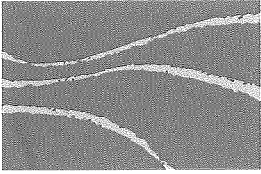


GEOTECHNICAL INVESTIGATION
BERKELEY BICYCLE/PEDESTRIAN BRIDGE
EASTERN TOUCHDOWN PLAZA
AQUATIC PARK
BERKELEY, CALIFORNIA



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July 31, 2007
1258-7, L-27974

Mr. John Hykes
Design Community & Environment
1625 Shattuck Avenue, Suite 300
Berkeley, CA 94709

RE: Geotechnical Investigation
Berkeley Bicycle/Pedestrian Bridge – Eastern Touchdown Plaza
Aquatic Park
Berkeley, California

Dear Mr. Hykes:

This report presents the results of our geotechnical investigation for the Berkeley Bicycle/Pedestrian Bridge – Eastern Touchdown Plaza to be located at the north end of Berkeley's Aquatic Park Lake directly east of the Eastshore Freeway (Highway 80). During the course of our study, we obtained information regarding the proposed project from you and Jerome de Verrier of TranSystems, the project Civil Engineer.

1.00 PROJECT DESCRIPTION

As shown on the attached Figures 1 and 2, the project site is at the north end of Berkeley's Aquatic Park Lake directly east of Highway 80 near the intersection of Addison Street and Bolivar Drive. We understand the project will include the following elements:

- A level circular lakeside plaza area about 24 feet in diameter at an approximate elevation of +3.0 feet;
- An approximately 120-foot-long boardwalk path leading from the foot of Addison Street to the plaza and ranging in elevation from about +1.58 feet to +3.0 feet;
- Improvements to landscaping including a low (approximately 30-inch high) retaining wall near the Aquatic Park Lake that will not support or be connected to any of the new improvements; and
- Resurfacing of the existing parking lots to the north of the future plaza.

The circular plaza and boardwalk will be located along the northern lake margin in an area currently covered in vegetation consisting of grasses and small trees. As currently envisioned, the southern edges of the plaza and boardwalk will extend out toward the lake and will be supported on a driven pile foundation.

2.00 PURPOSE AND SCOPE

The purpose of our investigation was to characterize the subsurface conditions at the site and provide geotechnical conclusions and recommendations for the design of the lakeside plaza and boardwalk. As outlined in our November 3, 2006 proposal, the scope of our Geotechnical Investigation included:

- Reviewing existing information relevant to the site;
- Conducting a geotechnical reconnaissance of the site vicinity;
- Drilling two, approximately 30-foot-deep, geotechnical test borings;
- Performing geotechnical laboratory tests on samples obtained from the test borings; and
- Preparing this report presenting design-level conclusions and recommendations for the project.

Our proposed project approach, also outlined in our November 3, 2006 proposal, focused on collecting only the site-specific data necessary to develop the geotechnical engineering recommendations for the foundations of the plaza and boardwalk. Please note that the scope of our services did not include: (1) collecting data to substantiate an opinion on the presence or absence of potentially liquefiable materials; (2) an analysis of liquefaction potential or effects; (3) recommendations pertaining to the resurfacing of the existing paved parking lots or the construction of low landscape-type retaining walls; or (4) an environmental assessment or investigation for the presence of hazardous, toxic, or corrosive materials on, below, or around the site.

3.00 METHODS OF INVESTIGATION

3.01 Review of Existing Data

A variety of published sources were reviewed to evaluate geotechnical data relevant to the subject site. These sources included geotechnical literature, reports, and maps published by various public agencies including, but not limited to, topographic, geologic, liquefaction, seismic hazard zones, and fault zone maps prepared by the United States Geological Survey and by the California Geological Survey (formerly the California Division of Mines and Geology). We also reviewed geotechnical data contained in an unpublished report by Geomatrix Consultants (1999) for the nearby Eastshore Pedestrian Overcrossing. Information obtained from the map and literature review is discussed in Section 4.01, Geologic Setting, and a complete list of the sources used in our investigation is presented at the end of this report.

3.02 Site Reconnaissance Visits

A site reconnaissance visit was performed on March 26, 2007. This visit was intended to make observations of the surficial conditions present and to note whether any obvious geotechnical concerns were exposed. Observations made during our site reconnaissance are discussed in Section 4.02, Surface Conditions.

3.03 Subsurface Exploration

Our subsurface exploration program was performed on March 26, 2007, to investigate and sample the subsurface materials. Two borings were drilled at the site to a depth of 31.5 feet at the locations shown on the Site Plan and Boring Location Map, Figure 2. Boring 1 was located about 23 feet east of the proposed plaza location and Boring 2 was located at the southeastern end of the proposed boardwalk near the foot of Addison Street. The logs of the borings are attached in Appendix A.

The test borings were drilled using a truck-mounted drill rig with a hollow stem auger. During drilling, our staff engineer logged the borings, directed the drilling, and visually classified the soil in general accordance with ASTM D2488 classifications which are based on the Unified Soil Classification System (USCS). The USCS is described on the Key to Exploratory Boring Logs, Figure A1. Soil samples were obtained using a 2-inch outside diameter (O.D.) Standard Penetration Test (SPT) sampler without liners, a 3-inch O.D. California Modified sampler with liners, and a thin-walled, 3-inch O.D. Shelby Tube. The samplers were driven with a

standard automatic-trip, 140-pound hammer falling 30 inches. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the boring logs. The Shelby Tubes were advanced hydraulically. Following the field operations, the borings were grouted in accordance with the City of Berkeley permit regulations.

We note that the attached boring logs and related information depict subsurface conditions only at the approximate locations shown on the Site Plan on the particular date designated on the boring logs. The attached boring logs represent our current interpretation of the subsurface materials at the boring locations and the passage of time may result in changes in the subsurface conditions. The boring locations indicated on the attached materials were determined by measuring from existing curbs and walls and should be considered approximate.

A summary of our findings from our subsurface exploration can be found in Section 4.03, Subsurface Conditions.

3.04 Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical properties of the soils that underlie the site. The following geotechnical laboratory tests were performed:

- Water content per ASTM Test Designation D-2216;
- Dry density per ASTM Test Designation D-2937;
- Atterberg Limits per ASTM Test Designation D-4318; and
- Percent minus #200 sieve per ASTM Test Designation D1140.

The results of the tests are presented on the boring logs at the corresponding sample depths.

4.00 SITE CONDITIONS

4.01 Geologic Setting

The topographic map for this area (the Oakland West Quadrangle) prepared by the United States Geological Survey (USGS) shows the site is located on a relatively level plain adjacent to the coastline of the San Francisco Bay. The site is shown at an approximate elevation of +5.0 feet MLLW (mean lower low water). The surrounding areas are generally level with grades gradually rising toward the base of the Berkeley Hills, which is about 2.5 miles east of the site. The 1878 Historic Atlas of Alameda County (Thompson and West, 1878) shows the original (circa 1850) shoreline of the San Francisco Bay just to the west of 2nd Street about one-half block east of the site.

The site is mapped as being underlain by artificial fill (Graymer, 2000) and artificial fill over San Francisco Bay Mud (Knudsen, et al., 2000). The artificial fill is described as man-made deposits of various materials and ages.

The site is approximately 4.0 kilometers (km) southwest of the Hayward fault, 23.7 km west of the Calaveras fault, 25.5 km northeast of the San Andreas fault, and 26.5 km southwest of the Concord-Green Valley fault (ICBO/California Department of Conservation Division of Mines and Geology, 1998). The site is not located within an Alquist-Priolo Earthquake Fault Zone designated by the State of California (CDMG, 1982).

Studies by the United States Geological Survey's Working Group on California Earthquake Probabilities have estimated a 62 percent probability of at least one magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region before the year 2031 (WGCEP, 2003). As part of their prediction, they estimated the probability to be 27 percent for a magnitude 6.7 or greater earthquake to occur on the Hayward/Rodgers Creek fault by the year 2031.

In 2003, the California Geological Survey released a Seismic Hazard Zone map for the Oakland West Quadrangle identifying areas that may be prone to earthquake-induced ground failure during a major earthquake. The map shows the site located within a zone that may be subject to earthquake-induced liquefaction (CGS, 2003).

4.02 Surface Conditions

As shown on the Site Plan and Boring Location Map, Figure 2, the site is located on the shoreline of Aquatic Park Lake. The site is surrounded by an asphalt concrete walkway/bike path to the north and wetlands/shoreline to the south. We observed the general condition of the streets and pavements in the general vicinity of the site. The existing surface improvements that we observed were in fair to good condition and showed no obvious indications of major distress.

4.03 Subsurface Conditions

The conditions encountered in the borings are briefly summarized below; the two-letter soil classifications in parentheses are based on the Unified Soil Classification System (USCS).

Fill -- Both borings encountered between 6 inches and 1 foot of soft, brown, silty topsoil (ML) over about 8 feet of soft to stiff clay and silt logged as fill or possible fill. The fill in Boring 1 consisted of about 4.5 feet of soft to stiff, moderately plastic, gravelly clay (CL) over about 1.75 feet of soft, blue gray, sandy clay/silt (CL/ML) over about 1.5 feet of firm to stiff, bluish black, fat clay (CH). The fill in Boring 2 similarly consisted of about 5.75 feet of soft to stiff, moderately plastic, gravelly clay (CL) over about 2.25 feet of soft, bluish black to black, sandy silt (ML).

Natural Soils -- Below the fill, both borings encountered layered deposits of natural soils which extended to the bottom of the borings at 31.5 feet. The natural soils encountered in both borings consisted mostly of firm to very stiff clays containing varying amounts of silt, sand and gravel. Boring 2 encountered two layers of granular material: a medium dense, silty sand (SM) lens between 16.75 and 18.5 feet and a medium dense, silty gravel/sand (GM/SM) layer between 20.75 feet and 24 feet.

Although a liquefaction analysis was beyond the scope of this project, we should mention that there were some layers of soil encountered that may be susceptible to earthquake-induced soil liquefaction. A more thorough discussion of these materials can be found in Section 5.05.3, Liquefaction.

Groundwater was encountered in Boring 1 at about 20 feet below the ground surface at the time of drilling and was measured at 19 feet immediately after drilling. Groundwater was encountered in Boring 2 at about 20 feet below the ground surface at the time of drilling and was measured at 3.6 feet 24 minutes after drilling was complete. Both borings were grouted shortly after drilling in accordance with the City of Berkeley permit requirements. We note, however, that the borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions and that fluctuations in the groundwater level likely occur due to seasonal variations in rainfall and other factors. Due to the close proximity of Aquatic Park Lake and the San Francisco Bay, we judge the groundwater may at times be relatively close to the ground surface.

Detailed descriptions of the materials encountered in the borings are found on the boring logs presented in Appendix A. A Key to Exploratory Boring Logs, Figure A1, is also presented. The attached logs and related information depict subsurface conditions only at the specific locations shown on the Site Plan and on the particular date designated on the logs. These logs may have been modified from the original logs recorded during drilling as a result of further study of the collected samples, laboratory tests or other efforts. Also, the passage of time may result in changes in the subsurface conditions due to environmental changes.

5.00 EVALUATION AND CONCLUSIONS

5.01 General

In general, the site is within an area that may experience ground failure as a result of strong earthquake shaking. This potential hazard will not be mitigated as part of the proposed project. Under non-earthquake conditions, the primary considerations for the geotechnical design at the site are:

- The presence of uncompacted fill and soft bay deposits, up to 9 feet deep;
- Moderately expansive near-surface soils; and
- Shallow groundwater.

5.02 Uncompacted Fill and Soft Bay Deposits

The near-surface soils at the site consisted of fill which is generally not suitable to support the proposed improvements. From a cost perspective, it may be preferred to support the boardwalk and plaza, as much as possible, on shallow foundations supported on an engineered fill pad at least 18 inches thick. This will depend, however, on the exact locations and elevations of the planned improvements in relation to the existing shoreline and water levels. It is our understanding that the elevations and exact locations of the plaza and boardwalk have not yet been determined. However, after conversations with you and Jerome de Verrier of TranSystems, the project Civil Engineer, we understand the current plan is to support the southern sides of the plaza and boardwalk on a deep foundation system due to the close proximity to the lake and to support the northern sides on shallow foundations on an engineered fill pad. Foundation recommendations can be found in Section 6.03, Foundations.

5.03 Moderately Expansive Near-Surface Soils

Near-surface soils at the site were found to be moderately expansive with plasticity indexes (PIs) of 20 and 26 at depths between 1.5 and 2 feet. Expansive soils shrink and swell with changes in moisture and have the potential to damage improvements that are supported upon them. We recommend removing and replacing the soils that underlie planned, near-surface improvements (e.g., slabs and footings) with non-expansive fill to mitigate the potential for future damage related to soil expansion. Recommendations for the non-expansive fill pad can be found in Section 6.01, Site Preparation and Earthwork.

In order to minimize the effects of expansive soils in areas with pile foundations (i.e., the southern edges of the plaza and boardwalk) we recommend that either a 4-inch void be created at the bottom of the pile cap or that the pile cap be designed to resist an uplift pressure.

5.04 Shallow Groundwater

During our investigation, groundwater was measured at 3.6 feet below the existing ground surface in Boring 2, 24 minutes after drilling was complete. We caution that this may not represent a stabilized groundwater level and that water levels at the site likely vary as a result of tidal fluctuations, rainfall and groundwater flow conditions. For preliminary design, we suggest assuming that groundwater may be present at an elevation of one foot above the maximum tidal elevation within the adjacent lake. If shallow foundations are to be used, we recommend carefully choosing the elevations and locations of planned improvements in order to avoid excavations that extend below groundwater.

5.05 Earthquake Hazards

5.05.1 Strong Ground Shaking and Seismic Performance

The site is likely to experience one or more episodes of strong ground shaking during the economic life of the project. The California Building Code (CBC) has adopted provisions for incorporation of strong ground shaking into the design of all structures. Our recommendations for geotechnical parameters to be used in the structural seismic design of the structure are presented in Section 6.05, Building Code Seismic Design Parameters.

We met with DC&E on April 5, 2007, to discuss the seismic performance requirements of the project. At the meeting we summarized the existing ground conditions (i.e., the presence of shallow liquefiable soils, uncompacted fill, and soft bay deposits) and explained that without ground modification, the proposed improvements may perform poorly during a strong seismic event at the site. However, since the planned Aquatic Park Plaza improvements do not include structures designed for human occupancy and the project has a limited budget, it was concluded that taking the most cost effective approach to designing the improvements would be appropriate. It is understood that in taking this approach, the improvements may not be functional after a strong seismic event at the site.

5.05.2 Fault Rupture

As noted earlier, the subject site is located in the highly seismic San Francisco Bay Area, and there is a strong probability that a moderate to severe earthquake will occur during the life of the structure. The site is not mapped in the immediate proximity of any known active faults (CDMG, 1982 and Jennings, 1994); therefore, the likelihood of fault rupture is considered to be remote.

5.05.3 Liquefaction

The site is mapped within a Seismic Hazard (Liquefaction) Zone by the State of California Geologic Survey (CGS). Liquefaction is a phenomenon whereby certain types of soils below groundwater lose strength in response to earthquake shaking. As noted earlier, a liquefaction evaluation was beyond the scope of this project; however, we should mention that there were some layers of soil encountered that may be susceptible to earthquake-induced soil liquefaction.

The soft sandy clay/silt (CL/ML) between 5.25 and 7 feet in Boring 1 and the soft sandy silt (ML) between 6.75 and 9 feet in Boring 2 appear to be potentially liquefiable with low blow counts and low plasticity. The granular layers (SM, GM/SM) in Boring 2 (between 16.75 and 18.5 feet and 20.75 and 24 feet, respectively) may also be potentially liquefiable. Because the granular layers are somewhat dense, additional analyses would be required to determine if they would liquefy during a strong seismic event.

Resulting effects of liquefaction include settlement, bearing capacity failure, sand boils, and lateral spreading. Any of these effects, if severe enough, have the potential to cause damage to site improvements. In our judgment, all of these effects are possible at the project site; and as previously stated, the proposed improvements may not be functional after a strong seismic event.

6.00 RECOMMENDATIONS

6.01 Site Preparation and Earthwork

6.01.1 Clearing and Site Preparation

Prior to the start of work, the contractor should locate and mark all active subsurface utilities in the general vicinity of the site. The contractor should protect all utilities that are to remain in and surrounding the site during onsite excavation and construction activities.

The site should then be cleared of pavements, sidewalks, aggregate base, and abandoned utilities. These materials should be removed from the site or stockpiled for reuse if approved by the owner in consultation with our firm.

6.01.2 Over-Excavation for Non-Expansive Fill Pad

For stability and to help mitigate the effects of expansive soils, we recommend constructing an engineered, non-expansive, fill pad beneath all at-grade improvements. We recommend that the fill pad extend at least 18 inches below and at least 18 inches beyond the perimeter of any at-grade improvements. This includes footings and concrete flatwork. To maintain edge stability, we recommend that the bottom of the fill pad be at least 5 feet, horizontally, from the adjacent shoreline, unless the outboard edge of the fill pad is retained by a wall (or other engineered means).

Extreme care should be exercised during excavation for the non-expansive fill layer to avoid disturbing the underlying soils, which may be weak and/or saturated. We recommend that a representative of our firm (AKA) observe during excavation to check that subgrade soils beneath the non-expansive fill layer are not disturbed and to check for adequate bearing material. Locally soft, yielding or otherwise unsuitable subgrade soils may need to be overexcavated or otherwise remediated in order to construct a stable subgrade. The bottom of excavated areas should be compacted to create a firm and stable subgrade and the subgrade should be confirmed by AKA to be firm and non-yielding before the fill placement operations begin.

6.01.3 Material for Fill

All proposed fill materials should be approved by our firm prior to their use. The different types of fill material are described below:

General Fill – General fill is on-site material that has an organic content of less than 3 percent by volume and does not contain rocks or lumps larger than 6 inches in greatest dimension. General fill can be used anywhere except for areas that require non-expansive fill.

Non-Expansive Fill – Non-expansive fill should conform to the requirements for general fill, have a Plasticity Index (PI) no greater than 15, and have a liquid limit (LL) no greater than 40. A layer of non-expansive fill is required beneath all at-grade improvements. See Section 6.01.2, Over-Excavation for Non-Expansive Fill Pad for more information.

Imported Fill – Import material should conform to the requirements for non-expansive fill and should be evaluated by our firm prior to its importation to the site.

Some of the materials cleared or excavated from the site may be suitable for re-use as general fill or non-expansive fill, from a geotechnical standpoint, if they can be processed to meet the above requirements. Material that cannot be mixed or processed to meet specification requirements should be disposed of offsite or stockpiled for other uses at the discretion of the owner. If the re-use of aggregate base or crushed asphalt is to be considered, it must first be approved by the owner in consultation with our firm.

6.01.4 Fill Placement and Subgrade Preparation

All fill should be spread in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned, as necessary, and compacted to at least 90 percent relative compaction based on the ASTM D-1557 test method (latest version). In pavement and slab-on-grade areas, the upper 6 inches of subgrade should be compacted to at least 95 percent relative compaction (per ASTM D-1557). AKA should observe proofrolling of pavement and slab-on-grade subgrades to confirm that they are firm and non-yielding. Aggregate base underneath flexible pavement should be compacted to at least 95 percent relative compaction.

It is possible that fill and/or subgrade soils may be excessively wet or dry depending on the moisture content at the time of construction. If the fill soils are too wet, they may be dried by aeration or by mixing with drier materials.

6.02 Surface Drainage

Positive surface drainage should be provided to direct surface water away from foundations of the proposed improvements. Ponding of surface water should not be allowed in any areas adjacent to the improvements.

6.03 Foundations

6.03.1 Shallow Foundations

The bearing capacity of footings supported directly on a properly-engineered, non-expansive fill pad can be evaluated using the following values:

Load Case	Allowable Bearing Pressure (psf)
Allowable dead loads (FS=3.0)	1500
Allowable dead plus live loads (FS=2.0)	2250
Allowable total loads, including seismic (FS=1.5)	3000

Note: FS=Factor of Safety

Footings should be underlain by at least 18 inches of compacted, non-expansive fill (Refer to Section 6.01, Site Preparation and Earthwork) and should be embedded at least 16 inches below the lowest adjacent finished grade. Completed footing excavations should be maintained in a moist condition until concrete is placed. All footing excavations should be free of water and loose material at the time the footings are cast. We recommend that the footing excavations be monitored by a representative of AKA to confirm the adequacy of the bearing materials, verify compliance with appropriate moisture control, and check for adequate cleanout prior to concrete placement.

Lateral load resistance may be developed in friction between the foundation bottoms and the supporting subsurface materials and by the passive resistance of soil acting on the vertical faces of below-grade structural elements. A friction coefficient of 0.30 is considered applicable. Passive resistance can be evaluated using an equivalent fluid weight of 300 pounds per cubic foot starting at a depth of one foot below lowest adjacent grade. Where passive resistance is required, foundation elements should be embedded at least 16 inches below the lowest adjacent finished grade. If the foundations are poured neat against the soil, friction and passive pressure may be used in combination.

6.03.2 Deep Foundations

Deep foundations may be needed where new improvements are to extend near or above the Aquatic Park Lake shoreline. We understand driven piles (either steel or concrete) have been chosen by the design team to support the southern edges of the plaza and boardwalk. Piles should be designed to resist the applied vertical loads through skin friction in the firm to stiff, native clayey soils encountered below about 9 feet. We recommend that piles extend at least 10 feet into firm native soils, regardless of load. Minimum pile lengths should be 20 feet.

Concrete piles should be at least 12 inches square, and steel pipe piles should have a minimum outside diameter of 12 inches and should be filled with concrete. Piles should be spaced no closer than three times the pile diameter center-to-center. Piles at closer spacing will have a reduced compressive capacity caused by group effects.

To determine whether the minimum depths are adequate to carry the structural loads of the proposed improvements, the following allowable (factored) skin friction values can be used either for concrete or steel piles:

In the native soils below about 9 feet, use allowable (factored) friction values of 450 pounds per square foot (psf) for dead plus live loads (factor of safety ≈ 2) and 600 psf for all loads, including wind or seismic (factor of safety ≈ 1.5). Skin friction in the fill material above 9 feet should be neglected when calculating vertical capacity.

In order to minimize the effects of expansive soils in areas with pile foundations (i.e., the southern edges of the plaza and boardwalk which may not be supported on a layer of non-expansive fill) we recommend that either a 4-inch void be created at the bottom of the pile cap or that the pile cap be designed to resist an ultimate (non-factored) uplift pressure of 2000 psf.

Piles should be installed under the full-time observation of a representative from our office. All piles should be driven continuously to achieve the indicated penetration to support their design loads. Pre-drilling should not be used. The contractor is responsible for selecting the pile hammer, equipment, and installation method best suited for the type of pile selected for the project. Our firm (AKA) should review the contractor's proposed equipment and installation methods prior to mobilization. Once the pile hammer, pile type, and design loads are known, AKA will develop recommended pile acceptance criteria (e.g. blows per foot) and construction monitoring protocols.

6.04 Exterior Slabs-on-Grade and Flatwork

Exterior slabs-on-grade and flatwork can be supported directly on an 18-inch-thick layer of non-expansive fill placed in accordance with the recommendations outlined in Section 6.01, Site Preparation and Earthwork.

6.05 Building Code Seismic Design Parameters

Based on our review of the site geology and the 2001 California Building Code (CBC), we recommend an S_D soil profile be used for seismic design of the structure. The nearest active fault is the Hayward fault, located about 4.0 km to the northeast. It is a Type A fault as identified in Table 16-U of the 2001 CBC. The site is located within Seismic Zone 4, as determined from Figure 16-2 of the 2001 CBC. We recommend near-source factors of $N_A=1.30$ and $N_V=1.73$.

6.06 Utility Trenches

Utility trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Trenches should be filled by placing a granular shading layer beneath and around the pipe, and then 6 to 12 inches of shading should be carefully placed and tamped above the pipe. The remaining portion of the trench should be backfilled with onsite or import soil. The backfill (above shading layers) should be placed and compacted to a minimum relative degree of compaction of 90 percent based on ASTM D1557. The compaction requirements given above should be considered minimum recommended requirements. If City and/or utility company specifications require more stringent backfill requirements, those specifications should be followed.

If imported granular soil is used, sufficient water should be added during the trench backfilling operations to prevent the soil from "bulking" during compaction. All compaction operations should be performed by mechanical means only. We recommend against jetting.

We recommend the contractor carefully evaluate the stability of all trenches and use temporary shoring, where appropriate. The design and installation of the temporary shoring should be wholly the responsibility of the contractor. In addition, all state and local regulations governing safety around such excavations should be carefully followed.

6.07 Plan Review

We recommend our firm be provided the opportunity for a general review of the geotechnical aspects of the final plans and specifications for this project in order that the geotechnical recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

6.08 Construction-Phase Services

The analyses and recommendations submitted in this report are based in part upon the data obtained from the two soil borings. The nature and extent of variations across the site may not become evident until construction. If variations then become apparent, it will be necessary to re-examine the recommendations of this report.

AKA should review all submittals from the contractor that are geotechnical in nature, before geotechnical materials are delivered or equipment is mobilized to the site.

We recommend our firm be retained to provide geotechnical engineering services during the construction of the proposed project. This is to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. Specifically, we recommend that a representative of our firm observe the following aspects of the construction:

- Earthwork: site clearing and debris removal, excavations, subgrade preparation fill pad, compaction operations (including retaining wall or trench backfill), as applicable;
- Foundations: footing excavations, pile driving; and
- Drainage: area drains, positive surface gradients, discharge location.

We request that the client or the client's representative (the contractor) contact our firm at least two working days prior to the commencement of any of the above operations.

6.09 Wet Weather Construction

Although it is possible for construction to proceed during or immediately following the wet winter months, a number of geotechnical problems may occur which may increase costs and cause project delays. The water content of onsite soils may increase during the winter and rise significantly above optimum moisture content for compaction of subgrade or backfill materials. If this occurs, the contractor may be unable to achieve the recommended levels of compaction without using special measures and would likely have to wait until the materials are dry enough to become workable, or dispose of the wet soils and import dry soils.

If excavations are left open during winter rains, caving of the walls may occur. Also, if excavations fill with water during construction or if saturated materials are encountered at the bottom of the excavation, the excavations may need to be dewatered and remedial work may be needed to provide an appropriate subgrade for foundations and/or non-expansive fill materials.

In general, we note that it has also been our experience that increased clean-up costs may be incurred, and greater safety hazards may exist, if the work proceeds during the wet winter months.

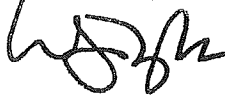
7.00 LIMITATIONS

This report has been prepared for the exclusive use of you and your consultants in accordance with generally accepted geotechnical engineering practices for specific application to the construction of the proposed Berkeley Bicycle/Pedestrian Bridge – Eastern Touchdown Plaza Project in Berkeley, California. No other warranty, either expressed or implied, is made. In the event the nature, design, or location of the improvements differs significantly from what has been noted above, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after three years without being reviewed by this office.

If you have any questions concerning this report, please call us.

Very truly yours,



Wayne Magnusen, G.E.
Associate Engineer

DKM/WM/sa

Copies: Addressee (4)
TranSystems (1)
Attn: Jerome de Verrier



Dona Mann, C.E.
Project Engineer

Berkeley Pedestrian Bridge Report (dkm+wdm)

REFERENCES

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Source: 2004 Thomas Brothers Maps



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VICINITY MAP

Berkeley Bicycle/Pedestrian Bridge - Eastern Touchdown Plaza
Berkeley, California

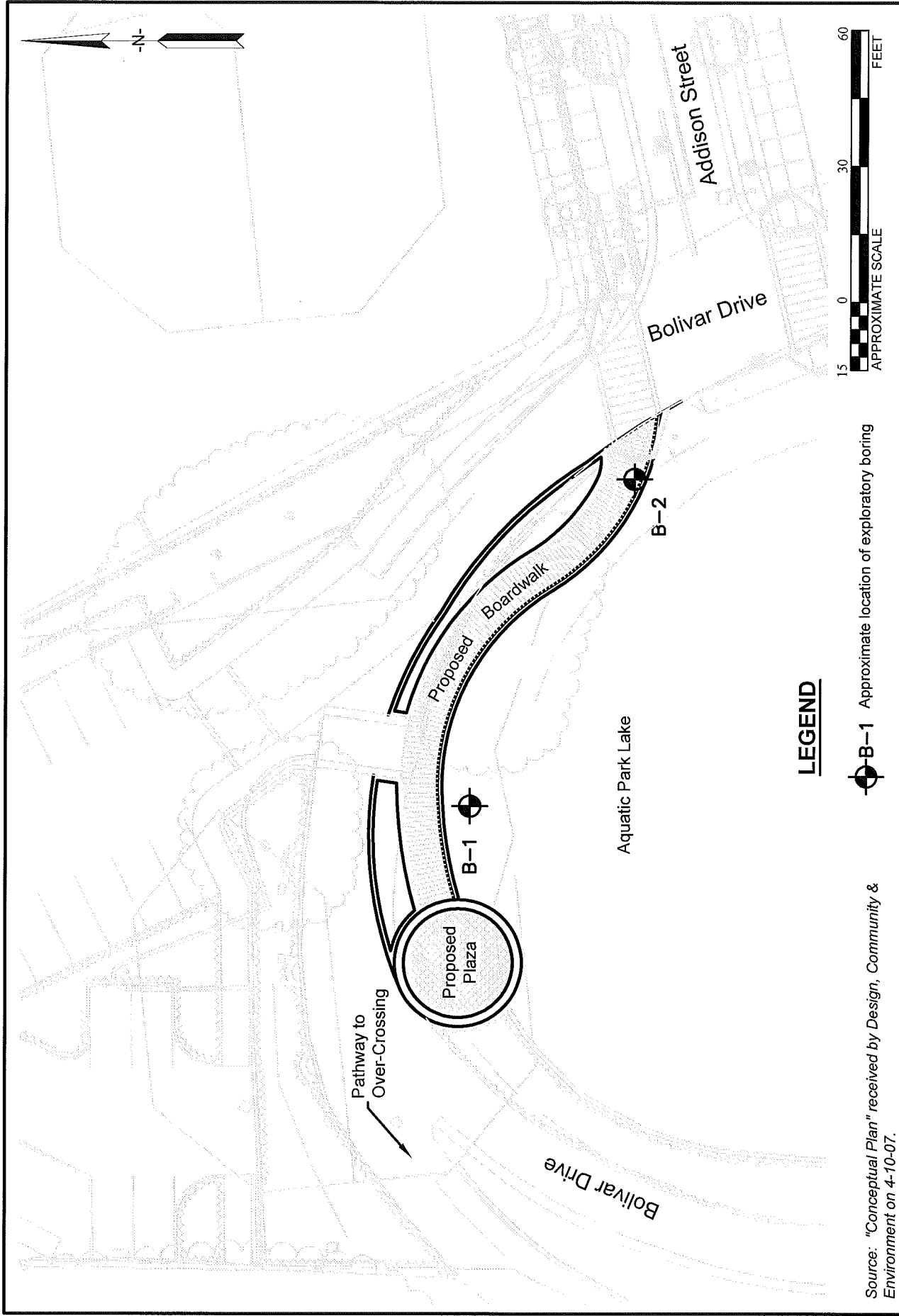
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FIGURE 1



LEGEND

 B-1 Approximate location of exploratory boring

Source: "Conceptual Plan" received by Design, Community & Environment on 4-10-07.

 ALAN KROPP & ASSOCIATES <i>Geotechnical Consultants</i>		SITE PLAN AND BORING LOCATION MAP Berkeley Bicycle/Pedestrian Bridge - Eastern Touchdown Plaza Berkeley, California	
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			FIGURE 2

APPENDIX A

Boring Logs

SOIL CLASSIFICATION CHART

PRIMARY DIVISIONS			SECONDARY DIVISIONS		
			CRITERIA *	GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	$C_u \geq 4$ AND $1 \leq C_c \leq 3^A$	GW	Well-graded gravel
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	Poorly-graded gravel
		GRAVELS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	GM	Silty gravel
			FINES CLASSIFY AS CL OR CH	GC	Clayey gravel
	SANDS 50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	Well-graded sand
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	Poorly-graded sand
		SANDS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	SM	Silty sand
			FINES CLASSIFY AS CL OR CH	SC	Clayey sand
FINE-GRAINED SOILS 50% OR MORE PASSES THE NO.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%	INORGANIC	PI > 7 AND PLOTS ON OR ABOVE "A" LINE	CL	Lean clay
			PI < 4 OR PLOTS BELOW "A" LINE	ML	Silt
		ORGANIC	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OL	Organic Clay & Organic Silt
			PI PLOTS ON OR ABOVE "A" LINE	CH	Fat clay
	SILTS AND CLAYS LIQUID LIMIT 50% OR MORE	INORGANIC	PI PLOTS BELOW "A" LINE	MH	Elastic silt
			$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	OH	Organic Clay & Organic Silt
		ORGANIC			
			PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR	PT	Peat
HIGHLY ORGANIC SOILS					

REFERENCE: Unified Soil Classification System (ASTM D 2487-00)

* Criteria may be done on visual basis, not necessarily based on lab testing

$$A - C_u = D_{60}/D_{100} \quad \& \quad C_c = (D_{30})^2 / (D_{10} \times D_{60})$$

GRAIN SIZES

U. S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENINGS		
200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS		SAND		GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	BOULDERS

ABBREVIATIONS

INDEX TESTS

- LL - Liquid Limit (%) (ASTM D 4318-00)
 PI - Plasticity Index (%) (ASTM D 4318-00)
 -200 - Passing No. 200 Sieve (%) (ASTM D 1140-00)

STRENGTH TESTS

- PP - Field Pocket Penetrometer test of unconfined compressive strength (tsf)
 TV - Field Torvane test of shear strength (psf)
 UC - Laboratory unconfined compressive strength (psf) (ASTM D 2166-00)
 TXUU - Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf)
 (ASTM D 2850-03a)

MISCELLANEOUS

- ATOD - At time of drilling
 psf/tsf - pounds per square foot / tons per square foot
 psi - pounds per square inch (indicates relative force required to advance Shelby tube sampler)

SYMBOLS



Standard Penetration
Test Split Spoon
(2-inch O.D.)



Modified California
Sampler
(3-inch O.D.)



Thin-walled Sampler
Tube (either Pitcher or
Shelby) (3-inch O.D.)



Rock Core



Bag Sample



Groundwater Level



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KEY TO EXPLORATORY BORING LOGS

Berkeley Bicycle/Pedestrian Bridge - Eastern Touchdown Plaza
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FIGURE **A1**

DRILL RIG: CME 75, Hollow Stem Auger	SURFACE ELEVATION: (see notes)	LOGGED BY: GH
DEPTH TO GROUNDWATER: 19 feet (see notes)	BORING DIAMETER: 8 inches	DATE DRILLED: 3/26/07

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
SILT - top soil, with roots.	Brown	Soft	ML						
CLAY, Lean, Gravelly - gravels up to 2.5" diameter, roots, wood pieces, organics, moderate plasticity, moist to wet (FILL).	Very Dark Brown and Light Olive Brown	Stiff	CL	1					PP = 2.0 tsf LL = 36; PI = 20 %-200 = 61
				2		[18]	22	96	
		Firm		3					Hydrocarbon odor
				4		[9]			
-wet below 5 feet.		Soft to Firm		5					PP = 0.5 tsf
CLAY/SILT, Sandy - fine sand, moist (POSSIBLE FILL).	Blue Gray	Soft	CL/ML	6		[9]	24	93	
CLAY, Fat - some gravel, thin rootlets, trace sand, organics, wet (POSSIBLE FILL).	Bluish Black	Firm to Stiff	CH	7					
				8					PP = 1.0 tsf Shelby Tube: 600 psi
CLAY, Lean - some silt, moderate plasticity, moist to wet.	Reddish Brown	Stiff to Very Stiff	CL	9					Hydrocarbon odor
				10					PP = 2.25 tsf
				11					
CLAY, Fat - with silt, trace sand, wet.	Reddish Brown	Stiff	CH	12					
				13					PP = 2.5 tsf
				14		[14]	28		PP = 2.5 tsf
				15					
				16					
				17					
-trace fine gravels.				18					PP = 1.25 tsf
				19		[17]			PP = 1.25 tsf

(Continued on Next Page)



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EXPLORATORY BORING LOG

Berkeley Bicycle/Pedestrian Bridge - Eastern Touchdown Plaza
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PROJECT NO.	DATE	SHEET	BORING NO.
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DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
<i>(Continued from Previous Page)</i>									
CLAY, Fat - with silt, trace sand, wet.	Reddish Brown	Stiff	CH	21					
				22					
				23					
-trace fine gravels/coarse sands.		Stiff to Very Stiff		24		[25]			PP = 1.5 tsf PP = 3.25 tsf
				25					
				26					
				27					
				28					
				29					
				30					
		Very Stiff to Hard		31					PP = 4.5 tsf

Bottom of boring at 31.5 feet.

NOTES:

1. Groundwater was encountered at approximately 20 feet at the time of drilling and was at a depth of about 19 feet after drilling was complete. (See report for discussion.)
2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
3. Penetration resistance values (blow counts) enclosed in brackets ([]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
4. The surface elevation is unknown.
5. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".
6. The boring was grouted following drilling in accordance with the City of Berkeley permit requirements.



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EXPLORATORY BORING LOG

Berkeley Bicycle/Pedestrian Bridge - Eastern Touchdown Plaza
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SHEET
2 of 2

BORING NO.

1

DRILL RIG: CME 75, Hollow Stem Auger	SURFACE ELEVATION: (see notes)	LOGGED BY: GH
DEPTH TO GROUNDWATER: 3.6 feet (see notes)	BORING DIAMETER: 8 inches	DATE DRILLED: 3/26/07

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
SILT - top soil, with roots.	Brown	Soft	ML	1					
CLAY, Lean, Gravelly - with silt, some coarse sand and gravel, roots, moderate plasticity, moist (FILL).	Mottled Dark Brown, Orange, Light Brown	Stiff	CL	2		[19]	14	100	PP = >4.5 tsf LL = 45; PI = 26 %-200 = 58
		Soft to Firm		3			16	91	PP = 4.5 tsf
				4		[7]			
				5					
				6					
SILT, Sandy - fine grained sand, some clay, wet (POSSIBLE FILL).	Bluish Black to Black	Soft	ML	7					Shelby Tube: 400 psi
				8			38		Hydrocarbon odor
-sandstone fragments at 8.8 feet.				9					LL = 31; PI = 4 %-200 = 79
CLAY, Lean to Fat - trace coarse sand, moist.	Reddish Brown	Firm	CL/CH	10		6	25		
				11					
				12					
				13					
				14					
				15					
-increase in plasticity.			CH	16					PP = 1.0 tsf
CLAY/SILT, Lean - with fine sand, moist.	Reddish Brown	Firm to Stiff	CL/ML	17					
SAND, Silty - fine grained sand, wet.	Reddish Brown	Medium Dense	SM	18		[24]	25		%-200 = 46
				19					
CLAY, Fat, Gravelly - with sand.	Reddish Brown	Stiff	CH						

(Continued on Next Page)

EXPLORATORY BORING LOG

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BORING NO. **2**

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DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
<i>(Continued from Previous Page)</i>									
CLAY, Fat, Gravelly - with sand.	Reddish Brown	Stiff	CH						PP = 1.25 tsf
GRAVEL/SAND, Silty - fine to coarse sand, fine gravel, some clay, wet. -clay seam between 22.2 and 22.3 feet.	Reddish Brown	Medium Dense	GM/SM	21		[47]	20		LL = 27; PI = 6 %-200 = 26
				22					
				23					
				24					
CLAY, Fat - some fine to medium grained gravels, wet. -trace gravels, some fine black sand.	Reddish Brown	Stiff to Very Stiff	CH	25		19			PP = 2.0 tsf PP = 3.0 tsf
				26					
				27					
				28					
		Stiff to Very Stiff		29					
				30					
				31					

Bottom of boring at 31.5 feet.

NOTES:

1. Groundwater was encountered at approximately 20 feet at the time of drilling and was at a depth of about 3.6 feet 24 minutes after drilling. (See report for discussion.)
2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
3. Penetration resistance values (blow counts) enclosed in brackets ([]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.
4. The surface elevation is unknown.
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EXPLORATORY BORING LOG

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2