MOORE TWINING ASSOCIATES, INC.

GEOTECHNICAL ENGINEERING INVESTIGATION FOR APPROXIMATE ½ MILE LONG UNDERGROUND STORM DRAIN PIPELINE AND DETENTION BASIN PACHECO BOULEVARD AT PLACE ROAD TO TALBOTT PARK LOS BANOS, CALIFORNIA

Project Number: E53606.01

Mr. Gary Rogers
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1120 West I Street, Suite C
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May 4, 2011 (Revised)

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May 4, 2011 (Revised) E53606.01

Mr. Gary Rogers
AECOM
1120 West I Street, Suite C
Los Banos, CA 93635

Subject: Geotechnical Engineering Investigation for
Approximate ½ Mile Long Underground Storm Drain Pipeline
And Detention Basin
Pacheco Boulevard at Place Road to Talbott Park
Los Banos, California

Dear Mr. Rogers:

We are pleased to present this geotechnical engineering investigation report prepared for the proposed City of Los Banos underground storm drain pipeline and detention basin. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

We recommend that those portions of the plans and specifications that pertain to earthwork and trenching be reviewed by Moore Twining Associates, Inc. (Moore Twining) to determine if they are consistent with our recommendations.

We appreciate the opportunity to be of service to AECOM. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience at (800) 268-7021.

Sincerely,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

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GEOTECHNICAL ENGINEERING INVESTIGATION FOR APPROXIMATE ½ MILE LONG UNDERGROUND STORM DRAIN PIPELINE AND DETENTION BASIN PACHECO BOULEVARD AT PLACE ROAD TO TALBOTT PARK LOS BANOS, CALIFORNIA

Project Number: E53606.01

This report presents the results of a geotechnical engineering investigation for about 2,900 feet of new storm drain line and a detention basin to be located at Talbott Park, City of Los Banos, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by the AECOM to conduct this investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The background information, a description of the investigative procedures used and the subsequent findings are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The appendices contain the drawings (Appendix A), the logs of borings (Appendix B), the results of laboratory tests (Appendix C), and General Dewatering Guidelines (Appendix D).

The Geotechnical Engineering Division of Moore Twining, headquartered in Fresno, California, performed the investigation.

1.0 PURPOSE AND SCOPE OF INVESTIGATION

The purpose of the investigation was to conduct a field exploration and a laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

1) Geotechnical recommendations for use in design of the proposed pipeline and detention basin improvements;

2) Descriptions of the subsurface soil and groundwater conditions encountered during our field exploration;

3) Recommendations for trench preparation and backfill, including pipe bedding and backfill and compaction of engineered fill soils;

4) Recommendations for on-site material for use as engineered fill, including recommendations for precautions and handling of expansive soils;

5) Recommendations for support of miscellaneous lightly loaded foundations foundations;

6) Determination of seismic coefficients and spectral acceleration factors for design of miscellaneous foundations;

7) Identification of pipe loading conditions due to existing soils/groundwater conditions;

8) A summary of soil corrosivity testing (pH, resistivity, chlorides and soluble sulfates);

9) A site plan identifying the boring locations, final test boring logs and final laboratory test results; and

10) General recommendations for temporary excavations, shoring design and temporary dewatering systems.

This report is provided specifically for the pipeline and detention basin referenced in the Anticipated Construction section of this report. This investigation was not intended to meet any specific geotechnical requirements of any public agencies, percolation or infiltration tests a flood plain investigation, compaction tests, environmental investigation, or environmental audit.

The actions undertaken during the investigation are summarized as follows:

I. The following plan sheets, dated January 13, 2001, prepared by AECOM, were reviewed:

Conceptual Layout - Basin, Sheet 1 of 3
Conceptual Layout - Pipeline, Sheet 2 of 3
Conceptual Layout - Pipeline, Sheet 3 of 3

II. A site reconnaissance and subsurface exploration were conducted.

III. Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.

IV. Mr. Gary Rogers (AECOM) was consulted during the investigation.

V. The data obtained from the investigation were evaluated to develop an understanding of the subsurface conditions and engineering properties of the subsurface soils.

VI. This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, evaluations, conclusions, and recommendations.

2.0 PROJECT DESCRIPTION

The previous studies, site description, and the anticipated construction are summarized in the following subsections.

2.1 Previous Studies: At the time of our investigation, no previous geotechnical engineering, geological, or compaction test reports conducted for the site area were provided for review. If other reports become available, these reports should be provided for review and consideration for this project.

2.2 Site Location and General Description: The underground storm drain pipeline project will extend under the future alignment of Place Road from the north side of Pacheco Boulevard about 1,640 feet north through an existing agricultural field to San Luis Street, then about 800 feet west under San Luis Street to the intersection with Miller Lane, then about 320 feet north under an existing asphaltic concrete parking lot and a portion of Talbott Park, and finally about 120 feet northwest to the outlet structure and detention basin to be located within Talbott Park. The ground surface elevations along the pipe alignment are relatively similar at 109 to 110 feet above mean sea level (AMSL). Drawing No. 1, Appendix A, shows the approximate location of the proposed improvements. Descriptions of the conditions along the proposed pipeline alignment and detention basin, including underground utilities, are provided in section 4.1 of this report.

2.3 Anticipated Construction: It is our understanding that the pipeline will likely be 48 inch diameter cast-in-place concrete or HDPE pipe, and the pipe invert depths will be about 14 feet below surface grade (BSG). The detention basin will be excavated to depths of about 6 to 10 feet below existing surface grade at Talbott Park. The basin will not have a liner and will be dual use (playfield/basin) with a storage capacity of 12 to 20 acre-feet with plan dimensions of about 210 feet by 440 feet. The Conceptual Layout plan sheet indicates basin side slopes of approximately 6 horizontal (H) to 1 vertical (V). It is also our understanding that a sidewalk is proposed around the basin, small retaining walls may be required as part of the outlet structure, and appurtenant construction may require lightly loaded shallow foundations and slabs-on-grade.

3.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing program conducted for this investigation are summarized in the following subsections.

3.1 Site Reconnaissance: The site reconnaissance consisted of visual observations of surface features. Reconnaissance was conducted by Mr. Shawn Vaughn (Moore Twining staff geologist) on February 17, 2011. The features noted are described in the background information section of this report.

3.2 Drilling Test Borings: Prior to drilling the test borings, an encroachment permit (# ENC2011-7) was obtained from the City of Los Banos Public Works Department and the boring sites were marked for Underground Service Alert. Seven (7) test borings were drilled on February 17, 2011 to depths ranging from 15 to 21½ feet below site grade (BSG). The boring locations were determined in consultation with Mr. Gary Rogers (AECOM) and the approximate locations are shown on Drawing No. 2, Appendix A. Borings B-1 through B-5 were drilled within about 30 feet of the proposed pipeline alignment. Borings B-6 and B-7 were drilled within the area of the proposed detention basin. The borings were drilled using a CME-75 drill rig equipped with 6 ⅝-inch outside diameter (O.D.) hollow stem augers, under the direction of a Moore Twining professional geologist. The soils encountered in the test borings were logged during drilling by a representative of our firm. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of the borings.

3.2.1 Soil Sampling: Standard penetration tests were conducted in the test borings, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜-inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel sampler lined with brass rings into the soil. The soil was retained in brass rings, 2.5 inches O.D. and 1-inch in height. The lower 6-inch portion of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for

transport to the laboratory. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

3.2.2 Installation of Piezometers: At the completion of drilling and sampling operations, temporary piezometers were installed in three (3) boreholes (borings B-1, B-3, and B-7) for future groundwater level monitoring to be conducted by others. Each piezometer consisted of 1-inch diameter PVC pipe installed to a depth of 21½ feet BSG. The bottom five (5) feet of each piezometer was slotted pipe and the remainder was solid/blank pipe. Sand was placed in the annulus from the bottom of the hole to about 1 foot above the slotted pipe section and an approximate two (2) foot section of bentonite chips was placed on top of the sand. The remainder of the annulus of the boring was backfilled with soil cuttings from the boring. Excess cuttings from the boreholes were spread out on the dirt surfaces at or near the boring sites. Some future settlement of the ground surface at the boreholes should be anticipated.

3.2.3 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and relatively undisturbed samples of the subsurface material encountered.

The results of laboratory tests on samples obtained from the test borings are included in Appendix C of this report.

4.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

4.1 Surface and Subsurface Conditions Along Pipeline Alignment: The initial approximately 1,630 feet of pipeline will extend north and northeast from Pacheco Boulevard north of Place Road to near the centerline of San Luis Street. The pipeline will extend through an undeveloped agricultural field which had recently been disced prior to the time of our field investigation. The backyard portions of residential lots with single family residences are located about 15 to 20 feet west of the planned pipeline alignment in this area. Wooden fences separate the properties from the agricultural field. The pipeline will then extend westward for about 790 feet under San Luis Street. San Luis Street is a two-lane asphaltic concrete covered street. A sewer manhole was noted within San Luis Street, south of East "B" Street. It is anticipated other underground utilities are located under San Luis Street which may be within or near the proposed pipeline alignment. Sidewalks, driveways, curbs and landscaped areas are located about 10 feet north of the pipeline alignment, adjacent to the north side of San Luis Street. Overhead utilities are located above the sidewalk north of San Luis Street. The undeveloped agricultural field is located south of San Luis Street. At the intersection with Miller Lane, the proposed pipeline will extend

about 320 feet north through the west portion of an existing parking lot. The parking lot has lighting and landscaped islands and it is anticipated that underground utilities exist within or near the proposed pipeline alignment in this area. The pipeline will extend north of the parking lot, about 100 feet northwest across the grass covered portions of the park grounds to the basin inlet structure. The area of the proposed basin was occupied by grass areas, a playground, and numerous mature trees. It is anticipated that underground electric and irrigation facilities are located within the area of the proposed basin.

4.2 **Soil Profile:** The subsurface soil profiles encountered in the seven (7) borings were highly variable over the approximate ½ mile long pipeline and within the basin area. Soils encountered in the upper 14 to 15 feet BSG consisted of lean clays, sandy silts, and silty sands. Lean clays and silty sands were the predominant soil type encountered below 15 feet BSG, however, a poorly graded sand was encountered at a depth of 15 feet BSG in boring B-5 (southeast corner of Miller Lane and San Luis Street). In addition, thin veins of hardpan (cemented soils) were encountered in borings B-3 and B-6 at depths of about 5 to 10 feet BSG.

The soil survey prepared by The United States Department of Agriculture, Natural Resources Conservation Services (NRCS), identifies two (2) primary surface soil types in the areas of the proposed pipeline and basin. The Woo Clay Loam, 0 to 2 percent slopes is indicated to be located throughout the portion of the site south of San Luis Street, and is described as a clay loam to 60 inches BSG. The survey also identifies the Woo Urban Land Complex, 0 to 2 percent slopes, located in the project area north of San Luis Street. This soil is described as a loam from 0 to 16 inches and a clay loam from 16 to 67 inches.

The foregoing is a general summary of the soil conditions encountered in the test borings drilled. Detailed descriptions of the soils encountered at the test boring are presented on the logs of borings in Appendix B of this report. The stratification lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

4.3 **Soil Engineering Properties:** The engineering properties of the subsurface soils encountered during this investigation are summarized below. The soil engineering properties have been summarized below by general soil types encountered.

Lean Clay Soils: The lean clay soils encountered in the upper 15 feet BSG were medium stiff to very stiff, as indicated by standard penetration resistance, N-values, ranging from 5 to 27 blows per foot. The moisture contents of the near surface lean clays tested ranged from 13.8 to 27.3 percent. The results of an Atterberg limits test, indicated a Liquid Limit of 44 and a Plastic Index of 31. The results of direct shear testing of samples collected from borings B-4 and B-7 at depths of 5 and 8½ feet BSG indicated angles of internal friction of 16 and 14 degrees with cohesion values of 270 and 320 pounds per square foot, respectively.

Lean clay soils encountered below 15 feet BSG were stiff to very stiff, as indicated by standard penetration resistance, N-values, ranging from 8 to 26 blows per foot. The moisture contents of these deeper lean clays tested ranged from 20.6 to 24.9 percent. The results of testing of one (1) ring sample indicated a dry density of 94.6 pounds per cubic foot and a moisture content of 20.6 percent. The results of an Atterberg limits test, indicated a Liquid Limit of 43 and a Plastic Index of 29.

Silty Sands: The native silty sands encountered were predominantly fine grained, and were loose to medium dense as indicated by standard penetration resistance, N-values, ranging from 6 to 22 blows per foot. Dense silty sand were encountered in boring B-3 at a depth of 5 feet BSG with thin veins of hardpan (cemented soils). The results of testing of two (2) ring samples indicated dry densities of 120.6 and 91.4 pounds per cubic foot, with moisture contents of 8.2 and 20.5 percent, respectively. Due to the relatively low laboratory density determined (91.4 pounds per cubic foot) for one (1) of the samples, the sample appeared to have been disturbed during sampling and the density may not be indicative of in-place conditions.

Silts: Sandy silt was encountered in boring B-2 at depths of about 1 to 10 feet BSG. The silt was medium stiff to stiff as indicated by standard penetration resistance, N-values, of 5 to 9 blows per foot. The moisture content of one silt soil sample was measured to be 16 percent.

Poorly Graded Sands: Poorly graded sands were encountered in boring B-5 at depths of 15 feet BSG, extending to the maximum depth explored of 16 ½ feet BSG, and were loose as indicated by a standard penetration resistance, N-value, of 6 blows per foot. The results of one (1) ring sample indicated a dry density of 124.9 pounds per cubic foot and a moisture content of 4 percent.

Chemical Tests: Chemical tests were performed on two (2) soil samples collected at depths of 0 to 1½ feet BSG and one (1) soil sample collected at depths of between 5 to 10 feet below site grades.

The results of the near surface tests indicated pH values of 7.4 and 8.0; resistivity values of 13,000 and 13,000 ohms-centimeter, respectively. The test results for the near surface samples indicated 0.0053 and 0.0035 percent by weight concentrations of sulfate; and 0.0010 and 0.00094 percent by weight concentrations of chloride. The test results for the sample collected between 5 and 10 feet below site grades indicated a pH value of 9.6; a resistivity value of 1,400, a 0.014 percent by weight concentration of sulfate; and a 0.029 percent by weight concentration of chloride, respectively.

4.4 <u>Groundwater Conditions</u>: Groundwater was not encountered in test borings drilled during our field investigations on February 17, 2011. However, it should be noted that "wet" soils were reported in borings B-2 and B-7 at about 15 feet BSG. Based on our experience, it is likely that the groundwater surface is present within a few feet below the bottom of these borings. Based on review of the Department of Water Resources On-Line Water Well Database, groundwater depths listed for one (1) well (10S11E17Q0001M) located about ½ mile west of the site area indicated groundwater depths generally ranging from less than a foot to about 15 feet BSG between the years

1958 and 1969. The Department of Water Resources On-Line Database also provided groundwater depths for one (1) well (10S11E16N001M) located about ½ mile east of the site area. The data indicated groundwater depths generally ranging from less than 3 feet to about 9 feet BSG, between the years 1958 and 1977.

Mr. Gary Rogers (AECOM) provided data that indicates groundwater levels were at about 19½ feet BSG and 21 feet BSG in the piezometers installed in borings B-1 and B-3, respectively, on March 9, 2011. The piezometer installed in boring B-7 was dry.

It should be recognized that water table elevations fluctuate with time since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field exploration may vary from those encountered both during the construction phase and the design life of the project.

5.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface conditions determined from the field investigation, and our understanding of the proposed construction. Moore Twining should be notified of any changes in the proposed pipeline alignment and basin location, or planned depths and should review the final plans. Changes to the locations or depths of the proposed improvements could significantly impact our evaluations and recommendations for this site.

5.1 Surface and Subsurface Conditions, Temporary Excavations and Monitoring:

Existing surface and subsurface conditions at and near the proposed pipeline alignment and basin are described in section 4.1 of this report. Surface improvements in the area of the proposed pipeline include streets, a parking lot, sidewalks, curbs, landscaped areas, and unknown residential backyard improvements. Subsurface improvements include underground utilities.

Based on the anticipated depth of excavation for the pipeline and proximity to existing improvements in some areas, shoring is anticipated for temporary excavations where sufficient space is not available to adequately slope the sides of the excavations. In addition, excavations should not extend below a 1.5H to 1V zone below existing utilities or structures which are to remain after construction. Excavations which are required to be advanced below the 1.5H to 1V envelope should be shored to support the soils, foundations, and slabs.

Thus, temporary excavation support, protection of existing improvements and monitoring of surface and subsurface improvements will be an important aspect of the project to reduce the potential for damage to existing facilities. The intent of the monitoring is to evaluate areas of potential movement/settlement of existing improvements due to nearby temporary excavations. All distress (if any) should be repaired by the Contractor at no cost to the Owner.

5.2 **Dewatering and Wet Soils:** Considering the wet soils noted in two (2) borings at depths of about 15 feet BSG, the groundwater levels recorded in two (2) piezometers on March 9, 2011 at depths of 19½ feet BSG and 21 feet BSG and the historic propensity for shallow groundwater conditions in the area, it should be anticipated that groundwater and/or soils with moisture contents greater than optimum may be encountered in excavations required for the pipeline and basin. These conditions may require dewatering, stabilization, or other methods to address seepage and wet soil conditions as part of construction of the project. Dewatering should be performed in accordance with all applicable local, regional, state and federal guidelines, whichever is most stringent.

In addition to dewatering, some of the soils excavated will likely be considerably over optimum moisture content. These soils will require processing such as by aeration or chemical soil treatment to obtain suitable moisture contents prior to compaction as engineered fill. It should also be anticipated that the bottom of utility trench excavations will require stabilization in some areas due to overly moist soil conditions and shallow groundwater. Stabilization of the bottom of the trenches may require over-excavation and placement of aggregate base. Alternative methods of trench bottom stabilization may be acceptable; however, they should be submitted to Moore Twining and the Owner for review and approval.

5.3 **Seismic Design Parameters:** The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The closest active or potentially active fault is the Great Valley Fault (Segment 9) located approximately 6 miles west of the project site. Accordingly, the potential for ground rupture at the site is considered low.

Based on the 2010 CBC, the site is classified as a class D site (soil profile type) with standard penetration resistance, N-values averaging between 15 and 50 blows per foot in the upper 100 feet below site grade.

A table providing the recommended seismic coefficients and earthquake spectral response acceleration values for the project site is included in the Foundation Recommendations section of this report.

5.4 **Corrosion Protection:** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member. Corrosion can eventually damage or destroy a metallic object.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soil tested are included in the Conclusions section of this report. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

5.5 **Sulfate Attack of Concrete:** Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfates present in the soils. The test results are then compared with the provisions of ACI 318, section 4.3 to provide guidelines for concrete exposed to sulfate-containing solutions. Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios.

The soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters.

6.0 CONCLUSIONS

Based on the data collected during the field exploration and laboratory testing, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, the following general conclusions are presented.

6.1 The near surface soils encountered varied throughout the proposed pipeline alignment. In general, the subsurface soils predominantly consisted of lean clays and silty sand soils with some silt and poorly graded sands.

6.2 Groundwater was not encountered in test borings drilled during our investigation on February 17, 2011. However, several piezometers were installed as part of the field exploration, which will be monitored by others. Groundwater levels measurements made by AECOM on March 9, 2011 indicate groundwater at depths of about 19½ feet BSG and 21 feet BSG at the location of piezometers installed in borings B-1 and B-3, respectively. Water was not detected to a depth of about 21½ feet BSG in the piezometer installed in boring B-7. Shallower groundwater and soils with moisture contents greater than optimum should be anticipated in excavations required for the pipeline and basin.

6.3 The near surface lean clay soils encountered exhibited high plasticity characteristics and are considered expansive. Accordingly, placement and compaction of a granular fill is recommended below concrete slabs on grade (if any) to reduce the potential for excessive heave.

6.4 Due to the proximity of surface and subsurface improvements to the pipeline alignment, it is anticipated that the contractor will need to provide temporary excavation support, protection of existing improvements and monitoring of surface and subsurface improvements to reduce the potential for damage to existing facilities along and adjacent to the proposed pipeline alignment.

6.5 Chemical testing of near surface soils (0 to 1 ½ feet BSG) indicated the soils exhibit a "mildly corrosive" corrosion potential. Chemical analyses of these soils also indicates a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils. Chemical testing of soil samples collected at 5 to 10 feet BSG indicated the soils exhibit a "very corrosive" corrosion potential. Chemical analyses of these deeper soils also indicates a "negligible" potential for sulfate attack on concrete placed in contact with these soils.

6.6 The site is not located in an Alquist-Priolo special studies zone and the potential for fault rupture on the site is considered low.

7.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, we present the following recommendations for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered. The recommended design consultation and observation of earthwork, trench preparation and backfill activities by Moore Twining are integral to the proper application of the recommendations. The contractor should be required to comply with the recommendations contained herein.

7.1 General

7.1.1 This report was prepared for the proposed improvements described in the Anticipated Construction section of this report. If foundations, slabs on grade, or other improvements not described in this report are planned, Moore Twining should be requested to prepare recommendations for these improvements.

7.1.2 It is recommended that the detention basin design and construction comply with applicable standards for vertical setbacks to groundwater in accordance with the City of Los Banos requirements, or the requirements of the applicable governing agency.

7.1.3 A preconstruction meeting including, as a minimum, the owner, general contractor, foundation and underground subcontractors, civil engineer, and Moore Twining should be scheduled by the general contractor at least one week prior to the start of work. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.

7.1.4 A preconstruction survey is recommended prior to construction to document the condition of the existing improvements adjacent to the proposed construction which are to remain.

7.1.5 The contractor should monitor the existing improvements (i.e. sidewalks, curbs, etc.) adjacent to excavations for cracking, excessive rotation, settlement, etc. during the new construction activities. If any existing improvements move excessively during the construction operations, or are damaged as a result of the construction, the contractor shall take immediate action to correct the problem and should notify the project owner, and the

design engineer promptly. The contractor should repair all damage as a result of the construction activities at no cost to the owner.

7.1.6 The proposed construction should be conducted in accordance with the applicable project plans, specifications, and the City of Los Banos requirements, whichever is the most stringent.

7.1.7 After review of the aforementioned documents, the contractor(s) bidding on this project should determine if the data are sufficient for accurate bid purposes. If the data are not sufficient, the contractor should conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect more data as required to prepare accurate bids.

7.1.8 The contractor should anticipate the need to conduct dewatering of temporary excavations and stabilization of areas of wet, unstable soils at the bottom of pipeline excavations. A general dewatering specification is attached to this report in Appendix D. In addition, excavated soils may contain excessive moisture to obtain compaction. Therefore, the contractor should be prepared to condition (i.e., aeration, chemically treatment) soils as necessary to achieve moisture contents suitable for compaction.

7.1.9 The Contractor is responsible for all shoring, dewatering, pumping, stabilization of "wet" soils, disposal of groundwater and SWPPP requirements required to construct the project. No change orders will be allowed for construction issues associated with shoring, groundwater, groundwater disposal, stabilization of the bottom of excavations, or treatment of "wet" soils.

7.2 Site Grading and Drainage

7.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from existing adjacent improvements or proposed improvements such as foundations and slabs - both during and after construction. At a minimum, surface drainage adjacent to these features should be designed and maintained at a minimum of 2 percent positive drainage away from the improvements.

7.2.2 In general, it is recommended landscaping and irrigation systems not be planned adjacent to foundations and concrete slabs on grade. Where required, landscape and planter areas near foundations and slabs should be irrigated using low flow irrigation (such as drip, bubblers or mist type

emitters). The use of plants with low water requirements are recommended. Care should be taken to maintain a leak-free sprinkler system.

7.3 Site Preparation

This section provides general recommendations for site preparation. Recommendations for excavation and preparation of the trenches are included in section 7.7 of this report.

7.3.1 Prior to the start of excavation, the work area should be stripped of existing vegetation, root structures, and organics. The general depth of stripping should be sufficiently deep to remove the entire root systems and organic topsoils. A minimum stripping depth of 4 inches should be assumed for bidding purposes. The actual depth of stripping should be reviewed by the project geotechnical engineer at the time of construction. Deeper stripping may be required if any roots larger than ¼-inch are encountered during grading and in localized areas, such as low areas where water may pond or former agricultural land. Stripping of vegetated areas should extend laterally throughout the limit of the work areas. These stripped materials will not be suitable for use as fill; however, stripped topsoil may be stockpiled and reused in landscape and/or field areas at the discretion of the owner.

7.3.2 Existing trees (if any) to be removed and all root structures and soils with greater than 3 percent organics should be removed from areas of the proposed improvements. After removal, the loose soils should be removed from the bottom and the cavity backfilled with engineered fill. All soils disturbed from the tree removal activities should be over-excavated and compacted as engineered fill.

7.3.3 Where encountered, overly moist, unstable soils at the bottom of trench excavations should be stabilized prior to placement of fill. Stabilization may include over-excavation of the subgrade soils and placement of a geotextile fabric and aggregate base, or other methods submitted by the Contractor and accepted by the owner and Moore Twining.

7.3.4 Lightly loaded foundations and slabs on grade (if any) should be prepared by over-excavation to a minimum of 12 inches below the bottom of the proposed foundations/slabs, to the depth necessary to remove undocumented fills or a minimum of 12 inches below preconstruction site grade, whichever is deeper. Prior to fill placement, the bottom of the over-excavation should be scarified 8 inches in depth, moisture conditioned or aerated to between one percent

above optimum and four (4) percent above optimum moisture content and compacted as engineered fill. The moisture content of the subgrade soils should be maintained prior to placement of the aggregate base and construction of the slab on grade. Foundations and slabs on grade for lightly loaded equipment slabs should be supported on a minimum of 12 inches of Caltrans Class 2 aggregate base over the prepared subgrade.

7.3.5 All fills required to bring excavations to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

7.3.6 The contractor is responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. (if any) that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The contractor is responsible for performing the tests, assessm ents, etc. to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

7.4 Engineered Fill

7.4.1 Onsite native soils will not be suitable for use within 12 inches below concrete slabs or foundations. Foundations and slabs should be supported on 12 inches of aggregate base over subgrade soils prepared as recommended in the Site Preparation section of this report.

7.4.2 Considering the clay and silt soils generally encountered, the native soils are not recommended for use as bedding or initial backfill for the pipeline due to the fine-grained nature of the material and the difficulty achieving compaction of these materials in a limited work space. At a minimum, an imported, granular material (see subsection 7.4.5) is recommended for bedding, haunching and initial backfill of the pipelines.

7.4.3 The native soils are considered suitable for use as final trench backfill provided aeration and conditioning of the soils are conducted (as necessary) to obtain the moisture contents recommended in this report, contain less than 3 percent organics and do not contain particles larger than 6 inches in size.

7.4.4 The onsite soils are not recommended within a 1 horizontal to 1 vertical plane from the back of retaining walls (such as wing walls), unless the higher lateral earth pressures given in section 7.5 of this report are applied in design

for use of the onsite clayey soils. Retaining walls should be backfilled with imported, non-contaminated, non-expansive granular free-draining backfill placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footings at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings.

7.4.5 Granular backfill for retaining walls, and bedding, haunching, and initial backfill for pipes should be imported and should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	90 - 100
Percent Passing No. 200 Sieve	0-10
Plasticity Index	Less than 5
Sand Equivalent	Min. 30
Sulfates	< 0.05 % by weight
Min. Resistivity	> 5,000 ohm-cm

7.4.6 Prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner and Moore Twining) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. In addition, Moore Twining should be requested to sample and test the material to determine compliance with the above geotechnical criteria. Contractors should provide a minimum of 7 working days to complete the testing to determine compliance with the requirements of section 7.4.5.

7.4.7 Recycled materials should not be used as fill unless approved by the Owner and Moore Twining.

7.4.8 Additional backfill requirements for the pipeline are included in section 7.7 of this report.

7.4.9 All fill required to bring the site to final grades and for backfill of trenches should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

7.4.10 Imported, granular bedding and initial trench backfill, should be placed in loose lifts approximately 6 inches thick or less, moisture-conditioned to between optimum and three(3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density

as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

7.4.11 To reduce the potential for future settlement of overlying improvements, onsite native soils used as engineered fill soil, including final trench backfill, should be placed in loose lifts approximately 8 inches thick or less, moisture-conditioned to between one(1) and four (4) above optimum moisture content, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557 for fills placed below a depth of 5 feet below finished grade and within the upper 12 inches below pavement areas. Fills placed within the upper 5 feet of finished grade, with the exception of the upper 12 inches below pavements, should be compacted to a minimum of 92 percent of the maximum dry density as determined by ASTM D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

In the event a higher risk of future settlement is acceptable, as may be the case in agricultural areas or City park areas without overlying improvements, onsite native soils used as engineered fill soil, including final trench backfill, should be placed in loose lifts approximately 8 inches thick or less, moisture-conditioned to between one (1) and four (4) above optimum moisture content, and compacted to a dry density of at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. If two (2) or more compaction requirements are to be specified, the locations where these requirements apply should be delineated on the plans.

7.4.12 Imported, granular engineered fill soil used as engineered fill, including final trench backfill, should be placed in loose lifts approximately 8 inches thick or less, moisture-conditioned to between optimum and three(3) percent above optimum moisture content, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557 for fills placed below a depth of 5 feet below finished grade and within the upper 12 inches below pavement areas. Fills placed within the upper 5 feet of finished grade, with the exception of the upper 12 inches below pavements, should be compacted to a minimum of 92 percent of the maximum dry density as determined by ASTM D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

In the event a higher risk of future settlement is acceptable, as may be the case in agricultural areas or City park areas without overlying improvements,

imported granular soils used as engineered fill soil should be placed in loose lifts approximately 8 inches thick or less, moisture-conditioned to optimum and three(3) percent above optimum moisture content, and compacted to a dry density of at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. If two (2) or more compaction requirements are to be specified, the locations where these requirements apply should be delineated on the plans.

7.4.13 In-place density tests should be conducted in accordance with the requirements of ASTM D6938 (nuclear density) at a frequency of at least:

Area	Minimum Test Frequency
Mass Fills or Subgrade	1 test per 2,500 square feet per compacted lift, but not less than 2 tests per lift
Pipeline Bedding and Backfill	1 test per 150 feet per compacted lift

The above testing frequencies are suggested rates for tests. Testing frequency should be adjusted considering the methods used for and the contractor's quality control; however, the actual testing should not be less than listed herein unless modified in writing by our firm.

7.4.14 The compactability of the native and import soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, they should be evaluated by the contractor during preparation of bids and construction of the project.

7.4.15 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency or pipe manufacturer for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material.

7.4.16 All aggregate base should meet the requirements of a Caltrans Class 2 aggregate base. Aggregate base material should be compacted to a minimum of 95 percent relative compaction (ASTM D1557).

7.5 Lightly Loaded Foundations, Slabs on Grade, and Retaining Walls

7.5.1 The following recommendations are intended for use for miscellaneous lightly loaded foundations and concrete slabs on grade with isolated loads of less than 15 kips and less than 1.5 kips per lineal foot. The recommendations also apply to retaining walls up to 5 feet high. In the event structures and/or improvements with higher loads or higher retaining walls are planned, Moore Twining should be contacted to provide additional recommendations. Moore Twining should be contacted to observe/test the foundation/slab subgrade prior to placement of aggregate base.

7.5.2 Foundations and slabs should be supported on 12 inches of aggregate base over subgrade soils prepared as recommended in the Site Preparation section of this report. Placing aggregate base over dry or slightly moist subgrade should be avoided. The moisture content of the native engineered fill below the base section should be verified to be between one (1) and four (4) percent above optimum moisture content to a depth of 12 inches prior to placing the aggregate base.

7.5.3 **Shallow foundations for lightly loaded miscellaneous structures** (e.g. wing walls, etc.) should be supported on spread or continuous footings placed entirely on the minimum depths of engineered fill described in the Recommendations section of this report titled, "Site Preparation." Spread and continuous footings for miscellaneous foundations extending a minimum depth of 24 inches supported on subgrade soils prepared as recommended in this report may be designed for a maximum net allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads. The soil bearing pressure value may be increased for temporary loading such as seismic loads. The weight of the footing and the soil backfill may be ignored in design.

7.5.4 The miscellaneous foundations should be designed and reinforced for the anticipated differential settlements (static and seismic). A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on: 1) a total static settlement of 1 inch, 2) and a differential static settlement of ½ inch in 40 feet.

7.5.5 The following values were developed using the Ground Motion Parameter Calculator provided by the United States Geological Survey (http://earthquake.usgs.gov/) in accordance with the 2010 CBC.

Seismic Factor	2010 CBC Value
Site Class	D
Spectral Response At Short Period (0.2 Second), Ss	1.36
Spectral Response At 1-Second Period, S_1	0.447
Site Coefficient (based on Spectral Response At Short Period), Fa	1
Site Coefficient (based on spectral response at 1-second period) Fv	1.533
Maximum considered earthquake spectral response acceleration for short period, S_{Ms}	1.36
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	0.695
Five percent damped design spectral response accelerations for short period, SDs	0.906
Five percent damped design spectral response accelerations at 1-second period, SD_1	0.463

7.5.6 An allowable coefficient of friction of 0.25 can be used for design.

7.5.7 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot to a maximum of 2,500 pounds per square foot. These values assume level conditions.

7.5.8 A minimum factor of safety of 2.0 should be used when combining the frictional and passive resistance of the soil to determine the total lateral resistance. The upper 6 inches of subgrade soils in landscape areas should be neglected in determining the total passive resistance.

7.5.9 **Lightly loaded equipment slabs** on grade should be supported on at least 12 inches of Class 2 aggregate base over subgrade soils prepared as recommended in the Site Preparation section of these recommendations. Lightly loaded slabs on grade may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. The bearing capacity value may be increased by one-third for short duration wind or seismic loads. The moisture content of the native engineered fill below the base section should be verified to be between one (1) and four (4) percent above optimum moisture content to a depth of 12 inches prior to placing the aggregate base section.

7.5.10 Thickened edges for slabs on grade should have a minimum depth of 18 inches below the adjacent ground surface, or extending to the bottom of the aggregate base, whichever is lower.

7.5.11 A professional engineer should recommend the thickness, design details and concrete specifications for concrete slabs on grade supporting equipment pads based on a total static settlement of 1 inch and a differential static settlement of ½ inch.

7.5.12 If the subgrade is prepared and then disturbed by equipment workers, weather or other source, we recommend that the exposed subgrade to receive slabs or foundations be tested to verify adequate compaction. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, moisture conditioned to between one (1) and four (4) percent above optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D1557.

7.5.13 **For retaining wall design**, active and at-rest pressures of the imported, engineered fill meeting section 7.4.5 of this report may be assumed to be equal to the pressure developed by a fluid with a density of 45 and 68 pounds per cubic foot, respectively, for a level backfill ground surface. If onsite soils

are used as backfill within a 1 Horizontal to 1 Vertical zone of the walls, active and at-rest pressures of 66 and 87 pounds per cubic foot, respectively, should be used in design. However, due to the high fines contents and the potential for moisture contents above optimum moisture, the native soils will likely be difficult to compact in a limited work space. For sloped backfill conditions, Moore Twining should be contacted to provide supplemental recommendations for earth pressures.

7.5.14 The recommended earth pressures above assume drained soils conditions and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations, roadways and hydrostatic water pressure. Surcharge loading should be determined by and included in design by the design engineer.

7.5.15 The Contractor should use light, hand operated or walk behind compaction equipment in the zone equal to one wall height behind retaining walls to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The Contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

7.6 Shoring and Temporary Excavations

7.6.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The grades, classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

7.6.2 Trenches and temporary cut slopes requiring human entry (such as for pipe construction, soil testing, etc.) should be excavated and maintained in accordance with CAL OSHA requirements. The soils encountered in the test borings classify as a Type B soil per Cal OSHA. However, the excavation contractor should classify the soils encountered during trenching in accordance with the Cal OSHA requirements.

7.6.3 In no case should excavations extend below a 1.5H to 1V zone below existing utilities or structures which are to remain after construction. Excavations which are required to be advanced below the 1.5H to 1V envelope should be shored to support the soils, foundations, and slabs.

7.6.4 Shoring systems, if used, should be designed by a professional engineer with experience in designing shoring systems and registered in the State of California. It is recommended temporary shoring systems be designed based on a soil density of 130 pounds per cubic foot and an active earth pressure coefficient of 0.42. These values assume a level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure. Additional lateral pressures will need to be included in the shoring design based on applicable surcharges.

7.6.5 The contractor should monitor adjacent improvements for movement, cracking, settlement, or failure during the excavation operations. If movement more than ⅛ of an inch is noted during the construction operations, or to limits determined by the structural engineer, the Contractor shall notify the owner, the owner, engineer, and Moore Twining immediately, and take action to correct the problem. All damage should be repaired by the Contractor at no cost to the owner.

7.7 Pipelines and Trenches

7.7.1 The allowable load capacity on the pipeline should be evaluated by the design engineer based on the anticipated loading to confirm the planned storm drain piping and depths are suitable. For the purpose of design, a unit weight of 140 pounds per cubic foot may be assumed for the trench backfill soils compacted in accordance with the recommendations of this report. In addition to the soil overburden loading, pavement/traffic surcharge loading should be included in design, as applicable.

7.7.2 Trenching, excavation and support should be conducted in accordance with the recommendations of this report, the plans, specifications and ASTM D2321, whichever is the most stringent.

7.7.3 Storm drains and/or utility lines should be designed to be "watertight." If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil movement causing damage to surface and subsurface structures, pavements, flatwork, etc., nuisance water conditions, trench settlement, and potential for leakage of groundwater into the pipe in the event that the water table rises.

7.7.4 The contractor should anticipate the need to conduct dewatering of temporary excavations and stabilization of areas of wet, unstable soils at the bottom of pipeline excavations. A general dewatering specification is attached to this report in Appendix D. Dewatering should be performed in accordance with all applicable local, regional, state and federal guidelines, whichever is most stringent. In addition, as indicated in this report, excavated soils may contain excessive moisture to obtain compaction. Therefore, the contractor should be prepared to condition (i.e., aeration, chemical treatment) soils as necessary to achieve moisture contents suitable for compaction.

7.7.5 The trench subgrade should be prepared by excavation of a neat trench into native soils without disturbance to the bottom of the trench. The contractor shall either slope the excavation to create a stable sidewall, shore the excavation, or a combination of both. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 90 percent relative compaction prior to placement of bedding material. The contractor is responsible for notifying Moore Twining when these conditions occur and arrange for observation and testing prior to placement of pipe bedding. The contractor shall use such equipment as necessary to achieve a smooth, undisturbed surface at the bottom of the trench with no loose material at the bottom of the trench. The contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of pipe and backfill of the trench. If the trench is inadvertently dug below the design trench subgrade, the grade should be raised using engineered fill placed and compacted in accordance with this report.

7.7.6 The Contractor will be required to provide safe access to excavations for observations and in-place density testing to be conducted during all phases of the work, including subgrade preparation, bedding, haunching, initial and final backfill of trenches.

7.7.7 Contractors should assume all bedding and initial backfill materials will need to consist of imported granular material that meets the recommendations provided under section 7.3 of this report.

7.7.8 In the event that the unstable material due to overly moist soils are encountered at the bottom of the proposed trench excavation, stabilization should be conducted whenever the bottom of the trench is soft, yielding, or unsuitable as a foundation for the pipe. The contractor may elect to remove the unstable soils to a minimum depth of 12 inches below the bottom of the pipe or structure, and then backfill the area with a Class 2 aggregate base material placed and compacted in accordance with the recommendations under section 7.3 of this report.

7.7.9 In accordance with ASTM D2321, a horizontal modulus of soil reaction (E') of 1,000 pounds per square inch may be considered for pipelines surrounded by the compacted, granular bedding material recommended in this report.

7.7.10 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For thermoplastic pipes, such as HDPE, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project unless the material is fully encapsulated in a suitable geotextile filter fabric). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, or to the width necessary to achieve the required compaction of the initial backfill, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site soils or imported, granular materials compacted in accordance with the recommendations in the Engineered Fill section of this report.

7.7.11 As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 1
Minimum Trench Widths for HDPE Pipe with Select Sand Bedding as Initial Backfill

Inside Diameter of HDPE Pipe (inches)	Outside Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321-00
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80
60	67.3	96

7.7.12 Trenches can often become conduits for allowing surface water intrusion and subsequent migration along the trench. These increases in moisture contents can increase the potential for settlement of the trench backfill. Therefore, provisions such as providing cutoff collars or other means to prevent migration of water into the trenches should be considered.

7.7.13 All imported bedding and initial fill soils, to a minimum of 1 foot above the pipe, should be placed in loose lifts approximately 6 inches thick or less, moisture-conditioned to between zero (0) and three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. Soils should be placed and compacted in the pipe haunches to meet the minimum compaction requirements. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

7.7.14 The use of open-graded rock such as a crushed gravel shall not be permitted unless the material is fully encapsulated in a geotextile filter fabric.

7.7.15 Jetting of trench backfill is not allowed to compact the backfill soils.

7.7.16 It is recommended that the pipelines be video inspected after installation and backfill to verify that the pipelines are constructed properly, within the manufacturer's requirements for allowable deflection and are watertight. The inspection and testing should be conducted to the satisfaction of the Owner. The Contractor is required to repair all noted deficiencies at no cost to the owner.

7.8 Sidewalks

The recommendations for sidewalks provided below are not intended for use for slabs subjected to vehicular traffic, rather sidewalks for pedestrian uses.

7.8.1 Sidewalks should be supported on a minimum of 12 inches of imported, non-expansive fill over a minimum of 12 inches of moisture conditioned native engineered fill. Imported, non-expansive fill should consist of 4 inches of Class 2 Aggregate Base overlying 8 inches of imported, non-expansive engineered fill. Prior to placement of the concrete slab, the underlying clay soils should be pre-saturated to a minimum moisture content of 3 percent over optimum moisture content to a minimum depth of 18 inches below the bottom of the imported non-expansive fill. This condition should be verified by hand auger sampling and laboratory testing of collected moisture samples. The minimum moisture content requirements should be established and maintained within 48 hours prior to placement of concrete.

As an alternative, sidewalks may be supported on a minimum of 4 inches of granular engineered fill over a minimum of 12 inches of moisture conditioned native engineered fill. However, it should be noted that the potential exists for an increase in vertical rise or swell and increased maintenance when compared with the previous recommendation. This would also result in an increased potential for distress (excessive heave, cracking, development of tripping hazards, etc.) and higher maintenance and replacement costs. For this alternative, prior to placement of the concrete slab and after compaction of the subgrade to a depth of 12 inches, the underlying clay soils should be pre-saturated to a minimum moisture content of 3 percent over optimum moisture content to a minimum depth of 18 inches below the bottom of the slab. This condition should be verified by hand auger sampling and laboratory testing of collected moisture samples. The minimum subgrade soil moisture content requirements should be established and maintained within 48 hours prior to placement of concrete.

7.8.2 If the subgrade is prepared, and then disturbed by equipment workers, weather or other source, we recommend that the exposed subgrade to receive slabs be tested to verify adequate compaction. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted as engineered fill.

7.8.3 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing non-expansive materials and/or concrete walks and finish work over dry or slightly moist native subgrade should be avoided. It will be critical to moisture condition the subgrade soils to at least 3 percent above optimum moisture content to a depth of 18 inches below the non-expansive material immediately prior to placement of the concrete. The Contractor should obtain written confirmation of in-situ moisture test results prior to placing the concrete.

7.8.4 To reduce the effects of drying around the edges of the flatwork and reduce the potential for infiltration of water into the granular fill, lateral cutoffs which extend at least 12 inches below the bottom of the slab at the edges of flatwork such as inverted curbs are recommended. As an alternative to a 12 inch thickened edge, a lesser thickness for the edge could be considered. However, it should be noted that the potential exists for an increase in vertical rise or swell when compared with the previous recommendation.

7.9 Corrosion Protection

7.9.1 Based on the resistivity values, the native soils exhibit a "very corrosive" corrosion potential. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "very corrosive"corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

7.9.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentrations of sulfates determined for the near-surface soils. According to provisions of ACI 318, section 4.3, the sulfate concentration falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete.

7.9.3 These soil corrosi on data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

8.0 DESIGN CONSULTATION

8.1 Moore Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork operations, slabs-on-grade, trenching and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not a part of this current contractual agreement.

8.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes. If Moore Twining is not afforded the opportunity for review, Moore Twining assumes no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

9.0 CONSTRUCTION MONITORING

9.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

9.2 Moore Twining can provide observation and field testing to determine if the recommendations of the project geotechnical report are achieved. Upon completion of the work, a written summary of our observations will be provided, field testing and conclusions regarding the conformance of the completed work to the intent of the project geotechnical report. This service is not, however, part of this current contractual agreement.

9.3 The construction monitoring is an integral part of this investigation. This phase of the work provides the geotechnical engineer the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

9.4 If Moore Twining is not afforded the opportunity to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. We recommend that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance of at least $2,000,000 and review this report. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

9.5 Upon the completion of work, Moore Twining should be requested to prepare a final report. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to provide this report. This service is not, however, part of this current contractual agreement.

10.0 NOTIFICATION AND LIMITATIONS

10.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations.

10.2 The nature and extent of subsurface variations between borings may not become evident until construction.

10.3 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and the recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

10.4 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (more than 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

10.5 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

10.6 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

10.7 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

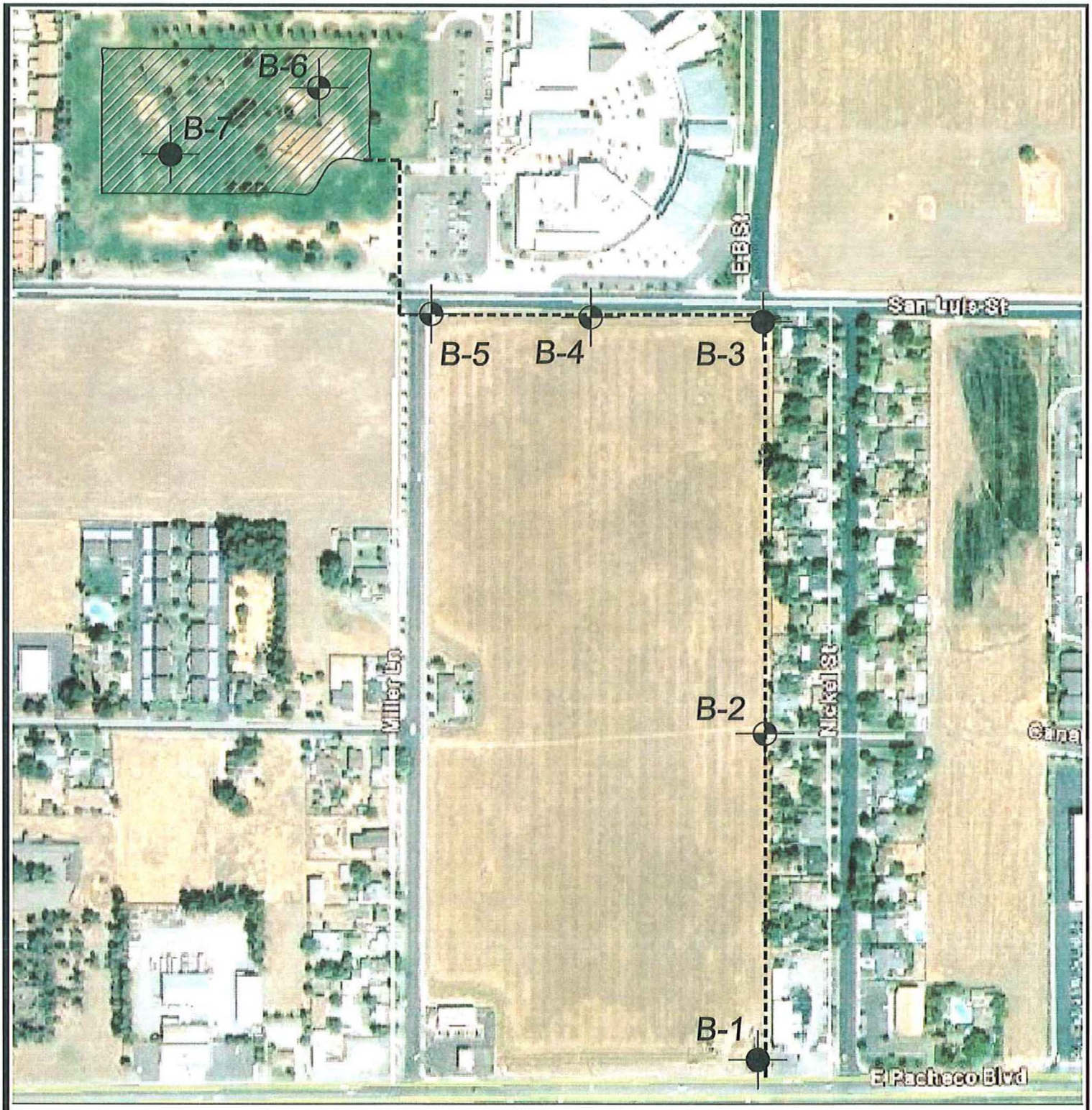
10.8 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

10.9 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

APPENDIX A

DRAWING

Drawing No. 1 - Boring Location Map



APPROXIMATE LOCATION OF TEST BORING

APPROXIMATE LOCATION OF TEST BORING AND PIEZOMETER

PROPOSED BASIN

APPROXIMATE PROPOSED PIPELINE ALIGNMENT

N

0 300

APPROXIMATE SCALE IN FEET

BORING LOCATION MAP NEW UNDERGROUND STORM DRAIN PIPELINE AND DETENTION BASIN CITY OF LOS BANOS, CALIFORNIA	FILE NO. 53606-01-02	DATE DRAWN: 03/05/11	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY: hc	
	PROJECT NO. E53606.01	DRAWING NO. 1	

APPENDIX B

LOG OF BORINGS

This appendix contains the final logs of the test borings soundings. The boring logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The boring logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these test boring locations. Also, the passage of time may result in changes in the soil conditions at these test boring locations.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Los Banos Storm Drain

Project Number: E53606.01

Drilled By: SR

Logged By: SV

Drill Type: CME 75

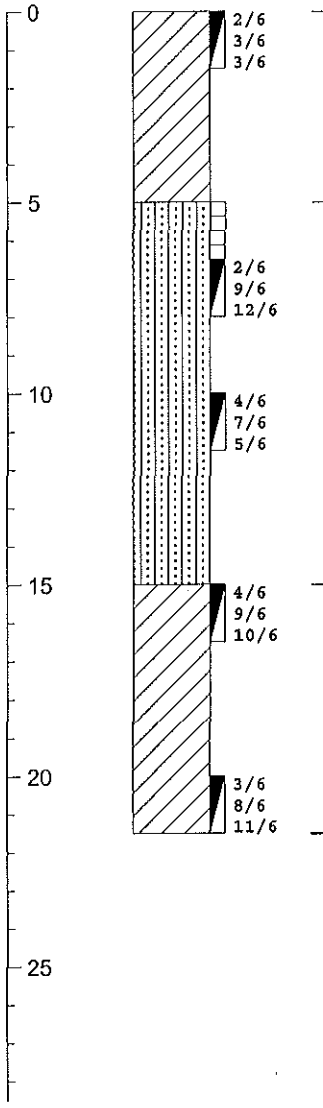
Date: 2/17/11

Auger Type: 6 5/8" HSA

Elevation: Approx. 110 feet AMSL

Hammer Type: 140lb Auto Trip

Depth to Groundwater: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		CL	LEAN CLAY; sandy, medium stiff, moist, dark brown, with organics	pH=7.4 SR=13,000 ohms/cm Cl=0.0010% SS=0.0053%	6	13.8
5		SM	SAND Silty; medium dense, moist, fine, brown	DD=120.6 pcf	--	8.2
			light brown		21	9.7
10					12	14.4
15		CL	LEAN CLAY; very stiff, wet, brown		19	24.9
20					19	23.1
			Bottom of boring at 21.5 feet			
25						

Notes: *One inch diameter PVC temporary piezometer installed in borehole to 21 1/2 feet BSG. Bottom 5 feet of piezometer comprised of slotted pipe. Sand placed from bottom of piezometer to 1 foot above slotted section. Two foot thick bentonite seal placed above sand and remainder of annulus backfilled with soil cuttings.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Los Banos Storm Drain

Project Number: E53606.01

Drilled By: SR

Logged By: SV

Drill Type: CME 75

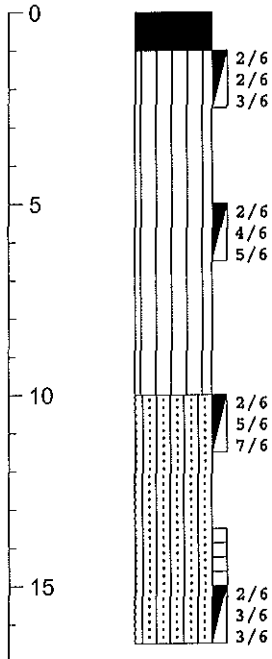
Date: 2/17/11

Auger Type: 6 5/8" HSA

Elevation: Approx. 110 feet AMSL

Hammer Type: 140LB Auto Trip

Depth to Groundwater: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		AC	Asphaltic Concrete = 4"			
		ML	Aggregate Base = 7"	-200=80.4%	5	16
			Silt, Sandy; medium stiff, moist, fine, dark brown			
			stiff, light brown to brown		9	
		SM	SAND Silty: medium dense, moist, fine, brown		12	
				DD=91.4 pcf	--	20.5
			loose, wet		6	
			Bottom of Boring at 16.5 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-3

Project: Los Banos Storm Drain

Project Number: E53606.01

Drilled By: SR

Logged By: SV

Drill Type: CME 75

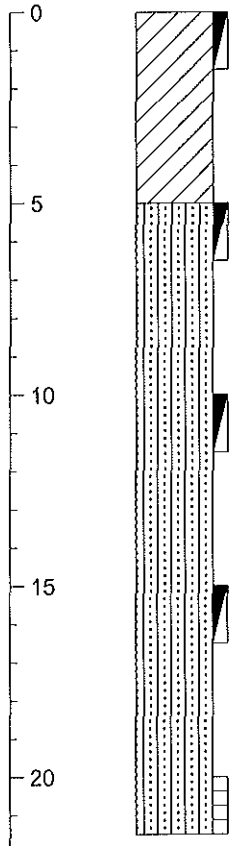
Date: 2/17/11

Auger Type: 6 5/8" HSA

Elevation: Approx. 109 feet AMSL

Hammer Type: 140LB Auto Trip

Depth to Groundwater: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		CL	LEAN CLAY; stiff, dry dark brown, some roots	ph=8.0 SR=13,000 ohms/cm Cl=0.00094% SS=0.0035%	11	13.8
		SM	SAND, Silty; dense, moist, light brown, fine grained, thin hardpan veinlets		35	7.1
			medium dense, brown		22	23.6
			loose		8	13.2
				DD=104.0 pcf		10.7
			Bottom of boring at 21.5 feet			

Notes: *One inch dia. PVC piezometer to 21 1/2 feet BSG. Bottom 5 feet of piezometer comprised of slotted pipe. Sand placed from bottom of piezometer to 1 foot above slotted section. Two foot thick bentonite seal placed above sand and remainder of annulus backfilled with soil. cuttings.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Los Banos Storm Drain

Project Number: E53606.01

Drilled By: SR

Logged By: SV

Drill Type: CME 75

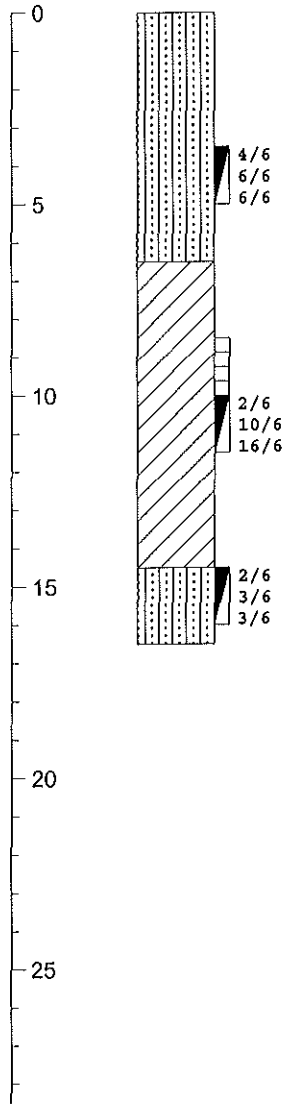
Date: 2/17/11

Auger Type: 6 5/8" HSA

Elevation: Approx. 110 feet AMSL

Hammer Type: 140LB Auto Trip

Depth to Groundwater: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		SM	SAND Silty; moist, fine, light brown, weak hard pan veinlets			
			medium dense		12	
		CL	LEAN CLAY; very stiff, moist, brown.			
			brown	ø=16° c=270 psf DD=106.4pcf	26	21.0
		SM	SAND, Silty: loose, fine grained, brown, damp.		6	
			Bottom of hole at 16.5 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-5

Project: Los Banos Storm Drain

Project Number: E53606.01

Drilled By: SR

Logged By: SV

Drill Type: CME 75

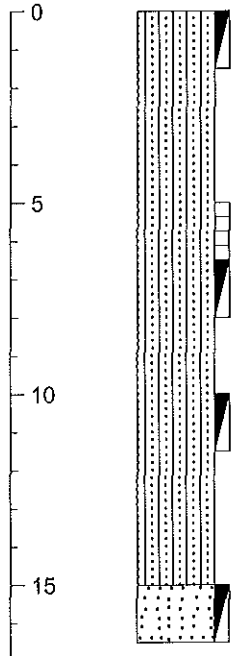
Date: 2/17/11

Auger Type: 6 5/8" HSA

Elevation: Approx. 111 feet AMSL

Hammer Type: 140lb Auto Trip

Depth to Groundwater: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		SM	SAND Silty; medium dense, dry fine, dark brown		15	
				DD=112.3pcf		9.1
			loose		8	8.8
			medium dense		11	14.9
		SP	SAND Poorly Graded; loose, moist, fine, brown	DD=121.0 pcf	6	8.9
			Bottom of hole at 16.5 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Los Banos Storm Drain

Project Number: E53606.01

Drilled By: SR

Logged By: SV

Drill Type: CME 75

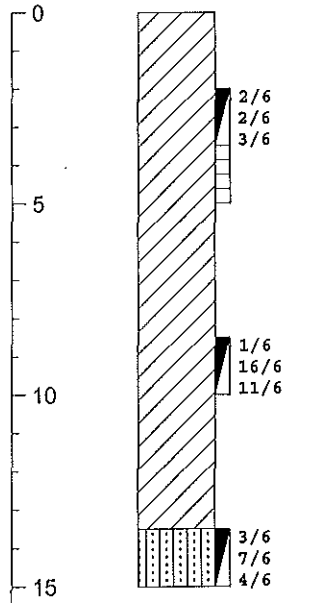
Date: 2/17/11

Auger Type: 6 5/8" HSA

Elevation: Approx. 110 feet AMSL

Hammer Type: 140lb Auto Trip

Depth to Groundwater: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		CL	LEAN CLAY; moist, dark brown to black			
			medium stiff, brown to dark brown		5	
			very stiff, medium to light brown, some veinlets of weak hardpan (caliche)	ph=9.6 SR=1,400 ohms/cm Cl=0.029% SS=0.014% PI=31 LL=44	27	21
		SM	SAND Silty; medium dense		11	
			Bottom of hole at 15 feet BSG			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-7

Project: Los Banos Storm Drain

Project Number: E53606.01

Drilled By: SR

Logged By: SV

Drill Type: CME 75

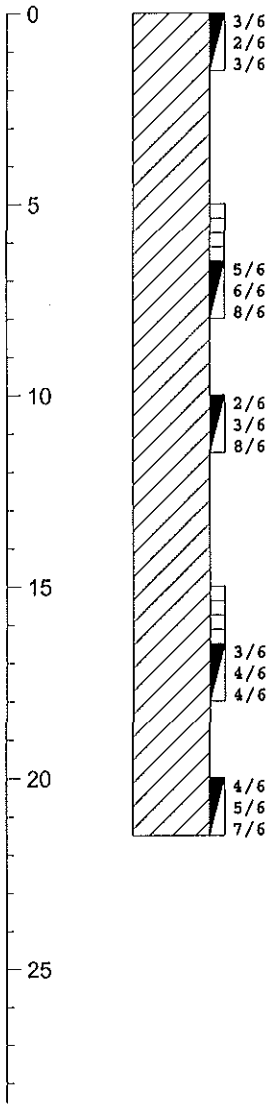
Date: 2/17/11

Auger Type: 6 5/8" HSA

Elevation: Approx. 110 feet AMSL

Hammer Type: 140lb Auto Trip

Depth to Groundwater: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		CL	LEAN CLAY; medium stiff, wet, brown to dark brown		5	
			brown, moist	ø=14° c=320 psf		18.4
			stiff	DD=105.2 pcf	14	20.6
			low plastic		11	27.3
				DD=94.6 pcf		20.6
			wet		8	
				-200=77.3% PI=29 LL=43	12	22.1
			Bottom of hole at 21.5 feet BSG			

Notes: *One inch dia. PVC piezometer to 20 feet BSG. Bottom 5 feet of piezometer comprised of slotted pipe. Sand placed from bottom of piezometer to 1 foot above slotted section. Two foot thick bentonite seal placed above sand and remainder of annulus backfilled with soil cuttings.*

Figure Number

KEY TO SYMBOLS

Symbol Description

Strata symbols

LEAN Clay

Silty sand

Asphaltic Concrete

Silt

Poorly graded sand

Soil Samplers

Standard penetration test

California Modified split barrel ring sampler

Notes:

1. Groundwater was not encountered during drilling of the borings.
2. Test Boring locations were located by pacing or steel tape with reference to the existing site features shown on the topographic map (see Drawing No. 1).
3. These logs are subject to the limitations, conclusions, and recommendations of the geotechnical report.
4. Results of tests conducted on samples recovered are reported on the logs.

5. Abbreviations used are:

DD =	Natural dry density (pcf)	LL =	Liquid limit (%)
UC =	Unconfined compression (psf)	PI =	Plasticity index (%)
-4 =	Percent passing #4 sieve (%)	pH =	Soil pH
-200 =	Percent passing #200 sieve (%)	SS =	Soluble sulfates(%
SR =	Soil resistivity (ohm-cm)	Cl =	Soluble chlorides(%)
c =	Cohesion (psf)	pcf =	pounds per cubic foot
psf =	pounds per square foot	N/E =	None encountered during drilling
N/A =	Not applicable		

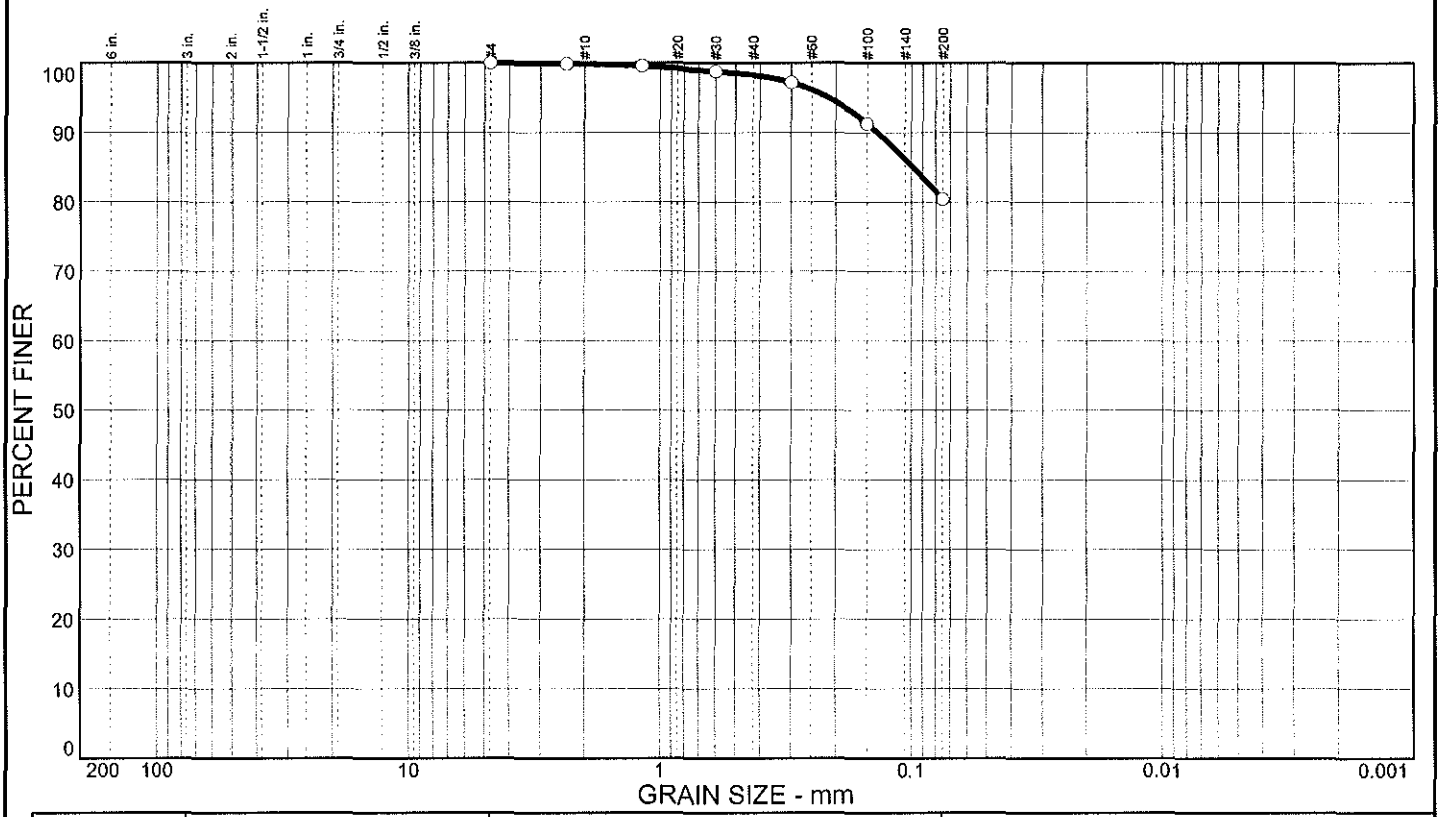
APPENDIX C

RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	To Determine:
Dry Density (ASTM D2216)	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.
Grain-Size Distribution (ASTM D422)	Size and distribution of soil particles, i.e., sand, gravel and fines (silt and clay).
Atterberg Limit (ASTM D4318)	The consistency and "stickiness," as well as the range of moisture contents within which the material is "workable."
Sulfate Content (ASTM D4327)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM D1125)	The potential of the soil to corrode metal.
pH (ASTM D4972)	The acidity or alkalinity of subgrade material.

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.1	1.6	17.9	80.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.0983							

MATERIAL DESCRIPTION	USCS	AASHTO
○		

Project No. E53606.01 **Client:**

Project: City of Los Banos Storm Drain

○ **Source:** **Sample No.:** B-2 **Elev./Depth:** 1-2.5 Feet

Remarks:

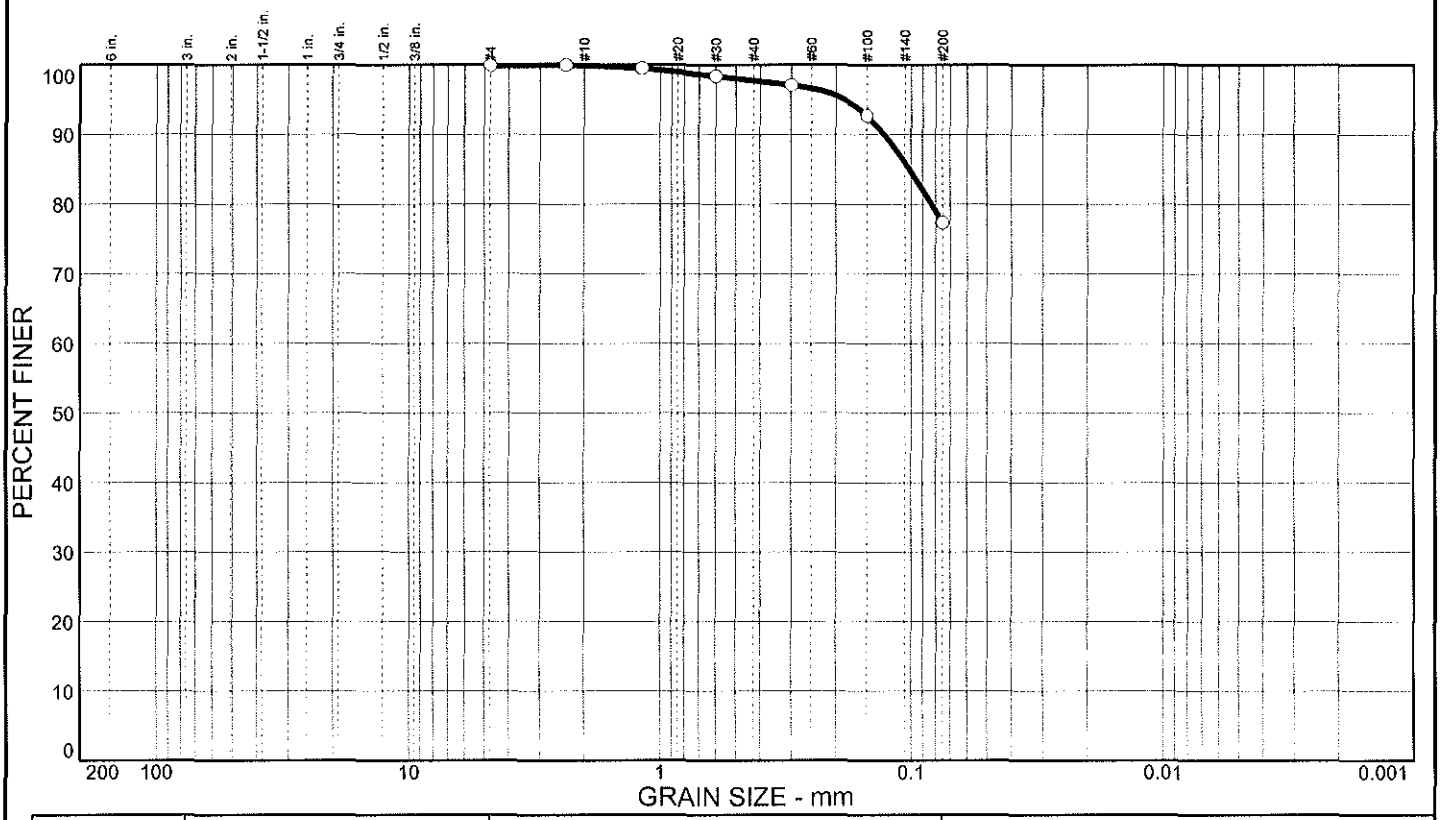
○ F.M.=0.13

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.1	2.1	20.5	77.3	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	43	14	0.102							

MATERIAL DESCRIPTION	USCS	AASHTO
○ Lean clay with sand	CL	

Project No. E53606.01 **Client:**

Project: City of Los Banos Storm Drain

○ **Source:** **Sample No.:** B-7 **Elev./Depth:** 20-21.5 Feet

Remarks:

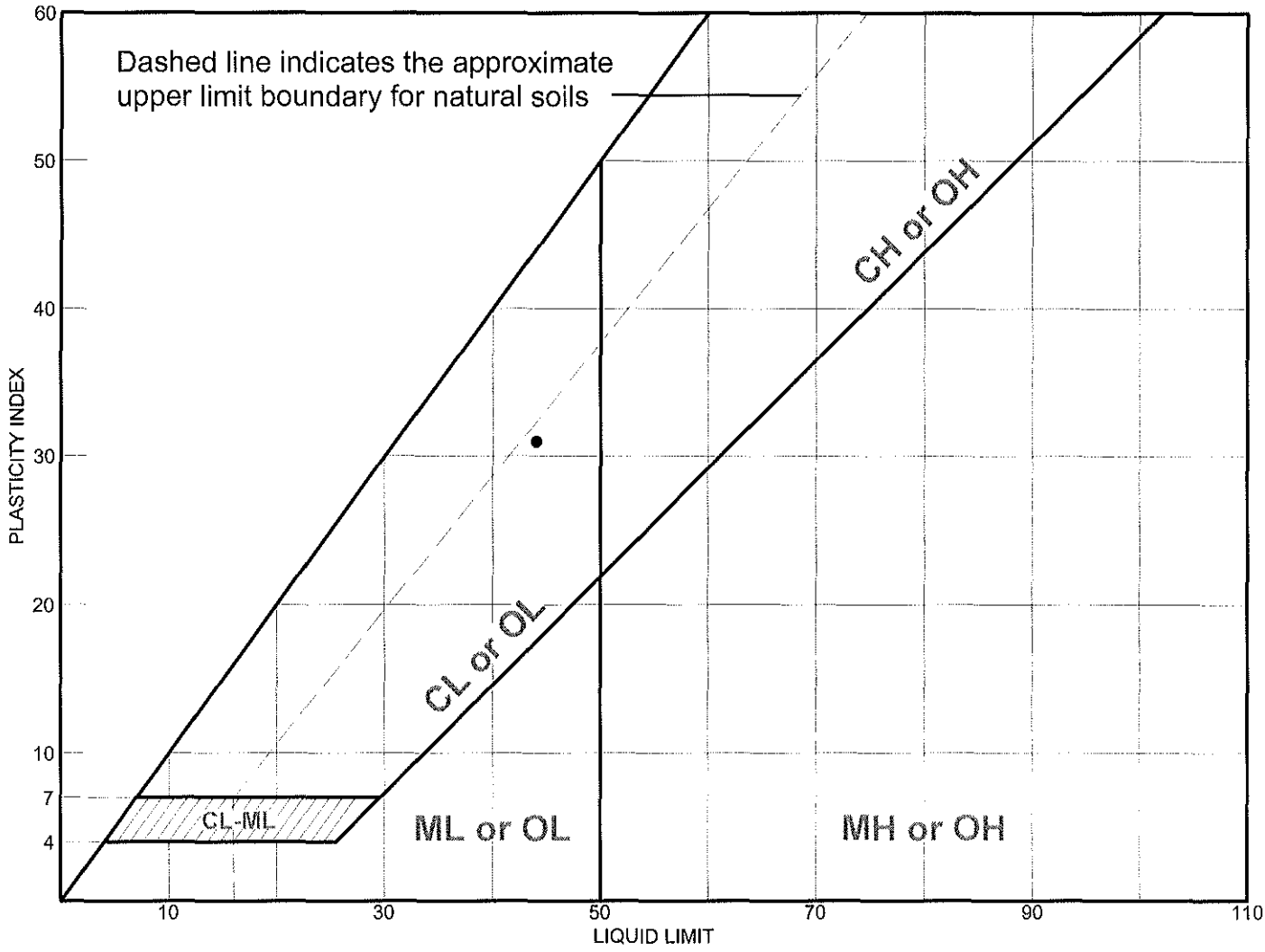
○ F.M.=0.12

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		44	13	31			

Project No. E53606.01 **Client:**

Project: City of Los Banos Storm Drain

● **Source:** **Sample No.:** B-6 **Elev./Depth:** 8.5-10 Feet

Remarks:

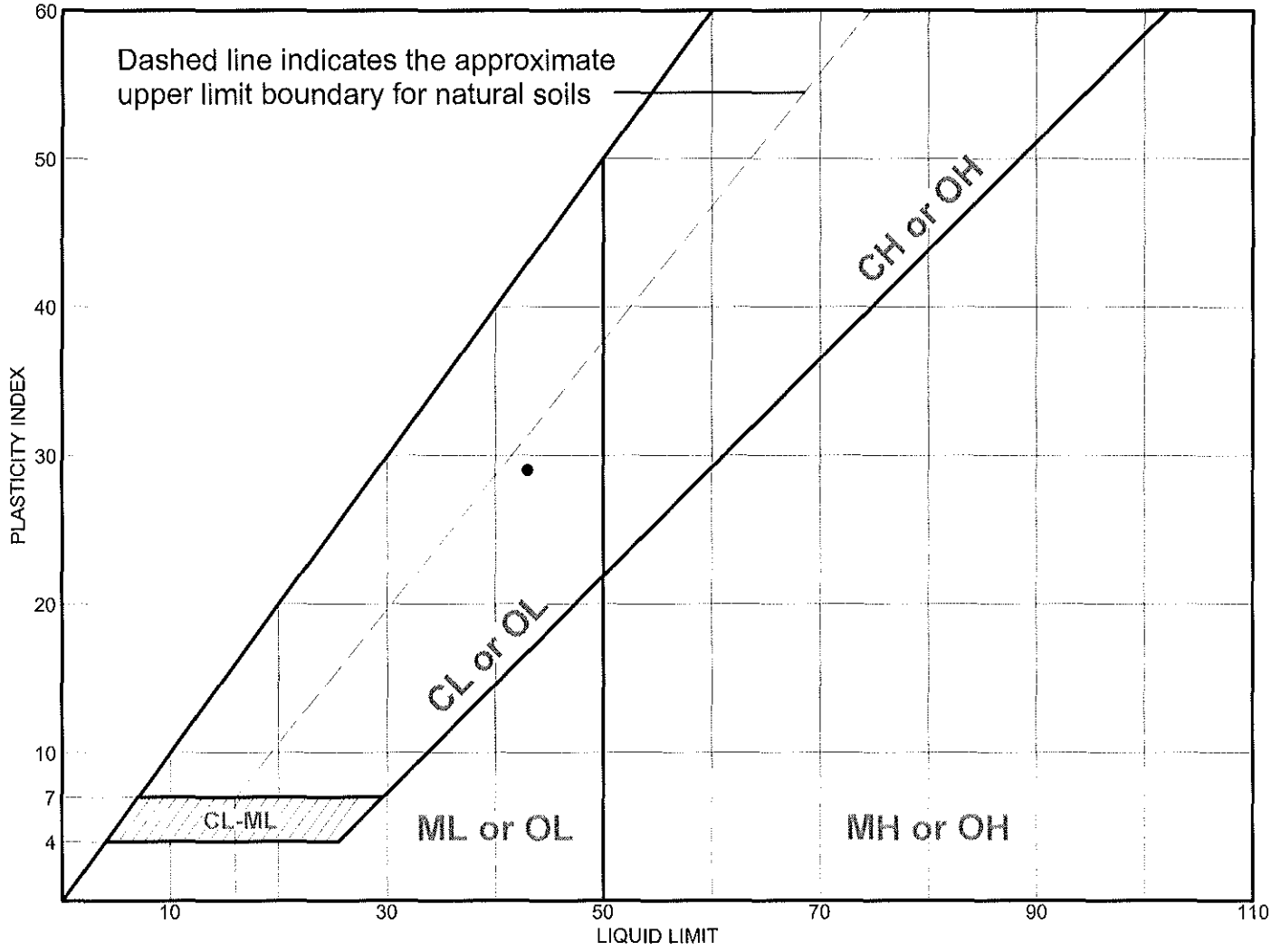
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay with sand	43	14	29	97.8	77.3	CL

Project No. E53606.01 **Client:**

Project: City of Los Banos Storm Drain

● **Source:** **Sample No.:** B-7 **Elev./Depth:** 20-21.5 Feet

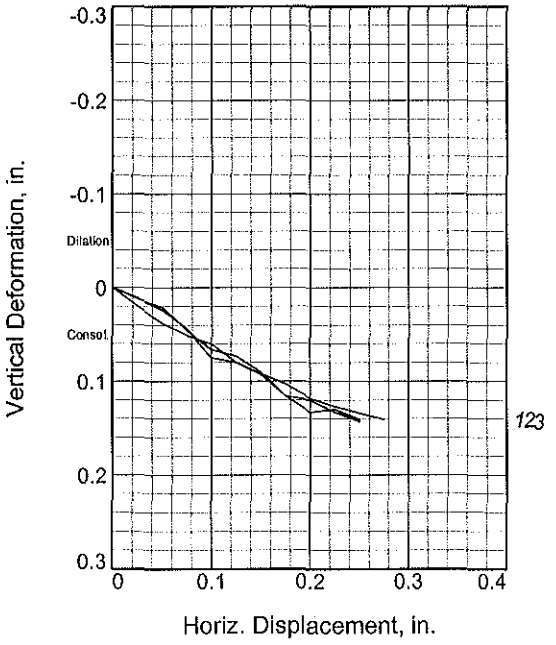
Remarks:

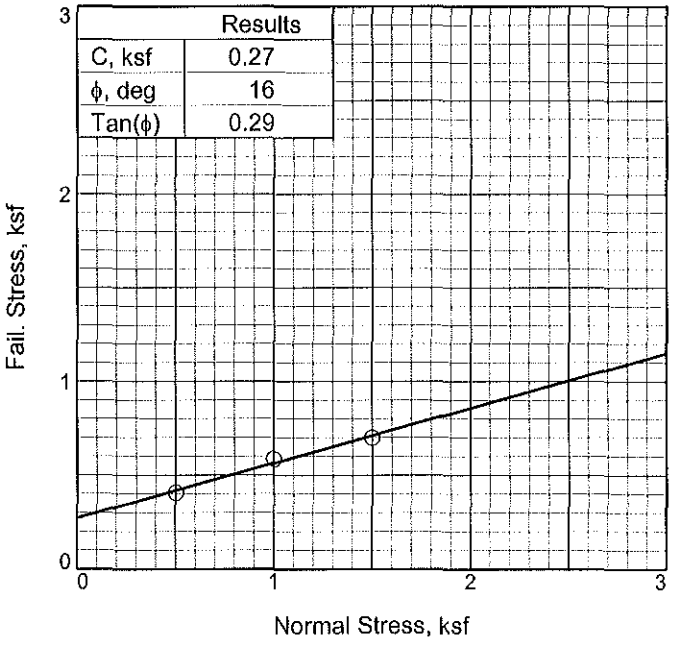
●

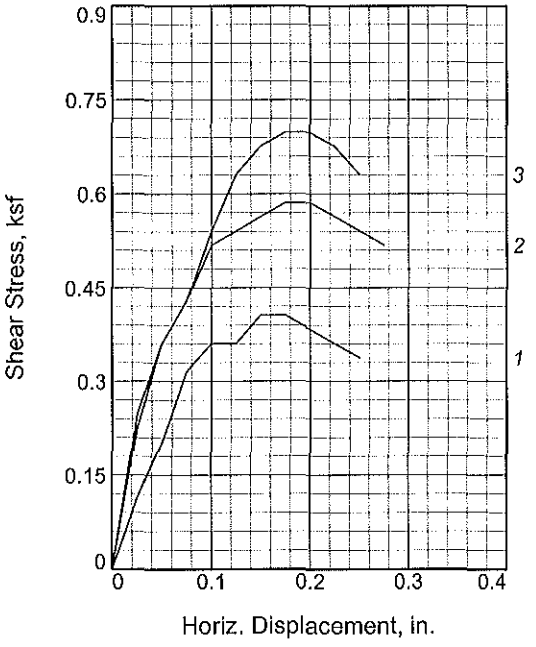
Moore Twining Associates, Inc.

Fresno, CA

Figure







Sample No.		1	2	3
Initial	Water Content, %	20.7	19.7	19.8
	Dry Density, pcf	103.0	102.0	101.7
	Saturation, %	90.3	84.1	83.7
	Void Ratio	0.6069	0.6213	0.6266
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	23.9	23.5	26.2
	Dry Density, pcf	104.1	103.4	103.1
	Saturation, %	107.3	103.9	115.1
	Void Ratio	0.5899	0.6004	0.6039
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.99	0.99
Normal Stress, ksf		0.50	1.00	1.50
Fail. Stress, ksf		0.41	0.59	0.70
Displacement, in.		0.15	0.17	0.17
Ult. Stress, ksf				
Displacement, in.				
Strain at peak, %		6.2	7.2	7.2

Sample Type:
Description:

Assumed Specific Gravity= 2.65
Remarks:

Figure _____

Client:

Project: City of Los Banos Storm Drain

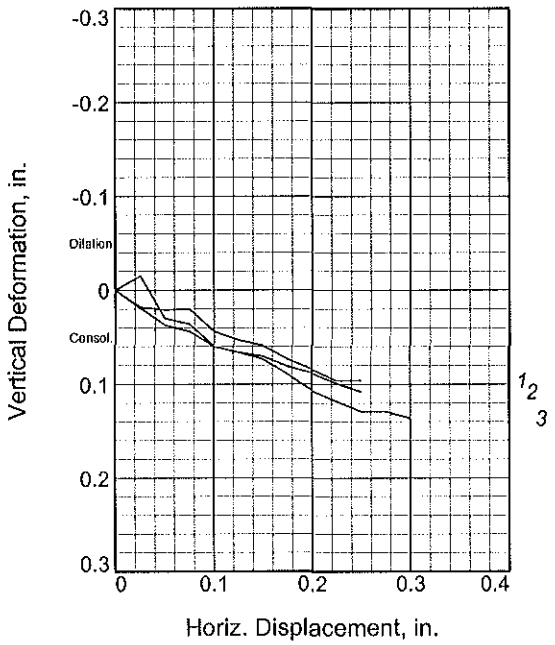
Sample Number: B-4 **Depth:** 8.5-10 Feet

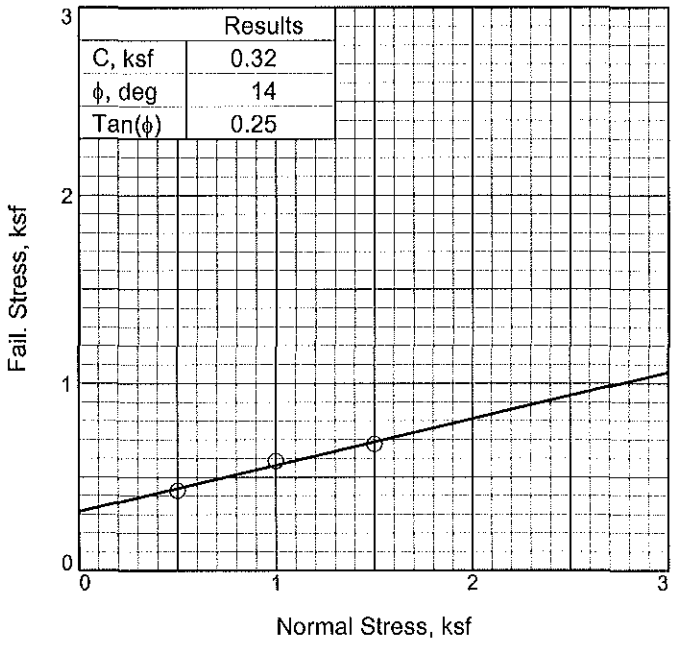
Proj. No.: E53606.01 **Date Sampled:** 02/17/11

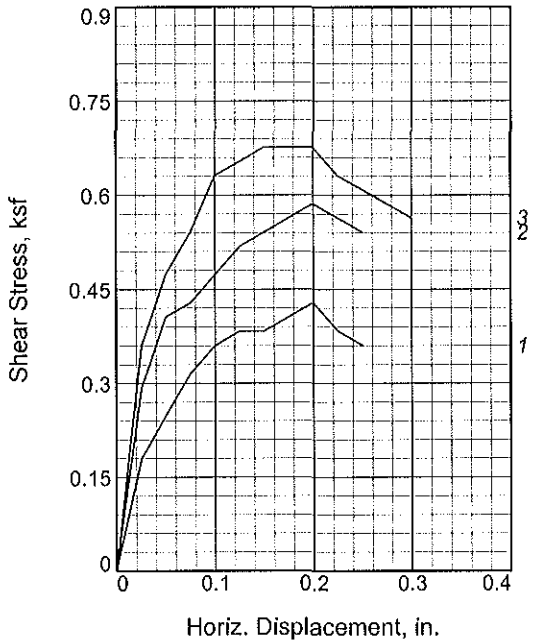
DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

Tested By: TD **Checked By:** MS







Sample No.		1	2	3
Initial	Water Content, %	15.5	15.9	15.8
	Dry Density, pcf	104.9	105.3	104.6
	Saturation, %	71.5	73.7	71.8
	Void Ratio	0.5765	0.5715	0.5815
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.2	20.0	20.7
	Dry Density, pcf	106.0	106.6	106.1
	Saturation, %	95.4	95.8	98.3
	Void Ratio	0.5614	0.5525	0.5593
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.99	0.99
Normal Stress, ksf		0.50	1.00	1.50
Fail. Stress, ksf		0.43	0.59	0.68
Displacement, in.		0.20	0.20	0.15
Ult. Stress, ksf				
Displacement, in.				
Strain at peak, %		8.3	8.3	6.2

Sample Type:
Description:

Assumed Specific Gravity= 2.65
Remarks:

Figure _____

Client:

Project: City of Los Banos Storm Drain

Sample Number: B-7 **Depth:** 5-6.5 Feet

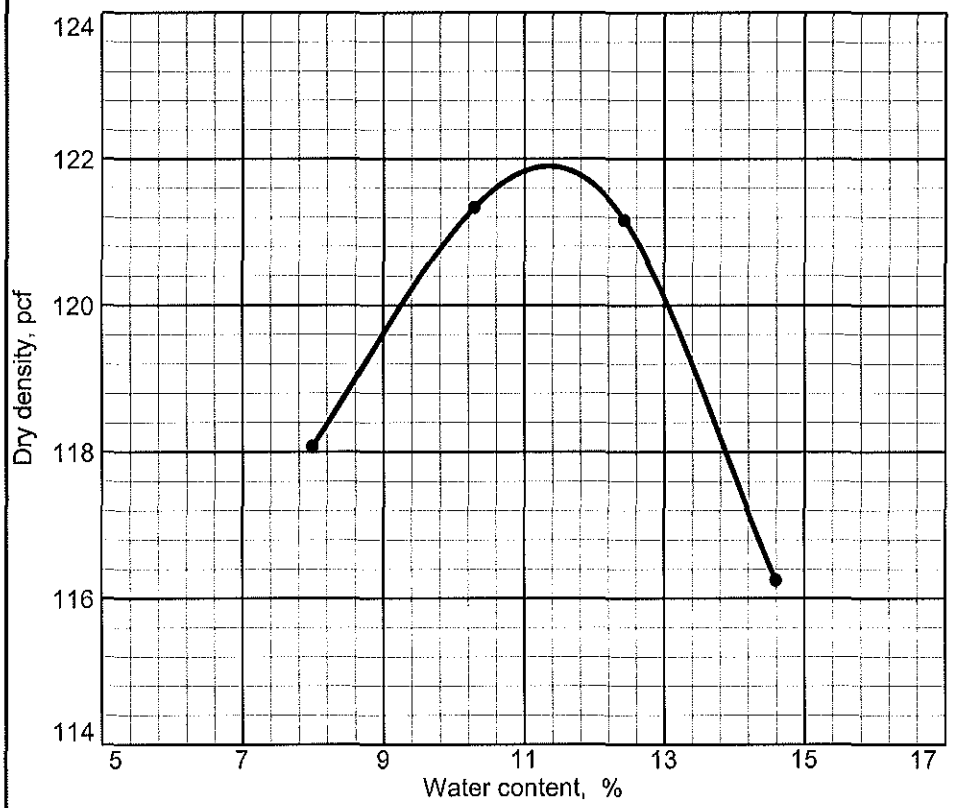
Proj. No.: E53606.01 **Date Sampled:** 02/17/11

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

Tested By: TD **Checked By:** MS

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-09 Procedure A Modified

Hammer Wt.: 10 lb.
Hammer Drop: 18 in.
Number of Layers: five
Blows per Layer: 25
Mold Size: .03333 cu.ft.

Test Performed on Material Passing No.4 **Sieve**

Soil Data

NM _____ **Sp.G.** _____
LL _____ **PI** _____
%>No.4 _____ **%<#200** _____
USCS _____ **AASHTO** _____

TESTING DATA

	1	2	3	4	5	6
WM + WS	8.55	8.76	8.84	8.74		
WM	4.30	4.30	4.30	4.30		
WW + T #1	250.00	256.30	238.90	241.10		
WD + T #1	231.50	232.40	212.50	210.40		
TARE #1	0.00	0.00	0.00	0.00		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	8.0	10.3	12.4	14.6		
DRY DENSITY	118.1	121.3	121.2	116.3		

TEST RESULTS

Maximum dry density = 121.9 pcf

Optimum moisture = 11.3 %

Material Description

Project No. E53606.01 **Client:**
Project: City of Los Banos Storm Drain

● **Source:** **Sample No.:** B-3 **Elev./Depth:** 0-10 Feet

Remarks:

Moore Twining Associates, Inc.
Fresno, CA

Figure

MOORE TWINING
ASSOCIATES, INC.

California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

March 07, 2011

Work Order #: 1B22029

Ken Clark
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: City of Los Banos Storm Drain

Enclosed are the analytical results for samples received by our laboratory on 02/22/11 . For your reference, these analyses have been assigned laboratory work order number 1B22029.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.

Juliane Adams
Director of Analytical Chemistry

MOORE TWINING ASSOCIATES, INC.

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Fresno, CA 93721
(559) 268-7021 Phone
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California ELAP Certificate #1371

MTA Geotechnical Division	Project: City of Los Banos Storm Drain	
2527 Fresno Street	Project Number: E53606.01	**Reported:**
Fresno CA, 93721	Project Manager: Ken Clark	03/07/11

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
B1@0-1.5	1B22029-01	Soil	02/17/11 00:00	02/22/11 14:06
B3@0-1.5	1B22029-02	Soil	02/17/11 00:00	02/22/11 14:06
B6@5-10'	1B22029-03	Soil	02/17/11 00:00	02/22/11 14:06

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MTA Geotechnical Division	Project: City of Los Banos Storm Drain	
2527 Fresno Street	Project Number: E53606.01	**Reported:**
Fresno CA. 93721	Project Manager: Ken Clark	03/07/11

B1@0-1.5

1B22029-01 (Soil) Sampled:02/17/11 00:00

Analyte	Notes.	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**10**	6.0	mg/kg	3	T1B2212	02/24/11	02/25/11	ASTM D-4327-84
Chloride		**0.0010**	0.00060	% by Weight	3	[CALC]	02/24/11	02/25/11	ASTM D4327-84
Sulfate as SO4		**0.0053**	0.00060	% by Weight	3	[CALC]	02/24/11	02/25/11	ASTM D4327-84
pH		**7.4**	0.30	pH Units	3	T1B2212	02/24/11	02/25/11	ATSM D4972-89 Mod
Resistivity		**13000**		ohms-cm	3	T1B2212	02/24/11	02/25/11	ASTM D1125-82
Sulfate as SO4		**53**	6.0	mg/kg	3	T1B2212	02/24/11	02/25/11	ASTM D4327-84

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ASSOCIATES, INC.

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California ELAP Certificate #1371

MTA Geotechnical Division	Project: City of Los Banos Storm Drain	
2527 Fresno Street	Project Number: E53606.01	**Reported:**
Fresno CA, 93721	Project Manager: Ken Clark	03/07/11

B3@0-1.5

1B22029-02 (Soil) Sampled:02/17/11 00:00

Analyte	Notes.	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**9.4**	6.0	mg/kg	3	T1B2212	02/24/11	02/25/11	ASTM D-4327-84
Chloride		**0.00094**	0.00060	% by Weight	3	[CALC]	02/24/11	02/25/11	ASTM D4327-84
Sulfate as SO4		**0.0035**	0.00060	% by Weight	3	[CALC]	02/24/11	02/25/11	ASTM D4327-84
pH		**8.0**	0.30	pH Units	3	T1B2212	02/24/11	02/25/11	ATSM D4972-89 Mod
Resistivity		**13000**		ohms-cm	3	T1B2212	02/24/11	02/25/11	ASTM D1125-82
Sulfate as SO4		**35**	6.0	mg/kg	3	T1B2212	02/24/11	02/25/11	ASTM D4327-84

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California ELAP Certificate #1371

MTA Geotechnical Division	Project: City of Los Banos Storm Drain	
2527 Fresno Street	Project Number: E53606.01	**Reported:**
Fresno CA. 93721	Project Manager: Ken Clark	03/07/11

B6@5-10'

1B22029-03 (Soil) Sampled:02/17/11 00:00

Analyte	Notes.	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**290**	30	mg/kg	15	T1B2212	02/24/11	02/28/11	ASTM D-4327-84
Chloride		**0.029**	0.0030	% by Weight	15	[CALC]	02/24/11	02/28/11	ASTM D4327-84
Sulfate as SO4		**0.014**	0.00060	% by Weight	3	[CALC]	02/24/11	02/25/11	ASTM D4327-84
pH		**9.6**	0.30	pH Units	3	T1B2212	02/24/11	02/25/11	ATSM D4972-89 Mod
Resistivity		**1400**		ohms-cm	3	T1B2212	02/24/11	02/25/11	ASTM D1125-82
Sulfate as SO4		**140**	6.0	mg/kg	3	T1B2212	02/24/11	02/25/11	ASTM D4327-84

Notes and Definitions

ug/L	micrograms per liter (parts per billion concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
mg/L	milligrams per Liter (parts per million concentration units)
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference

Inorganics - Quality Control

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch T1B2212										
Blank (T1B2212-BLK1)				Prepared: 02/24/11	Analyzed: 02/25/11					
Chloride	ND	6.0	mg/kg							
Sulfate as SO4	ND	6.0	mg/kg							
LCS (T1B2212-BS1)				Prepared: 02/24/11	Analyzed: 02/25/11					
Chloride	24.2	2.0	mg/kg	25.0		96.6	80-120		20	
Sulfate as SO4	24.0	2.0	mg/kg	25.0		95.9	80-120		20	
LCS Dup (T1B2212-BSD1)				Prepared: 02/24/11	Analyzed: 02/25/11					
Chloride	24.0	2.0	mg/kg	25.0		96.2	80-120	0.506	20	
Sulfate as SO4	24.1	2.0	mg/kg	25.0		96.5	80-120	0.590	20	
Duplicate (T1B2212-DUP1)		**Source: 1B16017-01**		Prepared: 02/24/11	Analyzed: 02/25/11					
Resistivity	27300		ohms-cm		27200			0.271	200	
pH	7.60	0.30	pH Units		7.70			1.31	20	
Chloride	4.52	6.0	mg/kg		4.53			0.265	20	
Sulfate as SO4	4.16	6.0	mg/kg		3.92			5.94	20	

Items for Project Manager Review

LabNumber	Analysis	Analyte	Exception
	Sulfate-Corrosion	(Soil)	Special Units: (mg/kg)
	Resistivity-Corrosion	(Soil)	Special Units: (ohms-cm)
	pH - Corrosion	(Soil)	Special Units: (pH Units)
	Chloride-Corrosion	(Soil)	Special Units: (mg/kg)
			VERSION 6.07:2008
			Default Report (not modified)

APPENDIX D
SPECIFICATIONS FOR DEWATERING

The contractor should anticipate wet or saturated soils near the base on the excavations required for the pipeline and basin. The contractor is responsible for all dewatering to construct the project in accordance with the plans and specifications, including but not limited to trenches, excavations, etc.

Dewatering General: As required to construct the project, the contractor should design and provide dewatering systems using accepted and professional methods consistent with current industry practice to eliminate water entering the excavation under hydrostatic head from the bottom and/or sides. The system should be designed to prevent differential hydrostatic head, which would result in floating out soil particles in a manner, termed as a quick or a boiling condition. System shall not be dependent solely upon sumps and/or pumping water from within the excavation where differential head would result in a quick condition, which would continue to worsen the integrity of the excavations' stability.

The contractor should provide dewatering systems of sufficient size and capacity to prevent ground and surface water flow into the excavation and to allow all Work to be installed in a dry condition.

The contractor should control, by acceptable means, all water regardless of source and the contractor is fully responsible for disposal of the water in a manner that is acceptable to all applicable agencies with jurisdiction.

The contractor should confine discharge piping and/or ditches to available easement or to additional easement obtained by Contractor. The contractor shall provide necessary permits and/or additional easement at no additional cost to Owner.

The contractor should control groundwater in a manner that preserves the strength of foundation soils, does not cause instability or raveling of excavation slopes, and does not result in damage to existing structures and offsite improvements. Where necessary to these purposes, the contractor should lower the water level in advance of excavation, utilizing wells, wellpoints, jet educators, or similar positive methods. The water level as measured by piezometers shall be maintained a minimum of 3 feet below prevailing excavation level or as necessary to achieve the required grading site preparation, and construction of the specified improvements.

The contractor should commence dewatering prior to any appearance of water in excavation and continue until Work is complete to the extent that no damage results from hydrostatic pressure, flotation, or other causes.

Open pumping with sumps and ditches shall be allowed, provided it does not result in boils, loss of fines, softening of the ground, or instability of slopes.

The contractor should install wells and/or wellpoints, if required, with suitable screens and filters, so that continuous pumping of fines does not occur. Discharge should be arranged to facilitate collection of samples by the Owner. During normal pumping, and upon development of well(s), levels of fine sand or silt in the discharge water shall not exceed five parts per million and comply with the current SWPPP requirements, whichever is more stringent. The contractor is responsible to install a sand tester on discharge of each pump during testing and conduct necessary water quality tests to verify that levels or RWQCB requirements are not exceeded.

The contractor should control grading around excavations to prevent surface water from flowing into excavation areas. No additional payment will be made for any supplemental measures to control seepage, groundwater, or artesian head.

Dewatering Design: The contractor should designate and obtain the services of a qualified dewatering specialist to provide dewatering plan as may be necessary to complete the Work.

The contractor should be responsible for the accuracy of the drawings, design data, and operational records required, and the contractor is solely responsible for the design, installation, operation, maintenance, and any failure of any component of the system.

Dewatering Damages: The contractor shall be responsible for and shall repair without cost to the Owner any damage to work in place, or other contractor's equipment, utilities, residences, highways, roads, railroads, private and municipal well systems, adjacent structures, natural resources, habitat, existing wells, and the excavation. Including, damage to the bottom due to heave and including but not limited to, removal and pumping out of the excavated area that may result from Contractors negligence, inadequate or improper design and operation of the dewatering system, and any mechanical or electrical failure of the dewatering system.

The contractor is responsible to remove subgrade materials rendered unsuitable by excessive wetting and replace with approved backfill material at no additional cost to the Owner.

Maintaining Excavation in Dewatering Condition: Dewatering shall be a continuous operation. Interruptions due to power outages, or any other reason will not be permitted.

The contractor should continuously maintain excavation in a dry condition with positive dewatering methods during preparation of subgrade, installation of pipe, and construction of structures until the critical period of construction and/or backfill is completed to prevent damage of subgrade support, piping, structure, side slopes, or adjacent facilities from flotation or other hydrostatic pressure imbalance.