechnical data available for the local vicinity of the site indicates that septic tanks, seepage pits/cesspools, or leach fields may be encountered during site grading. If encountered, these should be drained of effluent or drilled out if they have been backfilled. The cleaned-out area should be inspected by the soils engineer and the local building official prior to backfill. Seepage pits/cesspools may be filled with approved controlled fill materials, lean-mix concrete (or 2-sack slurry), or (¾) inch crushed rock gravel. Whichever backfill material is selected, at least (5) feet of controlled fill materials, placed at a minimum of (90) percent relative compaction should cap the backfilled seepage pit/cesspool.

Retaining Wall Backfilling

Walls to be backfilled must be reviewed by the project soils engineer prior to commencement of the backfilling operation.

1. Adequate permanent drainage is required behind the wall to minimize the buildup of hydrostatic pressures. A perforated pipe, with perforations placed down, shall be installed at the base of the wall footing. The pipe shall be encased in at least (1) foot of (¾) inch gravel. The pipe shall exit from behind the retaining wall and drain to a location approved by the architect or civil engineer.

   When space does not permit the installation of standard pipe and gravel drainage system, e.g. walls adjacent the property line, a flat drainage product is acceptable subject to approval of the governing agency.

   If a drainage system is not provided the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure in Retaining Walls section of this report. The entire wall should be design for full hydrostatic pressure based on a water level at the ground surface. In addition, floors would need to be designed for hydrostatic uplift and waterproofed.

2. A continuous vertical drain, consisting of a gravel blanket (6) inches thick or geotextile vertical drainage system, shall be placed along the back side of the wall to within (2) feet of the ground surface.

3. It is recommended that the retaining walls be waterproofed. Waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer. Bay City Geology, Inc. does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary.

4. After the wall backdrain system has been placed and the waterproofing installed, fill may be placed, if sufficient room allows, in layers not exceeding (4) inches in thickness and compacted to (90) percent of the maximum density, as determined by ASTM D1557. Where cohesionless soil having less than (15) percent finer than (0.005) millimeters is used for fill, the fill material shall be compacted to a minimum of (95) percent of the maximum dry density.

5. Where space does not permit compaction of material behind the wall (<24 inches wide), a granular backfill shall be used. This granular backfill shall consist of (½) inch to (¾) inch crushed rock gravel and should be densified by tamping into place. The crushed rock gravel backfill should not exceed a depth of (10) feet.

6. All granular free-draining wall backfills shall be capped with a clayey compacted soil within the upper (2) feet of the wall backfill. This compacted material should start below any required wall freeboard.

7. A concrete-lined swale drain should be placed behind any retaining wall that can intercept surface runoff from upslope areas. This surface runoff shall be transferred to an area approved by the building official.

8. A minimum freeboard of (24) inches shall be maintained at all times for all exterior retaining walls surcharged with a sloping back-cut. Any slough, debris or trash should be removed immediately. Swales shall be maintained, by sealing any and all cracks or repairing breaks that occur over the life of the swale.

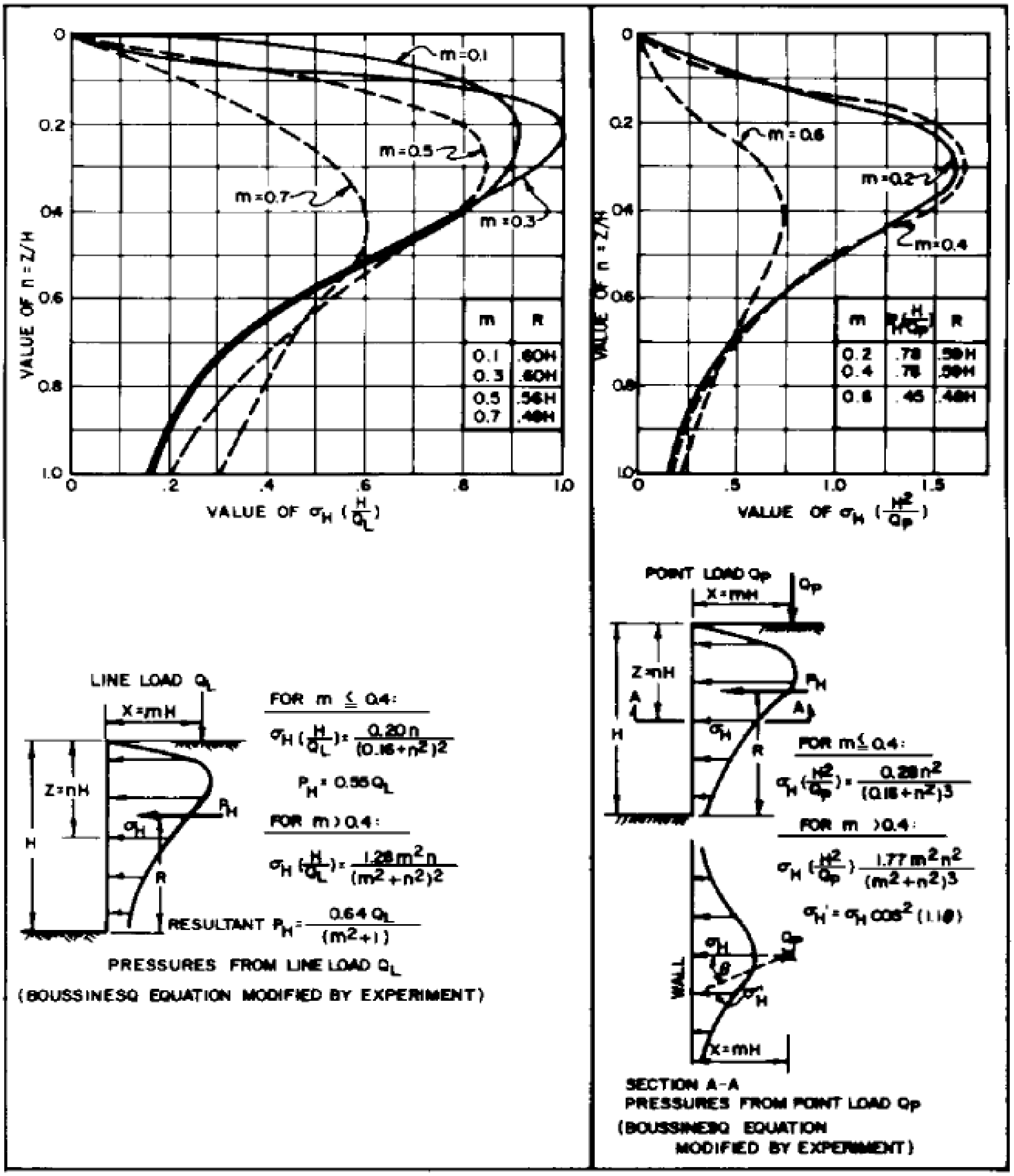


FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

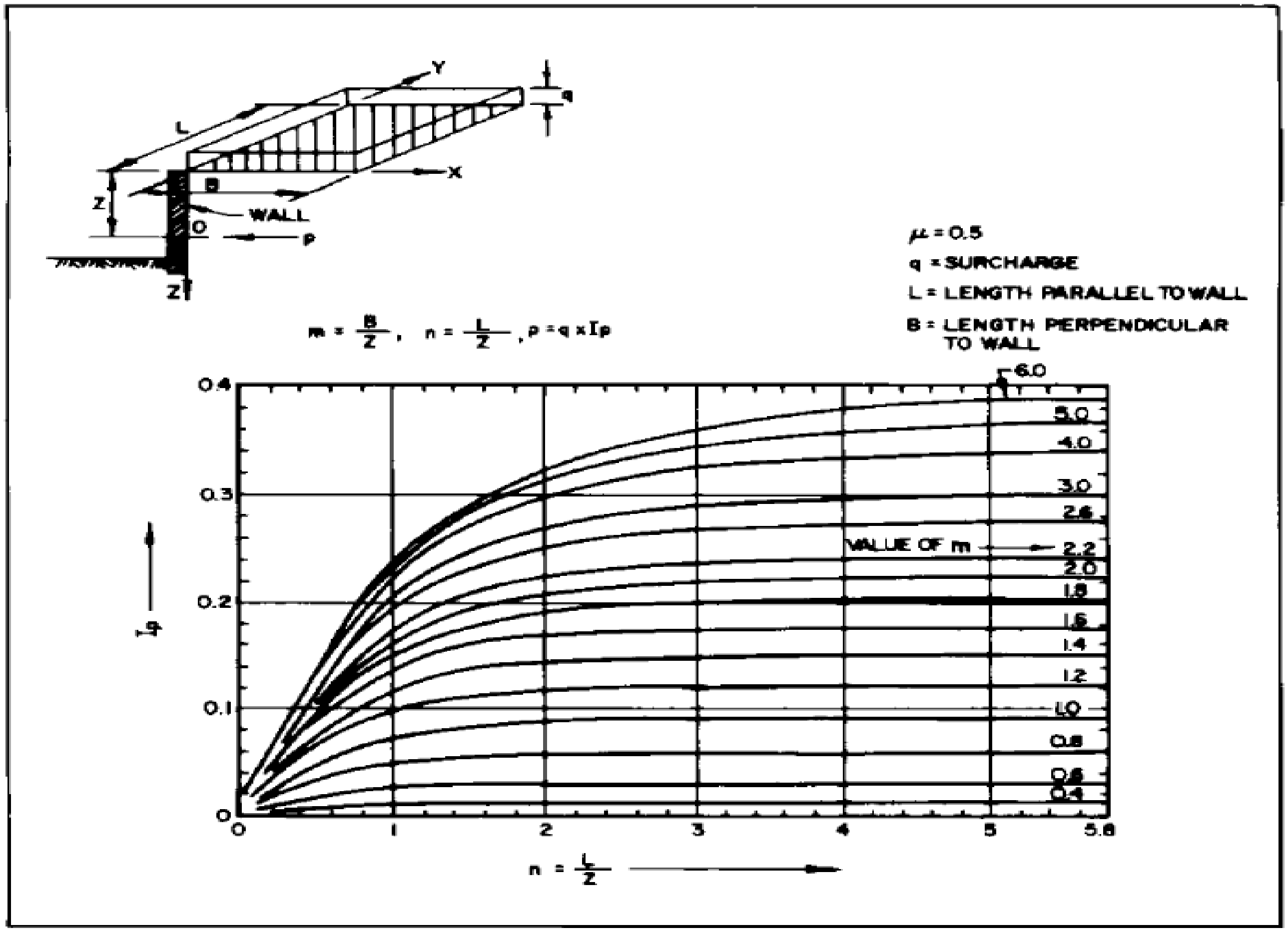


FIGURE 12
Lateral Pressure on an Unyielding Wall due to
Uniform Rectangular Surface Load

7.2-75

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