GEOTECHNICAL INVESTIGATION

JUNIPERO SERRA MUSEUM ADA IMPROVEMENTS

San Diego, California

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Report No. 1

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Subject: GEOTECHNICAL INVESTIGATION
JUNIPERO SERRA MUSEUM ADA IMPROVEMENTS
2727 PRESIDIO DRIVE
SAN DIEGO, CALIFORNIA

Dear Larry:

SCST, LLC (SCST) is pleased to present our report describing the geotechnical investigation performed for the subject project. We conducted the investigation in general conformance with the scope of work presented in our proposal dated November 30, 2017. Based on the results of our investigation, we consider the planned development feasible from a geotechnical standpoint, provided the recommendations of this report are followed. If you have any questions, please call us at (619) 280-4321.

Respectfully submitted,
SCST, LLC

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1. INTRODUCTION

This report presents the results of the geotechnical investigation SCST, LLC (SCST) performed for the subject project. We understand the project will consist of the design and construction of a new concrete ADA accessible path, driveway, parking area, landscaping, and biofiltration basins located south of the main terrace of Junipero Serra Museum. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project. Figure 1 presents the site vicinity map.

2. SCOPE OF WORK

2.1 FIELD INVESTIGATION

We explored the subsurface conditions by drilling four borings and 3 percolation test borings to depths between about 3 and 5 feet below the existing ground surface using a hand auger. Auger refusal was encountered in each boring except B-4. Figure 2 presents the approximate locations of the borings. An SCST geologist logged the borings and collected samples of the materials encountered for laboratory testing. Logs of the borings are presented in Appendix I. Soils are classified according to the Unified Soil Classification System illustrated on Figure I-1.

2.2 LABORATORY TESTING

We tested selected samples to evaluate soil classification and engineering properties and develop geotechnical conclusions and recommendations. The laboratory tests consisted of in situ moisture and density, particle-size distribution, Atterberg limits, R-value, expansion index, and corrosivity. The results of the laboratory tests and brief explanations of the test procedures are presented in Appendix II.

2.3 BOREHOLE PERCOLATION TESTING

We performed borehole percolation testing in general conformance with the City of San Diego BMP Design Manual at three locations (P-1, P-2, and P-3) to assess stormwater infiltration feasibility. The results of the testing are presented in Appendix III.

2.4 ANALYSIS AND REPORT

The results of the field and laboratory tests were evaluated to develop conclusions and recommendations regarding:

- Subsurface conditions beneath the site
- Potential geologic hazards
- Criteria for seismic design in accordance with the 2016 California Building Code (CBC)
- Site preparation and grading

- Excavation characteristics
- Concrete slabs-on-grade
- Pavement sections
- Foundation support, potential foundation settlement, resistance to lateral loads, and lateral earth pressures for retaining wall design
- Pipelines
- Soil corrosivity
- Infiltration test results and feasibility

3. SITE DESCRIPTION

The site is within a eucalyptus tree grove located in Presidio Park at 2727 Presidio Drive in the City of San Diego, California. The existing park consists of historic buildings, pavements, hardscape, and landscape areas. The site is located on a mesa south of Mission Valley. Existing site elevations range from about 175 feet near the top of the mesa with gradual slopes down to about 145 feet to the north and northwest, and gradual slopes down to about 165 feet to the southwest of the mesa.

4. PROPOSED DEVELOPMENT

We understand the project will consist of the design and construction of a new concrete ADA accessible path, driveway, parking area, landscaping, and biofiltration basins located south of the main terrace of Junipero Serra Museum. Current improvement grading plans show that cuts and fills from 5 to 10 feet will be required to achieve finish site grades.

5. GEOLOGY AND SUBSURFACE CONDITIONS

The materials encountered in our borings consist of fill and very old paralic deposits. Descriptions of the materials encountered are presented below. Figure 3 presents the regional geology in the vicinity of the site.

Fill: Fill was encountered in each boring except for P-1. The fill encountered in the borings extends to depths varying from about 1 foot below the existing ground surface to the maximum-explored depth of 5 feet and consists of loose to medium dense silty to clayey sand and soft to stiff sandy clay with varying amounts of gravel and cobbles.

Very Old Paralic Deposits: Very old paralic deposits underlie the entire site and were encountered in each boring except for B-4. These deposits consist of weathered, moderately to strongly cemented silty sandstone.

Groundwater: Groundwater was not encountered in the borings. The groundwater table is expected to be below a depth that will influence the planned construction. However, groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage. Because groundwater rise or seepage is difficult to predict, such conditions are typically mitigated if and when they occur.

6. GEOLOGIC HAZARDS

6.1 CITY OF SAN DIEGO SEISMIC SAFETY STUDY

Figure 4 shows the approximate site location on the City of San Diego Seismic Safety Study map. The site is located in Geologic Hazard Category 53, defined as level or sloping terrain and unfavorable geologic structure with moderate to low risk. In our opinion, the geologic risk is low.

6.2 FAULTING AND SURFACE RUPTURE

No active faults are known to underlie or project toward the site. However, the potentially active Rose Canyon fault zone is located about 800 feet southwest of the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, the probability of fault rupture is low.

6.3 CBC SEISMIC DESIGN PARAMETERS

A geologic hazard likely to affect the project is ground shaking as a result of movement along an active fault zone in the vicinity of the subject site. The site coefficients and maximum considered earthquake (MCE_R) spectral response acceleration parameters in accordance with the 2016 CBC are presented below:

Site Coordinates: Latitude 32.758357°
Longitude -117.193069°

Site Class: C

Site Coefficients, F_a = 1.000
F_v = 1.307

Mapped Spectral Response Acceleration at Short Period, S_s = 1.273g

Mapped Spectral Response Acceleration at 1-Second Period, S_1 = 0.493g

Design Spectral Acceleration at Short Period, S_{DS} = 0.849g

Design Spectral Acceleration at 1-Second Period, S_{D1} = 0.429g

Site Peak Ground Acceleration, PGA_M = 0.578g

6.4 LIQUEFACTION AND DYNAMIC SETTLEMENT

Liquefaction occurs when loose, saturated sands and silts are subjected to strong ground shaking. The soils lose shear strength and become liquid, resulting in large total and differential ground surface settlements and possible lateral spreading during an earthquake. Given the dense nature of the materials beneath the site, and due to the lack of a shallow groundwater table, the potential for liquefaction and dynamic settlement to occur is low.

6.5 LANDSLIDES AND SLOPE STABILITY

Evidence of landslides or slope instabilities was not observed during our investigation. However, Kennedy and Tan (2008) have noted that the Presidio Park area is underlain by an ancient landslide. Mike Hart (2014) conducted a detailed fault and landslide study of the Presidio Park area and noted that although probable ancient landslides do exist in the Presidio Park area, the mapped landslides are much smaller in size than what Kennedy and Tan mapped in 2008. Hart's mapped areas of landslides are presented in Figure 5 and are outside of the project limits. The potential for landslides or slope instabilities to occur at the site is considered low.

6.6 TSUNAMIS, SEICHES, AND FLOODING

The site is not located within a mapped area on the State of California Tsunami Inundation Maps (Cal EMA, 2009); therefore, damage due to tsunamis is considered low. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to lakes or confined bodies of water; therefore, the potential for a seiche to affect the site is considered negligible. The site is not located within a flood zone or dam inundation area (County of San Diego, 2012).

6.7 SUBSIDENCE

The site is not located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum); therefore, the potential for subsidence due to the extraction of fluids is considered low.

6.8 HYDRO-CONSOLIDATION

Hydro-consolidation can occur in recently deposited sediments (less than 10,000 years old) that were deposited in a semi-arid environment. Examples of such sediments are aeolian sands, alluvial fan deposits, and mudflow sediments deposited during flash floods. The pore spaces between the particle grains can re-adjust when inundated by groundwater causing the material to consolidate. The relatively dense materials underlying the site are not considered susceptible to hydro-consolidation.

7. CONCLUSIONS

Based on the results of our investigation, we consider the proposed construction feasible from a geotechnical standpoint provided the recommendations of this report are followed. The main geotechnical considerations affecting the proposed ADA access path, driveway, parking area, and other improvements are the presence of potentially compressible fill and potentially expansive soils. Remedial grading is recommended to reduce the potential for distress to the planned improvements. Remedial grading recommendations are provided herein. The recommendations presented herein may need to be updated once final plans are developed.

8. RECOMMENDATIONS

8.1 SITE PREPARATION AND GRADING

8.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, and debris. Subsurface improvements that are to be abandoned should be removed, and the resulting excavations should be backfilled and compacted in accordance with the recommendations of this report. Pipeline abandonment can consist of capping or rerouting at the project perimeter and removal within the project perimeter. If appropriate, abandoned pipelines can be filled with grout or slurry as recommended by and observed by the geotechnical consultant.

8.1.2 Remedial Grading

To improve subgrade support, the existing soils should be excavated to a depth of at least 1 foot below finish subgrade elevation for hardscape and pavements and at least 2 feet below the footing bottom elevation for retaining walls. Horizontally, the excavations should extend a distance equal to the depth of excavation, up to existing improvements or the limits of disturbance, whichever is less. An SCST representative should observe conditions exposed in the bottom of the excavation to determine if additional excavation is required.

8.1.3 Expansive Soil

The on-site soils tested have expansion indexes of 6 and 59. To reduce the potential for expansive heave, the top 2 feet of material beneath hardscape, pavements, and retaining wall footings should have an expansion index (EI) of 20 or less determined in accordance with ASTM D4829. The on-site silty to clayey sands are expected to meet this expansion index criteria. However, the on-site sandy clay is not expected to meet the expansion index criteria.

8.1.4 Compacted Fill

Prior to placing fill, the exposed surface should be scarified to a depth of 12 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. Excavated material, except for vegetation, debris, soils with an EI greater than 20, or rocks greater than 6 inches can be used as compacted fill. Fill should be placed in horizontal lifts at a thickness appropriate for the equipment spreading, mixing, and compacting the material, but generally should not exceed 8 inches in loose thickness. Fill should be moisture conditioned to near optimum moisture content and compacted to at least 90% relative compaction. The maximum density and optimum moisture content for the evaluation of relative compaction should be determined in accordance with ASTM D1557. The top 12 inches of subgrade beneath pavements subjected to vehicular traffic should be compacted to at least 95%.

8.1.5 Imported Soil

Imported soil should consist of predominately granular soil free of organic matter and rocks greater than 6 inches. Imported soil should be observed and, if appropriate, tested by SCST prior to transport to the site to determine suitability for the intended use.

8.1.6 Excavation Characteristics

It is anticipated that excavations can be achieved with conventional earthwork equipment in good working order. Difficult excavation should be anticipated in cemented zones within the very old paralic deposits. Gravel and cobbles should also be anticipated.

8.1.7 Oversized Material

Excavations may generate oversized material. Oversized material is defined as rocks or cemented clasts greater than 6 inches in largest dimension. Oversized material should be broken down to no greater than 6 inches in largest dimension for use in fill, used as landscape material, or disposed of off-site.

8.1.8 Temporary Excavations

Temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations in fill should be laid back no steeper than 1:1 (horizontal:vertical). Deeper temporary excavations in formational materials should be laid back no steeper than ¾:1

(horizontal:vertical). The faces of temporary slopes should be inspected daily by the contractor's Competent Person before personnel are allowed to enter the excavation. Zones of potential instability, sloughing, or raveling should be brought to the attention of the Engineer and corrective action implemented before personnel begin working in the excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. SCST should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces.

Slopes steeper than those described above will require shoring. Additionally, temporary excavations that extend below a plane inclined at 1½:1 (horizontal:vertical) downward from the outside bottom edge of existing structures or improvements will require shoring. Soldier piles and lagging, internally braced shoring, or trench boxes could be used. If trench boxes are used, the soil immediately adjacent to the trench box is not directly supported. Ground surface deformations immediately adjacent to the pit or trench could be greater where trench boxes are used compared to other methods of shoring.

As an alternative to shoring/underpinning, maximum 10-foot-wide slots can be excavated and immediately backfilled adjacent to existing structures and improvements. Care should be taken to not undermine existing footings. Slot excavations should be filled prior to performing adjacent excavations.

8.1.9 Temporary Shoring

For design of cantilevered shoring with level backfill, an active earth pressure equal to a fluid weighing 35 pounds per cubic foot (pcf) can be used. An additional 20 pcf should be added for shoring with 2:1 sloping ground. The surcharge loads on shoring from traffic and construction equipment working adjacent to the excavation can be modeled by assuming an additional 2 feet of soil behind the shoring. For design of soldier piles, an allowable passive pressure of 350 pounds per square foot (psf) per foot of embedment over two times the pile diameter up to a maximum of 5,000 psf can be used. Soldier piles should be spaced at least three pile diameters, center to center. Continuous lagging will be required throughout. The soldier piles should be designed for the full anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soils. For design of lagging, the earth pressure can be limited to a maximum value of 400 psf.

8.1.10 Slopes

Permanent slopes should be constructed no steeper than 2:1 (horizontal:vertical). Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment or by overfilling and cutting back to design grade. Fills should be benched into sloping ground inclined steeper than 5:1 (horizontal:vertical). In our opinion, slopes constructed no steeper than 2:1 (horizontal:vertical) will possess an adequate factor of safety. An engineering geologist should observe cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations. Slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

8.1.11 Surface Drainage

Final surface grades around structures should be designed to collect and direct surface water away from the structure and toward appropriate drainage facilities. The ground around the structure should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to the structure slope away at a gradient of at least 2%. Densely vegetated areas where runoff can be impaired should have a minimum gradient of at least 5% within the first 5 feet from the structure. Roof gutters with downspouts that discharge directly into a closed drainage system are recommended on structures. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed structures. Site irrigation should be limited to the minimum necessary to sustain landscape growth. Should excessive irrigation, impaired drainage, or unusually high rainfall occur, saturated zones of perched groundwater can develop.

8.1.12 Grading Plan Review

SCST should review the grading plans and earthwork specifications to ascertain whether the intent of the recommendations contained in this report have been implemented and that no revised recommendations are needed due to changes in the development scheme.

8.2 PAVEMENT SECTION RECOMMENDATIONS

The pavement support characteristics of the fill soils encountered during our investigation are considered poor. An R-value of 5 was used for design of preliminary pavement sections based on R-value testing conducted from on-site soils sampled. The actual R-value of the subgrade soils should be determined after grading, and the final pavement sections should be provided.

Based on an R-value of 5, the following preliminary pavement structural sections are provided for the assumed Traffic Indexes.

Traffic Type	Traffic Index	Asphalt Concrete (inches)	Portland Cement Concrete (inches)
Bikeways and Light Vehicles	4.5	3 AC / 8 AB	6 PCC
Medium Trucks	6.0	5½ AC / 9 AB	7 PCC / 6 AB

AC - Asphalt Concrete
AB - Aggregate Base
PCC - Portland Cement Concrete

The top 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction. Soft or yielding areas should be stabilized or removed and replaced with compacted fill or aggregate base. Aggregate base and asphalt concrete should conform to the Caltrans Standard Specifications or the "Greenbook" and should be compacted to at least 95% relative compaction. Aggregate base should have an R-value of not less than 78. Materials and methods of construction should conform to good engineering practices and the minimum local standards.

8.3 PERVIOUS PAVEMENT SECTION RECOMMENDATIONS

Pervious pavement section recommendations are based on Caltrans (2014) pavement structural design guidelines. The pavement sections below are based on the strength of the materials. However, the actual thickness of the sections may be controlled by the reservoir layer design, which the project civil engineer should determine.

Pervious Asphalt Pavement

Traffic Type	Category	*Asphalt Treated Permeable Base (ATPB) (inches)	Class 4 Aggregate Base (inches)
Landscape Areas, Sidewalks and Bike path (no vehicular access)	A	0**	6
Bike path and Light Vehicles	B	6	10

***1¼ inches of an open-graded friction course (OGFC) should be placed on top of the ATPB.**
****2½ inches of an open-graded friction course (OGFC) should be placed on top of the Class 4 Agg Base.**

Pervious Concrete Pavement

Traffic Type	Category	Pervious Concrete (inches)	Class 4 Aggregate Base (inches)
Landscape Areas, Sidewalks and Bike path (no vehicular access)	A	4½	0
Bike path and Light Vehicles	B	6	8½

Permeable Interlocking Concrete Pavers (PICP)

Traffic Type	Category	PICP (inches)	Class 3 Permeable (inches)	Class 4 Aggregate Base (inches)
Landscape Areas, Sidewalks and Bike path (no vehicular access)	A	2⅜*	4½	0
Bike path and Light Vehicles	B	3⅛*	4½	8½

***2 inches of bedding layer should be placed between Class 3 permeable material and pavers.**

The top 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction. Soft or yielding subgrade areas should be stabilized or removed and replaced with compacted fill or permeable base. Materials and methods of construction should conform to good engineering practices and the minimum local standards.

Deepened curbs or vertical cutoff membranes consisting of 30 mil HDPE or PVC should be installed at the edges of pervious pavements to reduce the potential for water-related distress to adjacent structures or improvements.

8.4 CONCRETE PEDESTRIAN WALKS

Exterior concrete slabs should be underlain by at least 2 feet of material with an EI of 20 or less. Exterior slabs not subjected to vehicular weight should be at least 4 inches thick and reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project architect should select the final joint patterns. A 1-inch maximum size aggregate mix is recommended for concrete for exterior slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the "Greenbook" Standard Specifications for Public Works Construction.

8.5 CONVENTIONAL RETAINING WALLS

8.5.1 Foundations

The planned retaining walls can be supported on spread footings with bottom levels on compacted fill or very old paralic deposits. Footings should extend at least 18 inches below lowest adjacent finished grade and be at least 24 inches wide. Footings located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 7 feet exists between the lower outside footing edge and the face of the slope.

8.5.2 Allowable Soil Bearing

An allowable bearing capacity of 2,500 psf can be used. The allowable bearing capacity can be increased by 500 psf for each foot of depth below the minimum and 250 psf for each foot of width beyond the minimum up to a maximum of 5,000 psf. The bearing value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces.

8.5.3 Resistance to Lateral Loads

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. An allowable coefficient of friction of 0.35 can be used. Passive pressure can be computed using an allowable lateral pressure of 350 psf per foot of depth below the ground surface for level ground conditions. The passive pressure can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

8.5.4 Settlement Characteristics

Total foundation settlements are estimated to be less than 1 inch. Differential settlements across continuous footings are estimated to be less than ¾ inch over a distance of 40 feet. Settlements should be completed shortly after structural loads are applied.

8.5.5 Foundation Plan Review

SCST should review the foundation plans to ascertain that the intent of the recommendations in this report has been implemented and that revised recommendations are not necessary as a result of changes after this report was completed.

8.5.6 Foundation Excavation Observations

A representative from SCST should observe the foundation excavations prior to forming or placing reinforcing steel.

8.5.7 Lateral Earth Pressures

The active earth pressure for the design of unrestrained retaining walls with level backfill can be taken as equivalent to the pressure of a fluid weighing 35 pcf. The at-rest earth pressure for the design of restrained retaining walls with level backfills can be taken as equivalent to the pressure of a fluid weighing 55 pcf. These values assume a granular and drained backfill condition. Higher lateral earth pressures would apply if walls retain clay soils. An additional 20 pcf should be added to these values for walls with a 2:1 (horizontal:vertical) sloping backfill. An increase in earth pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. The above values do not include a factor of safety. Appropriate factors of safety should be incorporated into the design. If other surcharge loads are anticipated, SCST should be contacted for the necessary increase in soil pressure.

Retaining walls should be designed to resist hydrostatic pressures or be provided with a backdrain to reduce the accumulation of hydrostatic pressures. The backdrain can consist of a 2-foot-wide zone of ¾-inch crushed rock. The backdrain should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. Weep holes should be provided, or a perforated pipe should be installed at the base of the backdrain and sloped to discharge to a suitable storm drain facility. As an alternative, a geocomposite drainage system such as Miradrain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project architect should provide damp proofing specifications and details. Figure 6 presents typical conventional retaining wall backdrain details.

8.5.8 Seismic Earth Pressure

If required, the seismic earth pressure can be taken as equivalent to the pressure of a fluid weighing 20 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, static active earth pressure. The passive pressure and bearing capacity can be increased by ⅓ in determining the seismic stability of the wall.

8.5.9 Backfill

Wall backfill should consist of granular, free-draining material having a sand equivalent of 20 or more. The backfill zone is defined by a 1:1 plane projected upward from the heel of the wall. Expansive or clayey soil should not be used. Additionally, backfill within 3 feet from the back of the wall should not contain rocks greater than 3 inches in dimension. Backfill should be compacted to at least 90% relative compaction. Backfill should not be placed until walls have achieved adequate structural strength. Compaction of wall backfill will be necessary to minimize settlement of the backfill and overlying settlement sensitive improvements. However, some settlement should still be anticipated. Provisions should be made for some settlement of concrete slabs and pavements supported on backfill. Additionally, utilities supported on backfill should be designed to tolerate differential settlement.

8.6 MECHANICALLY STABILIZED EARTH RETAINING WALLS

The following soil parameters can be used for design of mechanically stabilized earth (MSE) retaining walls.

MSE Wall Design Parameters

Soil Parameter	Reinforced Soil	Retained Soil	Foundation Soil
Internal Friction Angle (degrees)	32°	32°	32°
Cohesion (psf)	0	0	0
Moist Unit Weight (pcf)	130	130	130

The reinforced soil should consist of granular, free-draining material with an expansion index of 20 or less. The bottom of MSE walls should extend to such a depth that a total of 5 feet exists between the bottom of the wall and the face of the slope. Figure 7 presents a typical MSE retaining wall backdrain detail. MSE retaining walls may experience lateral movement over time. The wall engineer should review the configuration of proposed improvements adjacent to the wall and provide measures to help reduce the potential for distress to these improvements from lateral movement.

8.7 PIPELINES

8.7.1 Thrust Blocks

For level ground conditions, a passive earth pressure of 350 psf per foot of depth below the lowest adjacent final grade can be used to compute allowable thrust block resistance. A value of 150 psf per foot should be used below groundwater level, if encountered.

8.7.2 Modulus of Soil Reaction

A modulus of soil reaction (E') of 2,000 psi can be used to evaluate the deflection of buried flexible pipelines. This value assumes that granular bedding material is placed adjacent to the pipe and is compacted to at least 90% relative compaction.

8.7.3 Pipe Bedding

Pipe bedding as specified in the "Greenbook" Standard Specifications for Public Works Construction can be used. Bedding material should consist of clean sand having a sand equivalent not less than 20 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The on-site materials are not expected to meet "Greenbook" bedding specifications. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. Ponding or jetting the pipe bedding should not be allowed.

8.8 SOIL CORROSIVITY

Representative samples of the on-site soil were tested to evaluate corrosion potential. The test results are presented in Appendix II. The project design engineer can use the sulfate results in conjunction with ACI 318 to specify the water/cement ratio, compressive strength, and cementitious material types for concrete exposed to soil. A corrosion engineer should be contacted to provide specific corrosion control recommendations.

8.9 INFILTRATION FEASIBILITY

Figure 2 shows the approximate locations of the three borehole percolation tests we performed to assess stormwater infiltration feasibility. Appendix III presents the field data and test results. The table below presents the observed tested infiltration rates.

Infiltration Rate Test Results

Test Location	Test Depth (feet)	Material Type at Test Depth	Infiltration Rate (inch/hour)
P-1	3	Very Old Paralic Deposits: Silty Sandstone	0.0
P-2	3	Very Old Paralic Deposits: Silty Sandstone	0.0
P-3	3	Very Old Paralic Deposits: Silty Sandstone	0.0

The observed tested infiltration rates do not support stormwater infiltration in an appreciable quantity. Based on our test results, the feasibility screening category is No Infiltration. BMP facilities should be lined with an impermeable geomembrane to reduce the potential for water-related distress to adjacent structures or improvements. A subdrain system should be installed at the bottom of BMP facilities. Foundations should be set back at least 10 feet from BMP facilities, or the foundation should be depended to a depth that extends below the bottom of the BMP.

9. GEOTECHNICAL ENGINEERING DURING CONSTRUCTION

The geotechnical engineer should review project plans and specifications prior to bidding and construction to check that the intent of the recommendations in this report has been incorporated. Observations and tests should be performed during construction. If the conditions encountered during construction differ from those anticipated based on the subsurface exploration program, the presence of the geotechnical engineer during construction will enable an evaluation of the exposed conditions and modifications of the recommendations in this report or development of additional recommendations in a timely manner.

10. CLOSURE

SCST should be advised of changes in the project scope so that the recommendations contained in this report can be evaluated with respect to the revised plans. Changes in recommendations will be verified in writing. The findings in this report are valid as of the date of this report. Changes in the condition of the site can, however, occur with the passage of time, whether they are due to natural processes or work on this or adjacent areas. In addition, changes in the standards of practice and government regulations can occur. Thus, the findings in this report may be invalidated wholly or in part by changes beyond our control. This report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations to site conditions at that time.

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the boring locations and that our data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

11. REFERENCES

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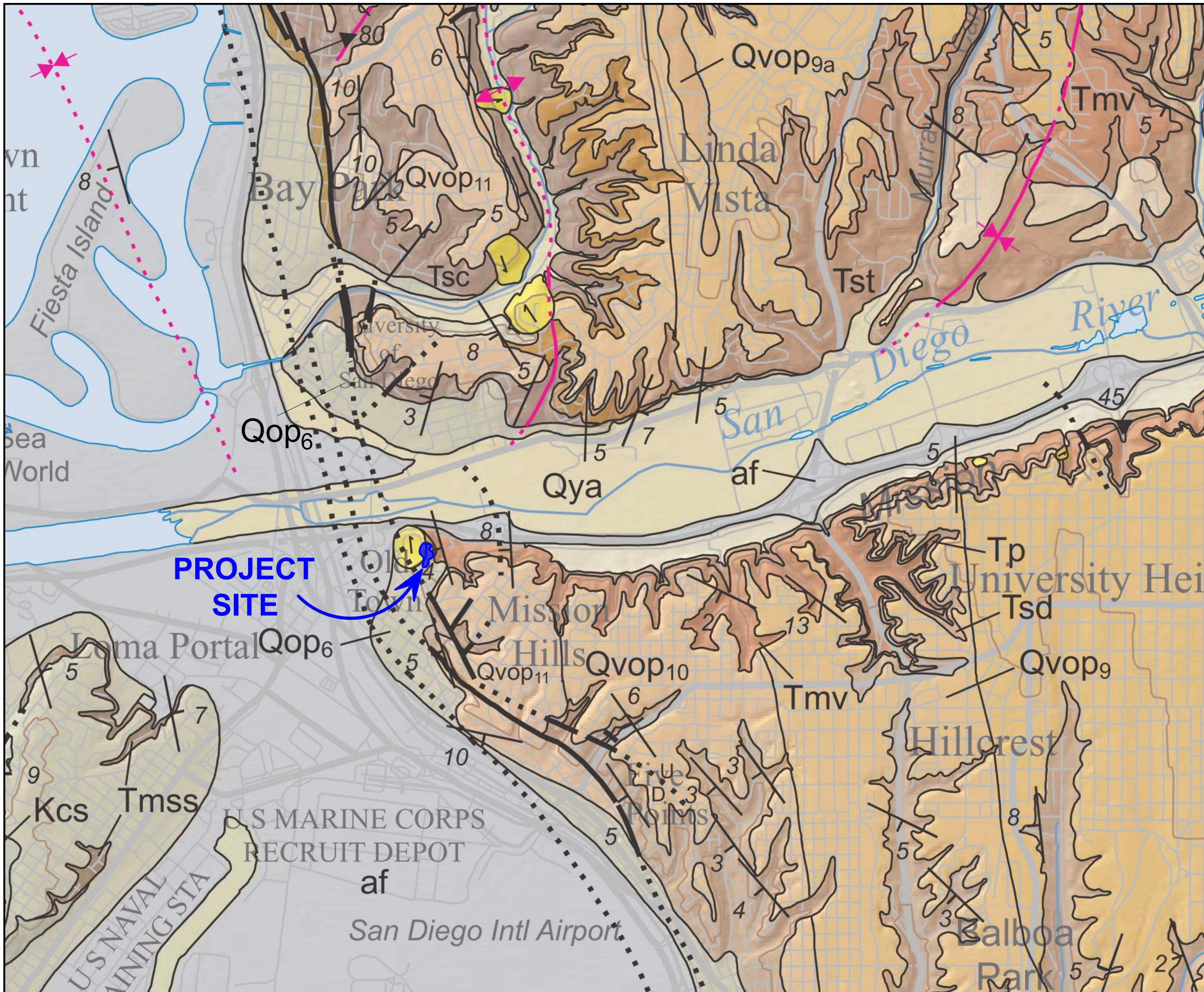


EXPLANATION:

af	Artificial fill
Qya	Young alluvial flood-plain deposits
Qop_6	Old paralic deposits, undivided Unit 6
Qvop	Very old paralic deposits, undivided
Tsd	San Diego Formation, undivided
Tp	Pomerado Conglomerate
Tmv	Mission Valley Formation
Tsc	Scripps Formation
Tmss	Mount Soledad Formation, sandstone
Kcs	Cabrillo Formation, sandstone

Anticline Fold - Solid where well defined; short dash where inferred

Syncline Fold - Solid where well defined; short dash where inferred

4

Strike and **dip** of beds
Inclined

Qls?

Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.

N

0 1.5 3.0
SCALE (miles)

Note: All locations are approximate

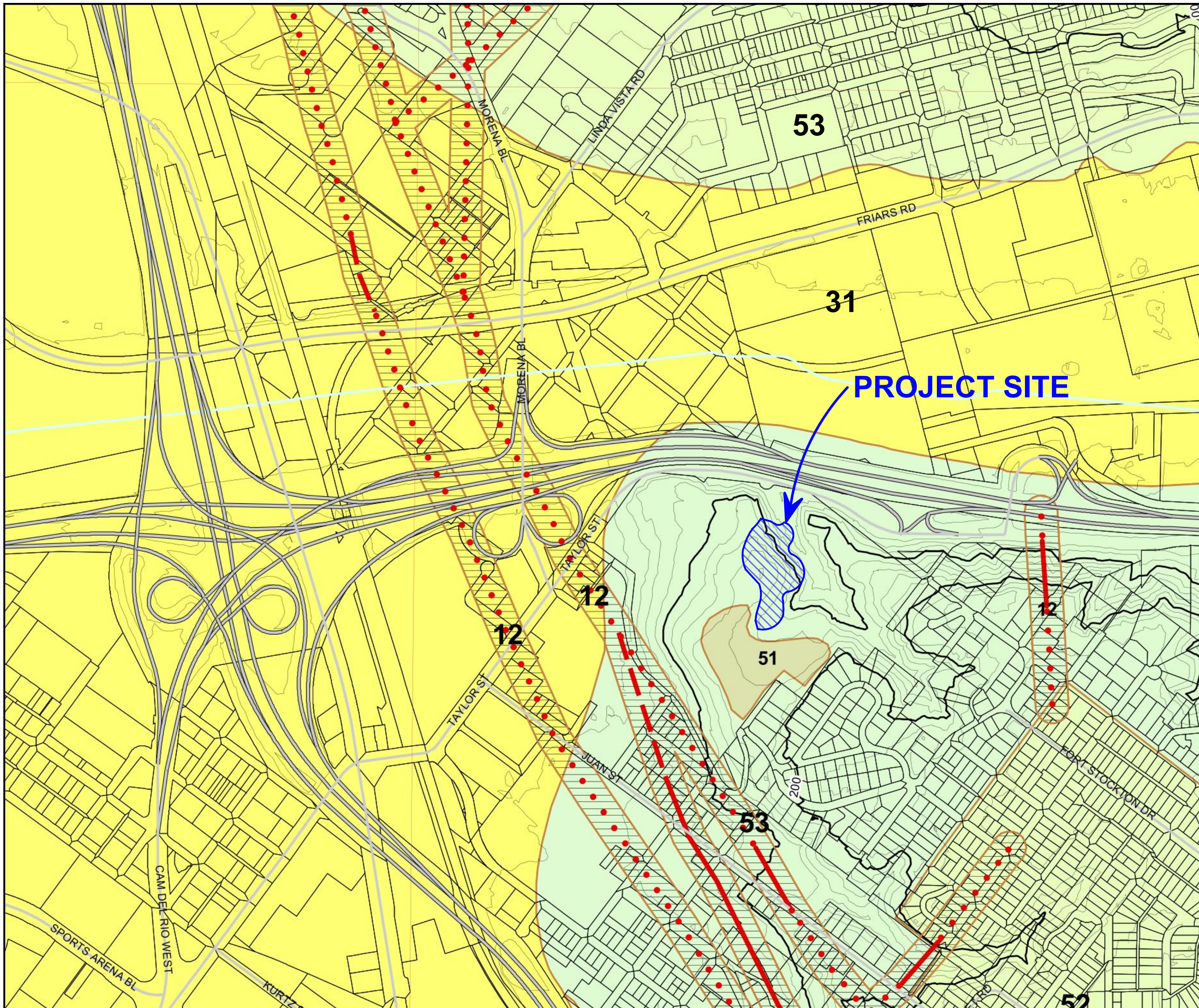
Reference:
Kennedy, M.P. and Tan, S.S. (2008), Geologic Map of the San Diego 30' x 60' Quadrangle, California, California Geological Survey, Scale 1:100,000

SCST, LLC

REGIONAL GEOLOGY MAP
Junipero Serra Museum ADA Improvements
San Diego, California

Date:	March, 2019
By:	NDK/NNW
Job No.:	180320P4.1

Figure: 3



EXPLANATION:

FAULT ZONES
- 11 — Active, Alquist-Priolo Earthquake Fault Zone
- 12 — Potentially Active, Inactive, Presumed Inactive, or Activity Unknown

LIQUEFACTION
- 31 — High Potential -- shallow groundwater; major drainages, hydraulic fills

OTHER TERRAIN
- 51 — Level mesas -- underlain by terrace deposits and bedrock; nominal risk
- 52 — Other level areas, gently sloping to steep terrain, favorable geologic structure; Low risk
- 53 — Level or sloping terrain, unfavorable geologic structure; Low to moderate risk

FAULTS
- Fault
- Inferred Fault
- Concealed Fault

N

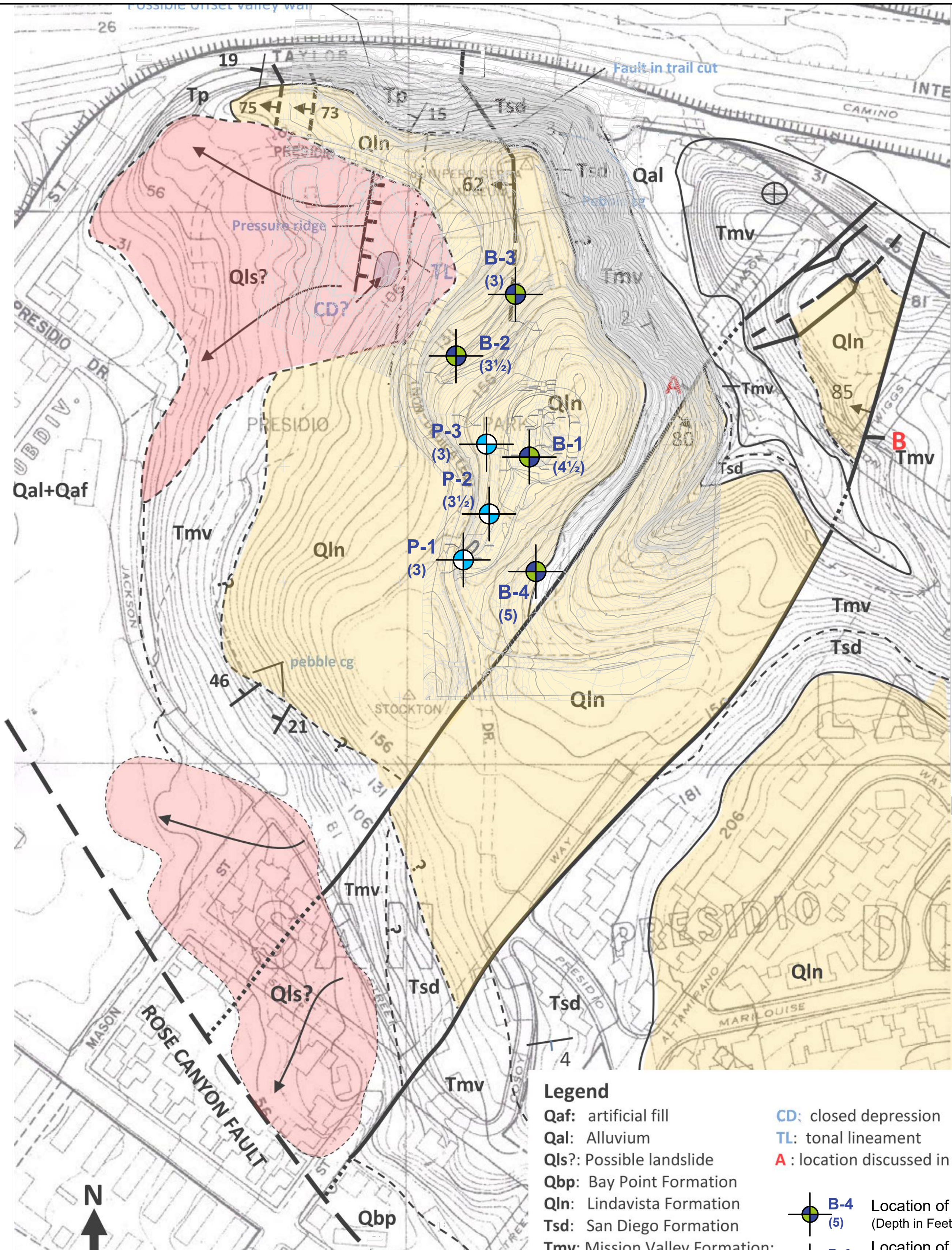
0 1,000 2,000

SCALE (feet)

Note: All locations are approximate

Reference:
City of San Diego (2008), Seismic Safety Study, Geologic Hazards and Faults, Grid Tile: **20**, Development Services Department, April 3, Scale 1:800.

SCST, LLC AN ATLAS COMPANY	**CITY OF SAN DIEGO SEISMIC SAFETY STUDY** Junipero Serra Museum ADA Improvements San Diego, California	Date: March, 2019 By: NDK/NNW Job No.: 180320P4.1	Figure: 4



Legend

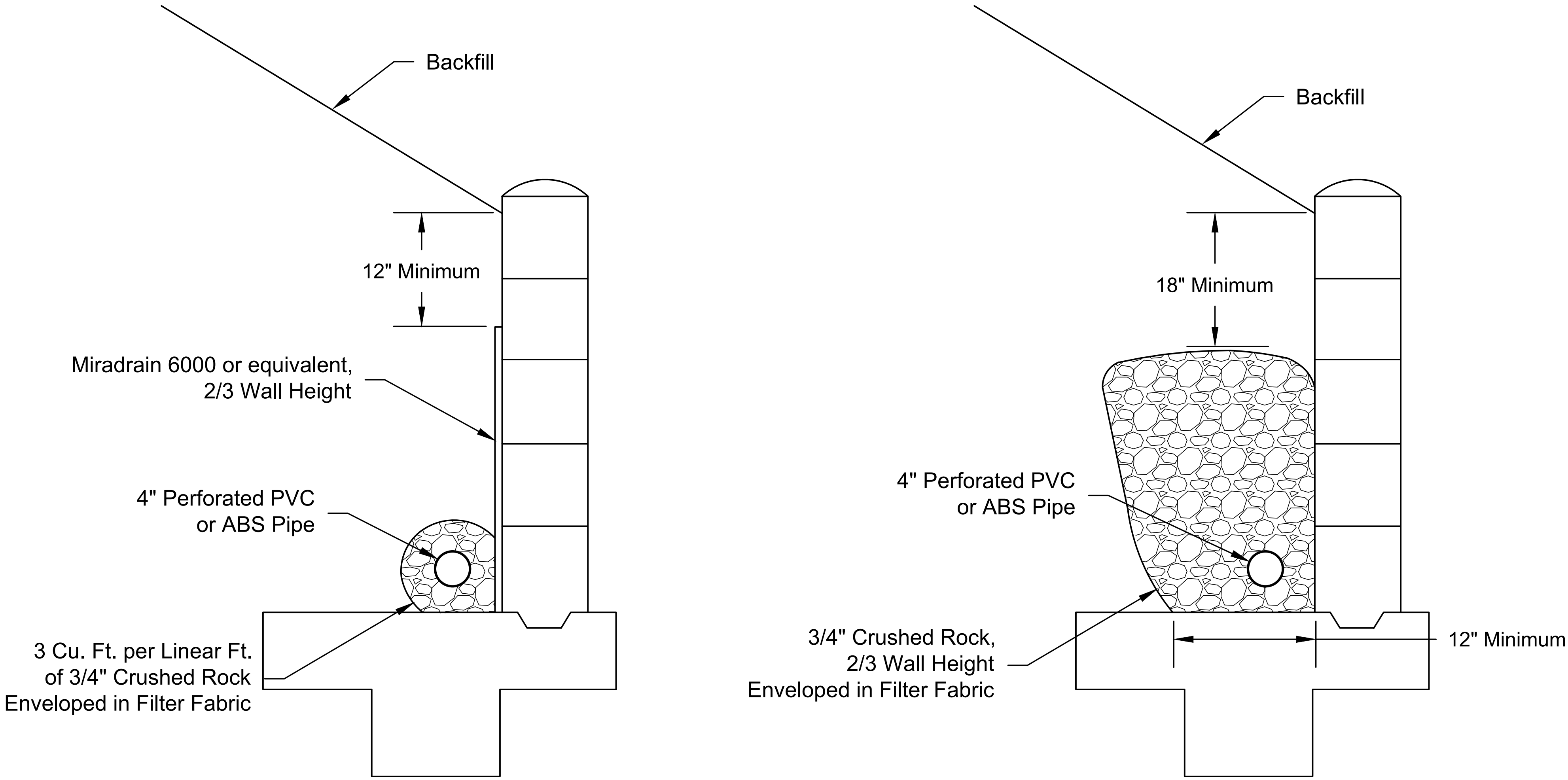
Qaf: artificial fill	**CD:** closed depression
Qal: Alluvium	**TL:** tonal lineament
Qls?: Possible landslide	**A** : location discussed in text
Qbp: Bay Point Formation	
Qln: Lindavista Formation	**B-4** (5) Location of Boring (Depth in Feet)
Tsd: San Diego Formation	
Tmv: Mission Valley Formation;	**P-3** (3) Location of Percolation Test (Depth in Feet)
Tp: Pomerado Conglomerate	

0 250 500

SCALE (feet)

Note: All locations are approximate.
Reference: Hart, MW. Geologic Map Presidio Park, San Diego California, Figure 1

SCST, LLC	**GEOLOGIC MAP** Junipero Serra Museum ADA Improvements San Diego, California	Date: March, 2019 By: DTC Job No.: 180320P4-1	Figure: 5

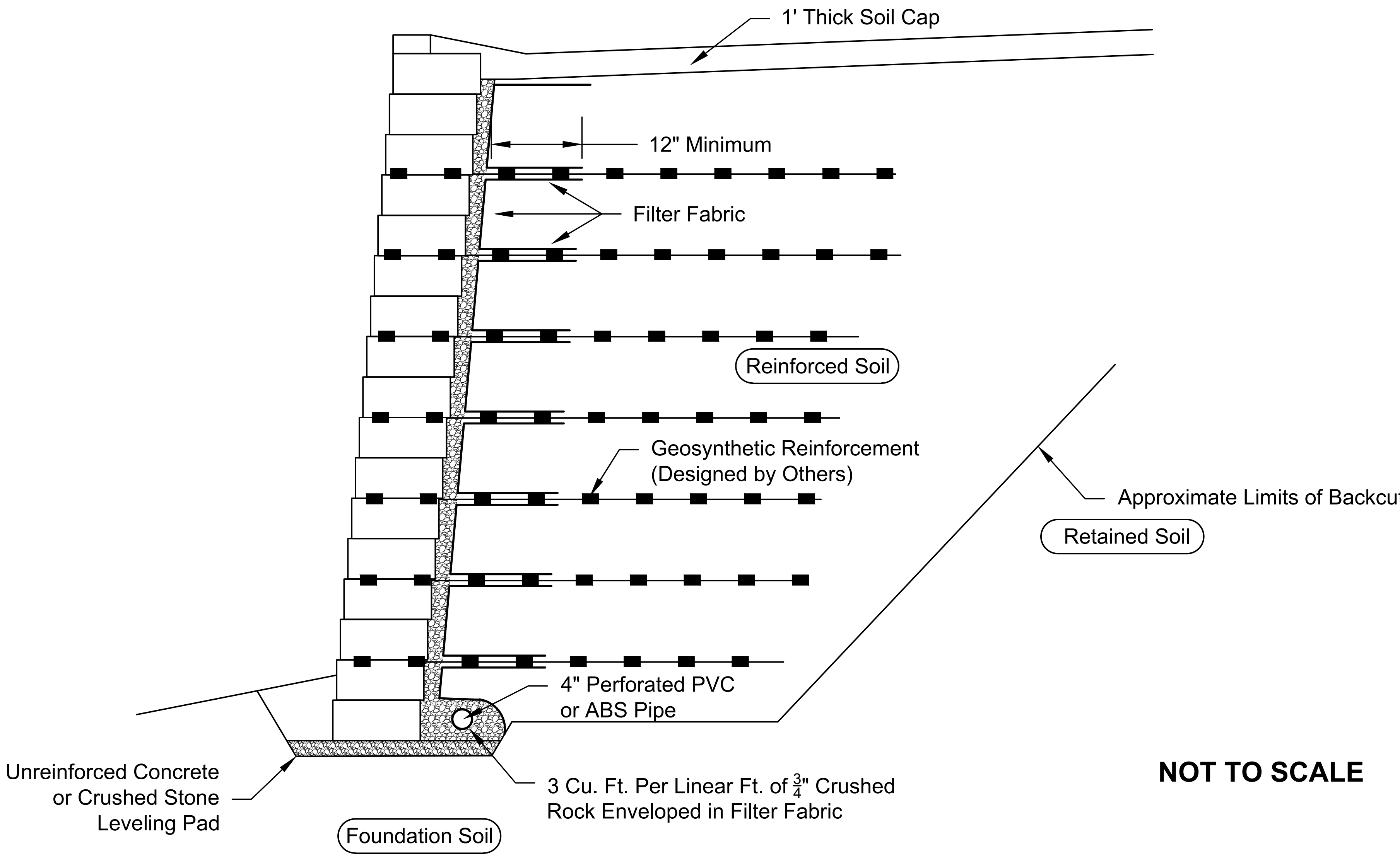


NOT TO SCALE

NOTES:

1) Dampproof or waterproof back of wall following architect's specifications.
2) 4" minimum perforated pipe, SDR35 or equivalent, holes down, 1% fall to outlet. Provide solid outlet pipe at suitable locations.
3) Drain installation and outlet connection should be observed by the geotechnical consultant.

SCST, LLC	**TYPICAL RETAINING WALL BACKDRAIN DETAILS** Junipero Serra Museum ADA Improvements San Diego, California	Date: March, 2019 By: NDK/NNW Job No.: 180320P4.1	Figure: 6



NOTES:

1) Backcut as recommended by the geotechnical report or field evaluation
2) Additional drain at excavation backcut may be recommended base on conditions observed during construction.
3) Filter fabric should be installed between crushed rock and soil. Filter fabric should consist of Mirafi 140N or equivalent. Filter fabric should be overlapped approximately 6 inches.
4) Perforated pipe should outlet through a solid pipe to an appropriate gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

SCST, LLC	**TYPICAL MSE RETAINING WALL DETAIL** Junipero Serra Museum ADA Improvements San Diego, California	Date: March, 2019 By: NDK/NNW Job No.: 180320P4.1	Figure: 7

APPENDIX I
FIELD INVESTIGATION

Our field investigation consisted of a visual reconnaissance of the site and drilling four borings and three percolation tests borings on February 25, 2019 to a depth of about 3 to 5 feet below the existing ground surface using a hand auger. Auger refusal was encountered in all but one of the borings (B-4). Figure 2 presents the approximate locations of the borings. Our field investigation was performed under the observation of an SCST geologist who also logged the borings and obtained samples of the materials encountered. Disturbed bulk samples were obtained from the drill cuttings. The soils are classified in accordance with the Unified Soil Classification System as illustrated on Figure I-1. Logs of the borings are presented in the following Figures I-2 through I-8.

APPENDIX II
LABORATORY TESTING

Laboratory tests were performed to provide geotechnical parameters for engineering analyses. The following tests were performed:

- **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soil Classification System.

- **IN SITU MOISTURE AND DENSITY:** The in situ moisture content and dry unit weight were evaluated on samples collected from the borings. The test results are presenting on the boring logs in Appendix I.

- **PARTICLE-SIZE DISTRIBUTION:** The particle-size distribution was determined on selected soil samples in accordance with ASTM D6913.

- **ATTERBERG LIMITS:** The Atterberg limits were determined on one soil sample in accordance with ASTM D4318.

- **R-VALUE:** R-value tests were performed on selected soil samples in accordance with California Test Method 301.

- **EXPANSION INDEX:** The expansion index was determined on selected soil samples in accordance with ASTM D4829.

- **CORROSIVITY**: Corrosivity tests were performed on selected soil samples. The pH and minimum resistivity were determined in accordance with California Test 643 and ASTM G51. The total chloride ion content was determined in accordance with California Test 422. The soluble sulfate content was determined in accordance with California Test 417.

Soil samples not tested are now stored in our laboratory for future reference and analysis, if needed. Unless notified to the contrary, all samples will be disposed of 30 days from the date of this report.

APPENDIX III
INFILTRATION RATE TEST RESULTS

We performed borehole percolation testing at three locations (P-1 through P-3) in general conformance with the City of San Diego BMP Design Manual. Prior to starting the testing, the test hole was presoaked with clean potable water for about 24 hours. The infiltration test was performed after presoaking by placing clean potable water in the hole and measuring the drop in the water level. Figures III-1 through III-3 present the results of the testing.

APPENDIX IV
FORM I-8A

This appendix is the form from the City of San Diego's BMP Design Manual filled out for this project. It is used to help evaluate infiltration conditions.