APPENDIX B:

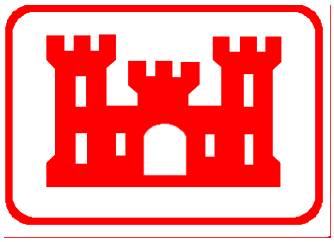
Government Geotechnical Report

GOVERNMENT GEOTECHNICAL REPORT

FOR DESIGN-BUILD RFP

ICE SECURE HOUSING UNIT

PORT ISABEL, TEXAS



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U.S. ARMY CORPS OF ENGINEERS

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ENGINEERING AND CONSTRUCTION DIVISION

GEOTECHNICAL BRANCH

CESWF-ECG

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GOVERNMENT GEOTECHNICAL REPORT

1. General

The purpose of this report is to provide subsurface information, foundation design, and subgrade preparation considerations, guidance, and requirements for the planned Secure Housing Unit (SHU) at the Immigration and Custom Enforcement (ICE), Service Processing Center (SPC) campus in Port Isabel, Texas. Design and construction of the project will be accomplished under a Design-Build (D-B) contract.

The project requires the design and construction of a new, single-story SHU consisting of 26 modular cells, two of which must comply with Architectural Barriers Act (ABA) Standards; three-day rooms; three ABA-compliant showers; detainee search; storage spaces; security office control room; desk officer office; vestibules; and recreation yards. The new building must be architecturally compatible with other structures on the SPC campus. There are two sidewalks planned to the southwest of the SHU building connecting to Semper Fi Avenue. No other pavements are planned for this project.

The new SHU is in the northwestern quadrant of the Port Isabel Detention Center campus. The site is bounded by Semper Fi Avenue to the southwest, the existing kitchen facility and access drive to the northwest, Memorial Drive to the northeast, and the recreational soccer field to the southeast. The site is currently being used as a drainage swale and stormwater detention area for the kitchen facility. The site is within a security fence for the kitchen facility and soccer field.

If project descriptions and assumptions in the section are not correct, the Geotechnical Branch of the U.S. Army Corps of Engineers, Fort Worth District (CESWF-ECG) should be contacted to determine if the recommendations presented in this report need to be reevaluated.

2. Subsurface Investigation

A site-specific subsurface investigation was not performed at the site. The subsurface investigation performed previously at the site of the kitchen facility located immediately northwest of the planned SHU was

used to develop the geotechnical recommendations for the planned SHU. The borings performed previously for the wastewater treatment plan (WWTP) improvements project, approximately 0.35 miles southeast of the planned SHU, were also considered. The locations of the planned SHU, the existing kitchen facility and WWTP are shown in Appendix I of the report. The D-B Contractor shall perform a site-specific subsurface investigation at the site of the planned SHU to obtain the actual soil conditions, which may differ from those encountered at the sites of the kitchen and WWTP.

The previous subsurface investigations at the sites of the existing kitchen facility and WWTP were performed by ETTL Engineers & Consultants, Inc (ETTL) between January 8 and 11, 2019 and consisted of a total of eleven (11) borings, seven (K-1 through K-7) for the kitchen facility and four (W-1 through W-4) for the WWTP. The field investigations were performed using a truck-mounted CME-55 drill rig. Test hole advancement and sample recovery was performed using hollow-stem augers, nominal 3-inch diameter Shelby tube samplers and 2-inch diameter split-spoon samplers.

2.1 Groundwater Conditions

Groundwater conditions were monitored during and upon completion of drilling operations, and after an observation period of up to 21.3 hours after completion of drilling operations. Groundwater level measurements in the borings are presented in the table below.

Water Level Elevation Measurements

Boring No	Ground Surface Elevation (ft, NAVD 88)	Water Level Elevation During Drilling (ft, NAVD 88)	Water Level Elevation at Completion of Drilling (ft, NAVD 88)	Water Level Elevation After Observation Periods indicated in Parenthesis (ft, NAVD 88)
K-1	17.4	8.4	-3.0	8.3 (5 hrs.)
K-2	17.8	-2.5	-7.0	7.2 (1.8 hrs.)
K-3	17.9	9.4	-2.1	Dry (3.7 hrs.)
K-4	18.6	-2.4	-8.2	8.1 (5.2 hrs.)
K-5	17.9	-0.8	-11.6	Dry (10 mins)
K-6	18.7	4.9	-9.4	6.3 (3.5 hrs.)
K-7	17.7	3.2	-8.1	Dry (5 mins)
W-1	11.9	-7.6	-8.6	6.5 (21.3 hrs.)
W-2	12.1	-0.4	-3.5	6.9 (16.5 hrs.)
W-3	12.2	-0.3	-2.0	-2.0 (10 mins)
W-4	17.1	1.9	-3.5	Dry (17.8 hrs.)

It should be noted that groundwater conditions are relative to the time of drilling, annual precipitation, and drainage conditions at the site. Fluctuations in groundwater levels shall be expected throughout the year depending on variations in amount of precipitation, surface runoff, and other hydrological factors not apparent at the time of the field explorations.

3. Subsurface Conditions

3.1 General Geology

The site is located within the Coastal Prairies of the West Gulf Coastal Plains Section of the Coastal Plains physiographic province. The Coastal Prairies along the Gulf of Mexico are dominated by Pleistocene deltaic sands, silts and clays. The geologic materials at the site are alluvium in Rio Grande, subdivided into areas predominately of sand, Quaternary and Holocene in age. The alluvium in Rio Grande consists of point-bar and distributary yellowish-gray or brownish-gray quartz sand and silt and contains plant debris and some biotite. Accretionary point-bar, levee, and crevasse-splay deposits formed by distributary channels on the Rio Grande delta. Point-bar deposits grade upward from cross bedded basal sandy gravel and coarse sand to silt and fine sand and locally are as thick as 40 feet.

3.2 Subsurface Soil Conditions

At the kitchen facility site, the borings (K-1 through K-7) generally encountered low plasticity lean clay, silty sand, clayey sand and silt to depths of about 4.5 to 19.5 feet below ground surface underlain by high plasticity fat clay to the boring termination depths of about 26.5 to 31.5 feet below ground surface. A layer of silt was encountered in the fat clay stratum starting at 14.5 to 17 feet and extending to about 18 to 19.5 feet in five of the seven borings drilled in the kitchen area. The clays and silts exhibited very soft to very stiff, generally soft to medium stiff, in consistency and the sands exhibited very loose to medium dense in relative density. Liquid limits measured from representative samples of the silt and lean clay soils ranged from 29 to 47, plastic limits varied from 15 to 26 (with plasticity indices ranging from 3 to 28). Liquid limits measured from representative samples of the fat clay soils ranged from 50 to 78, plastic limits varied from 18 to 26 (with plasticity indices ranging from

30 to 53).

At the WWTP site, the borings (W-1 through W-4) generally encountered high plasticity fat clay starting from the ground surface and extending to the boring termination depths of about 21.5 to 31.5 feet below ground surface. Lean clay and silt layers/seams with varying thicknesses were generally observed in the fat clay at varying depths. The fat clay soils were soft to stiff in consistency. Liquid limits measured from representative samples of the fat clay soils ranged from 57 to 75, plastic limits varied from 20 to 28 (with plasticity indices ranging from 33 to 49).

At both sites, the upper 2 feet of the subgrade soils were dark brown in color and contained root material that appeared to be from former crops. It was believed that the upper 2 feet of subgrade soils likely represent an old plow zone left over from when the SPC campus was farmland prior to its development.

The boring location plan, boring logs along with the laboratory test results and plots for the kitchen facility and WWTP improvements projects are included in Appendix II. The legend on the individual boring logs shows overburden materials as classified in the laboratory using procedures presented in ASTM D 2488. It should be noted that the actual interface between material types might be far more gradual or abrupt than presented; therefore, actual subsurface conditions in areas not sampled may differ from those predicted. The nature and extent of variations across the sites may not become evident until construction commences, and the actual construction process may alter subsurface conditions as well. If variations become evident at the time of construction, CESWF-ECG should be contacted to determine if the recommendations presented in this report need to be reevaluated.

4. Laboratory Testing

Representative soil samples recovered from selective test holes at the kitchen facility and WWTP sites were subjected to laboratory testing for identification, moisture content, grain-size distribution, Atterberg limits, density, unconfined compressive strength, modified Proctor, and one-dimensional swell/collapse. Analytical tests including resistivity, redox potential, soil pH, water soluble sulfide, sulfate, and chloride content, were also

performed on selected near-surface soil samples. The test results and plots are presented in Appendix II.

4.1 Swell/Collapse Testing.

Swell/collapse testing was performed on four specimens, one fat clay specimen and three lean clay specimens, in accordance with ASTM D4546. The lean clay specimens were collected at different depths ranging from about 3.5 to 10 feet below ground surface. These lean clay specimens exhibited liquid limits ranging between 36 and 42, plastic limits ranging between 19 and 24, plasticity indices ranging between 17 and 18, and natural moisture content ranging between 14.7 and 30.9 percent. The natural moisture content range is about 4.3 percent lower to 6.9 percent higher than their plastic limit. Under overburden pressure, the lean clay specimens had a collapse of about 0.18 percent for one specimen, and swell ranging from 0.42 to 0.63 percent for other two specimens. The fat clay specimen that was tested was collected at a depth of about 5.3 feet below ground surface and exhibited a liquid limit of 53, a plastic limit of 21, and a plasticity index of 32. The natural moisture content of the specimen was 10.4 percent. The fat clay specimen has a collapse of about 0.02 percent under overburden pressure.

Based on swell/collapse test results, the clay specimens appeared to have a low expansion potential. However, it should be noted that the expansion potential could increase when the native soils become drier. Swell/collapse test results are presented in Appendix II of this report.

4.2 Shear Strength Testing.

Shear strength characteristics of select clayey samples were analyzed in the laboratory using unconfined compressive strength (ASTM D2166) testing. Tabulated below are the compressive strengths and dry densities of the clayey specimens that were tested. Shear strength test results are also presented in Appendix II at the end of this report.

Boring	Depth, feet	γ_d, pcf	Q_u, psf	Material Type
K-4	2.8-3.3	107.1	4,710	Lean Clay
K-6	12.5-13	92.1	1,420	Lean Clay
W-2	7.5-8	90.9	1,650	Fat Clay
W-4	5.2-5.7	93.6	4,230	Lean Clay

4.3 Analytical Testing.

Analytical tests including resistivity, redox potential, soil pH, water soluble sulfide, sulfate, and chloride content were performed on three select clay specimens. The results are summarized in the following table.

Results of Analytical Testing

Boring No	Depth (ft)	Resistivity (ohm-cm)	Redox Potential (mV)	pH	Sulfides (mg/kg)	Chlorides (mg/kg)	Sulfates (mg/kg)
K-4	2 to 5	300	186	8.08	ND*	1,500	970
W-1	2 to 5	1,200	196	8.22	ND*	110	1,900
W-4	2 to 5	280	190	8.09	ND*	1,600	870

*Not detected

5. Discussions

The following discussions are provided in support of foundation design and subgrade preparation recommendations and requirements made for the planned SHU building in the ICE IPC campus in Port Isabel, Texas. It should be noted that a site-specific field investigation was not performed at this site and the discussions presented herein are based on the results of the geotechnical field investigation conducted previously at the sites of the kitchen and WWTP improvements projects, laboratory testing program conducted on soil samples collected at the kitchen and WWTP sites, our past experiences, as well as engineering studies.

The Design-Build (D-B) Contractor shall heed the information provided in this report and comply with the recommendations and requirements presented herein**.** The Design-Build Contractor's designs are required to comply with and to meet or exceed the minimum foundation design and subgrade preparation recommendations and requirements presented herein. The subsurface information presented in this report may be used by the bidders for this D-B contract for the purposes of developing a bid for the project in line with the Request for Proposal (RFP) solicitation. The successful D-B Contractor bidder shall supplement the information provided herein with his/her own site-specific geotechnical field investigation at the site and laboratory testing program for the purpose of verifying the data presented herein with the actual site conditions, which may differ. Supplemental geotechnical field investigations conducted by the D-B Contractor shall be only for the purpose of supplementing and verifying

the data regarding the subsurface conditions provided by the Government geotechnical field investigation at the referenced site. Additional test holes for the foundation design shall each be drilled to a minimum depth of 30.0 feet below existing grade. All borings and test holes shall be grouted full depth with a lean grout mixture to seal holes from water penetration. Laboratory testing shall be conducted on soil specimens as described in Section 4 (Laboratory Testing), including but not limited to classification (ASTM D 2488), moisture content (ASTM D 2216), grain size analysis (ASTM D 422), Atterberg limits (ASTM D 4318), shear strength (ASTM D2166 and/or D2850 for cohesive soils, and ASTM D7012 for rock), swell/collapse test (ASTM D 4546), and modified Proctor (ASTM D1557).

Development of the final foundation designs is the responsibility of the D-B Contractor; however, the D-B Contractor's final foundation designs and subgrade preparation shall be in full compliance with the requirements prescribed herein (including foundation type, foundation design parameters, and minimum subgrade preparation). The D-B Contractor shall provide to the Government engineering studies and design calculations that support the foundation design recommendations they or their associates propose. The D-B Contractor's foundation design recommendations shall be reviewed for technical adequacy and compliance with the requirements and criteria established herein and in the Request for Proposal (RFP). Specific requirements for the D-B Contractor's foundation design analysis are provided in Section 7.

5.1 Foundation Design Considerations

The foundation design recommendations and requirements presented in this report are based on criteria contained in UFC 3-220-01, and engineering judgment.

The foundation for the proposed SHU building must meet several criteria. The foundation must be compatible with the superstructure it supports, its movements must be within acceptable tolerances, it must meet functional requirements of the facility, and it must be economical. The SHU building is a single-story structure and structural loads are anticipated to be relatively light. A deep foundation system is not considered well-suited to or cost effective for lightly loaded facilities, and it is recommended a shallow foundation system is used to support the planned building.

The previous subgrade explorations were performed during January 2019. Even though based on the lab results, the expansion potential of the native clay soils was relatively low at their then moisture content at the time of the previous subsurface explorations, the native high plasticity subgrade soils could become drier, especially during summers and their expansion potential could increase with decrease in in-situ moisture content. A shallow foundation system such as dug and formed spot and continuous spread footings founded on high plasticity clays could experience relatively large differential movement due to shrink-swell movements of the foundation soils. Due to the heave potential of the underlying clays and the cost of framework construction, the shallow foundation system such as spot and continuous spread footings is not considered a viable foundation system for the proposed structure.

Based on the available facility information and results of the previous investigations conducted at the sites of the kitchen and WWTP improvements projects and previous engineering experience with similar structures on high plasticity clay soils, a shallow foundation system consisting of either a reinforced concrete ribbed mat slab, or a reinforced concrete flat mat slab shall be used for the planned SHU building. Recommendations for design and construction of the reinforced concrete ribbed mat slab or a reinforced concrete flat mat slab are presented below.

Properly designed and constructed reinforced concrete ribbed mat slab and reinforced concrete flat mat slab foundation systems have performed well in other military installations with high plasticity clay subgrade soils, particularly for facilities with relatively light structural loads, which are anticipated for the planned SHU building. These two shallow foundation systems, when properly designed and constructed, will act monolithically when subjected to shrink-swell movements of the subsurface materials, in contrast to other, unsuitable shallow foundation systems such as spot and continuous spread footings.

6. Recommendations and Requirements

The following foundation design and subgrade preparation recommendations and requirements are based on results of the field investigations and laboratory testing performed at the kitchen and WWTP sites, and

engineering studies. The Design-Build Contractor's designs are required to comply with and to meet or exceed the minimum design recommendations and requirements provided in the section.

6.1 Mat Foundation Design Recommendations and Requirements

The proposed structures shall be founded on either a reinforced concrete ribbed mat or flat mat foundation system, in accordance with the recommendations and requirements specified herein. The mat slabs shall be conventionally reinforced – post-tensioned slabs are not allowed. Based on the plasticity level of the near surface clayey soils, the mat slabs shall be analyzed and designed for 1 inch of long-term differential movement, provided the subgrade preparation is performed in accordance with the recommendations in Section 6.2 of this report. For this reason, interior ribs for ribbed mat slabs shall be spaced no farther than 20 feet center-to-center, and diagonal stiffener ribs shall be placed at each corner of the mat slab. Design of the mat slabs shall meet the minimum requirements as presented in CESWD-ED-TS/G Criteria Letter, dated 29 January 1988 – Design Criteria for Ribbed Mat Foundations, SWDED-G Criteria Letter, dated 16 April 1987 – Criteria for Developing Geotechnical Design Parameters for SWD Ribbed Mat Design Methodology, and the recommendations and requirements provided herein in Appendix III.

Interior and exterior beams for ribbed mat slabs shall bottom a minimum of 24 inches below outside finished grade. An allowable bearing capacity of 2.0 ksf (net) shall be used to size the beams. For this phase of design, it should be noted that (1) the structural load is supported solely on the beam and the beam intersections, (2) load transfer occurs over the effective beam width, and (3) the beam and soil remain in contact. Beam intersections shall be widened at column locations to accommodate the above allowable bearing value for the anticipated load condition. The load used to size the beams shall consist of full dead load plus that portion of the live load that acts more or less continuously, usually 50 percent.

The ribbed mat slab foundations shall incorporate adequate stiffness such that the deformations do not exceed the structural tolerance of any elements in the foundation or superstructure. Analyses shall consider a vertical separation of the foundation slab and beams from the subgrade of 1.0 inch at the outside of all perimeter beams, with loss of support beneath the foundation over a horizontal distance of not less than 5.5 feet. This loss

of support condition corresponds to the center lift mode. Additionally, edge lift analyses shall consider an edge moisture variation distance equal to 2.5 feet, and an edge lift heave of 1.0 inch shall be used in the design of the ribbed mat slab. This edge lift heave corresponds to an applied structural pressure of 100 psf. For edge lift considerations, two additional combinations of pressure and swell are required. For an allowable bearing capacity of 2.0 ksf, an edge lift heave of 0.75 inch can be expected to occur. At an ultimate bearing capacity of 6.0 ksf, 0.50-inch of heave should be anticipated. It should be noted that these anticipated heave amounts are based on the removal of a minimum of 3.0 feet of existing materials and replacement with compacted non-expansive structural fill, as required herein.

A modulus of subgrade reaction equal to 200 psi/inch shall be used when analyzing the ribbed mat slabs to determine in-service deformations provided that the subgrade preparation and fill requirements as specified below are carried out. This value, however, shall be factored to account for width effects such that $k_{design}=k_1/(B_{eff})$, where k_1 is the modulus of subgrade reaction and B_{eff} is the effective beam width in feet. Design of the ribbed mat slabs may use the *SWD-AEIM* sections as a minimum stiffness "first approximation".

A flat mat shall have a uniform thickness of not less than 18 inches. An allowable bearing capacity of 2.0 ksf (net) shall be used to size the mat foundation. The flat mat shall be tapered as required to ensure the perimeter of the slab extends to a constant elevation and is at least 24 inches below outside finish grade. Flat mat slabs shall also incorporate adequate stiffness such that the deformations do not exceed the structural tolerance of any element of the foundations or superstructures nor cosmetic cracking of interior and exterior finishes. Foundation movement is expected to occur despite the removal and replacement requirements. Therefore, the structure shall incorporate appropriate control joints within interior finishes and exterior facades to mitigate the potential for structural or cosmetic crack development caused by foundation movement. The load used to size the flat mat slab shall consist of full dead load plus that portion of the live load that acts more or less continuously, usually 50 percent.

The mat slabs will, by design, be supported on-grade. A polyethylene vapor barrier (10-mil minimum thickness) and a minimum 6-inch capillary water barrier shall be placed beneath the mat slab. The polyethylene

vapor barrier and 6-inch capillary water barrier are not required beneath mat slabs that will not be enclosed (i.e., mechanical equipment pads that will be completely open and exposed to the elements); these mat slabs can be founded directly on the compacted non-expansive structural fill.

6.2 Subgrade Preparation and Fill Requirements

The upper 3.0 feet (minimum) of existing soils within the structure footprint shall be removed and replaced with compacted non-expansive structural fill, which should limit the magnitude of predicted movement to approximately 1.0 inch or less, remove the existing soils associated with the old plow zone and provide a more uniform and strong foundation support. The over-excavation shall also extend laterally at least 5 feet beyond the perimeters of the structure.

The site is currently being used as a drainage swale and stormwater detention area for the existing kitchen facility. It is anticipated that soft and loose deposits are present along the drainage swale and near the detention area. These soft and loose deposits, if present, shall be removed full depth and replaced with non-expansive structural fill. Over-excavation depths of over 3 feet may be required to remove these soft and loose deposits.

Any additional fill required to achieve the final subgrade elevation below the mat foundation shall be non-expansive structural fill material. The upper 6 inches of existing subgrade exposed after excavation operations, or cleared prior to fill placement shall be scarified, moistened, manipulated, and recompacted to at least 90 percent of laboratory maximum density at a moisture content at least 1 percent above the optimum moisture content (in accordance with ASTM D 1557). Non-expansive structural fill, as defined in Section 6.8.4 of the report, shall be placed in controlled lifts not exceeding 8 inches in loose thickness and compacted to at least 95 percent of laboratory maximum density at a moisture content within 2 percent of its optimum moisture content (in accordance with ASTM D 1557).

Based on previous experience, if structural fill is placed outside the limits of the structure footprint, the relatively higher permeability of the structural fill will allow moisture to infiltrate to the highly expansive soils adjacent to and beneath the foundation, potentially resulting in heave of the foundation. To limit moisture penetration to the area around and beneath the foundation, excavated areas beyond the limits of the structure

footprint shall be backfilled with select clay backfill materials The definition of the select clay soils is included in Section 6.8.5 of the report. This select clay cap shall be a minimum of 2 feet in thickness and shall extend from the structure perimeter to the limits of the excavation. Select clay backfill materials shall be compacted to at least 92 percent of laboratory maximum density at a moisture content within 2 percent of its optimum moisture content as determined by ASTM D 1557.

6.3 Seismicity Site Class

The site class for seismic design is classified based on the existing soil properties averaged over a depth of 100 feet and designated with Site Classes A to F, considering Site Class A as hard rock down to Site Class F as potentially collapsible soil. Based on Section 1613.3.2 of the 2015 International Building Code and in accordance with Chapter 20 of ASCE 7, we recommend using Site Class E for seismic design at this site. This recommendation is based on soil samples encountered to the maximum boring termination depth of 31.5 feet.

6.4 Material Testing Requirements

Testing shall be the responsibility of the contractor to ensure that the subgrade, fill, and backfill materials are properly compacted. To this end, the following frequencies of testing shall be included in the contract as a minimum:

- In-place density of the subgrade, fills, and backfills shall be performed for every 2,000 square feet per lift in accordance with ASTM D 1556 or ASTM D 2922.
- Optimum Moisture and Laboratory Maximum Density of non-expansive fill and backfill shall be performed for every 500 cubic yards or when any change in material occurs.

6.5 Drainage Conditions

Proper site drainage is imperative to ensure satisfactory long-term foundation performance. Exterior grading adjacent to the structure shall be sloped away from the structure a minimum of 5 percent for the first 10 feet. Runoff from the roofs shall be adequately discharged away from foundation edges. In no case should water be allowed to pond adjacent to or beneath the structure, both during and after construction.

6.6 Care of Water

Drainage of ground and surface water from the project site continually throughout the construction

contract is essential. The contractor shall be required to protect the excavations and all constructed work throughout the life of the contract by means of ditches, berms, sumps with pumps, and any other means required to always remove water continually and effectively from the site. Ponding of water in excavations is unacceptable at any time. In addition, site excavations to include utility trenching, grading, and retention basins shall be constructed so as not to supply water directly or indirectly to the building or underlying active clays via the more pervious silt and sand stratum that is present at the site. Such exposure of the underlying high plasticity clays to surface runoff during construction could extend the active zone (via fractures) to much greater depths within the fat clay. This would activate these highly expansive materials and result in expansion pressures on the foundation elements that could create distress on the structural system of the facility. The designer shall grade the site to ensure positive drainage of all water away from the structure.

6.7 Mechanical Connections

All exterior mechanical connections shall be of the flexible type. Flexible connections shall be capable of resisting a minimum of 4 inches of both vertical and horizontal movement. All mechanical/structural connections between slabs on grade or building perimeter entry/exit points must be designed and constructed to handle up to 4 inches of vertical movement. All condensate lines shall drain away from foundation edges. It is recommended that below-grade utility lines connecting to the structures be aligned under the centerline of the respective structure footprints. The use of exterior lines connecting to the structures by numerous utility trenches is discouraged, as this would create high permeability pathways for moisture to gain access to, and activate, the expansive clay materials within the structure footprint.

6.8 Material Definitions

6.8.1 Satisfactory Materials.

Satisfactory materials may be used as backfill outside structure areas and include materials classified in ASTM D 2487 as GW, GM, GC, GP, GW-GM, GW-GC, GP-GM, GP-GC, SW, SP, SP- SM, SP-SC, SW-SM, SW-SC, SC, SM, CL, CL-ML, ML, and CH, and shall be free of trash, debris, roots or other

organic matter, or stones larger than 3 inches in any dimension. Satisfactory materials shall not be used in structure areas unless they meet specific requirements as stated in the report.

6.8.2 Unsatisfactory Materials.

Unsatisfactory materials include materials classified in ASTM D 2487 as Pt, OH, OL, and MH, or materials containing trash, debris, roots or other organic matter.

6.8.3 Expansive Soils.

For the purposes of this report, expansive soils are defined as soils having a liquid limit above 35 or a plasticity Index above 15.

6.8.4 Non-expansive Structural Fill.

Non-expansive structural fill should meet the requirements of Texas Department of Transportation Standard Specification for Base Course, Item 247, Type A, Grade 1 or 2, with plasticity index of not less than 4 percent nor greater than 12 percent when tested in accordance with ASTM D 4318, and at least 5 percent of fines (material passing the No. 200 sieve).

6.8.5 Select Clay Soils.

Select clay soils shall be classified as a Lean Clay (CL) and have a liquid limit of 35 percent or less and a plasticity index not less than 8 nor greater than 15 when tested in accordance with ASTM D 4318. Select clay soils should also meet the requirements of Satisfactory Materials as stated under 6.8.1.

6.8.6 Cohesionless and Cohesive Materials.

Cohesionless materials include materials classified in ASTM D 2487 as GW, GP, SW, and SP. Cohesive materials include materials classified as GC, SC, ML, CL, MH, and CH. Materials classified as GW-GM, GW-GC, GP-GM, GP-GC and materials classified as SW-SM, SW-SC, SP-SM, SP-SC may be considered for identification purposes as cohesionless.

6.8.7 Capillary water barrier.

Capillary Water Barrier shall consist of clean, crushed, nonporous rock, crushed gravel, or uncrushed gravel. The maximum particle size shall be 1.5 inches and no more than 2 percent by weight shall pass the 3/16-inch size (No. 4) sieve. ASTM #57 and #67 aggregates may also be specified as capillary water barrier.

7. Requirements for the D-B Contractor's Foundation Design Analysis.

The successful proposer shall provide a Foundation Design Analysis after contract award. The Foundation Design Analysis (Report) shall include a description of the project, including a discussion of any unusual features of the project, and a discussion for each structure that requires a foundation system. The foundation, and material analyses shall be performed, and signed and sealed by a licensed professional engineer.

7.1 Foundation System(s)

The best suitable foundation type shall be recommended for each structure. If more than one foundation system is recommended for different structures, separate subparagraphs shall be used to discuss each foundation system. The subparagraphs shall provide a detailed description of the foundation system as well as specific design and construction requirements. The location and type of structure supported by that foundation system should also be discussed. Foundation design parameters and considerations should be provided and shall include as a minimum the following items: allowable bearing pressure(s); bearing elevations for each recommended foundation system; a minimum depth the foundation system shall bear below outside finished grade; foundation spacing requirements; foundation structural design methodology to be used; the design loads used to size the foundation elements; a modulus of subgrade reaction; soil unit weights; anticipated settlement/differential settlement; and applicability (of each of the aforementioned items) to the design.

7.2 Subgrade Preparation

This section shall include a discussion on all requirements for excavation of existing subgrade materials, removal of existing unsuitable materials, replacement of excavated materials with non-expansive and satisfactory

materials, and minimum thickness of non-expansive fill beneath foundations. Provide compaction requirements (in accordance with ASTM D 1557) for the raw subgrade, fill, and backfill materials. Foundation material definitions shall be presented.

7.3 Exhibits to be Included in the D-B Contractor's Foundation Design Analysis

The following exhibits shall be included in the Design-Build contractor's Foundation Design Analysis. The Design-Build contractor shall perform a site-specific subsurface exploration and laboratory testing program for his/her design. Required exhibits to be included with the Design-Build contractor's Foundation Design Analysis include:

- Site Plan with Boring Locations and Legend;
- Boring Logs;
- Plasticity Chart;
- Standard Penetration Tests versus Depth of Boring (if applicable);
- Moisture Content versus Depth (Chart);
- Moisture Content-Liquid Limit-Plastic Limit versus Depth (Chart);
- Strength Tests Results versus Depth (Chart);
- Tabulation of Laboratory Test Results (to include Boring Number, Sample Number, Depth, Laboratory Classification, Visual Descriptions, Grain Size Analysis (%Gravel, %Sand, %Fines), LL, PL, PI, MC, Unit Weight, and Strength Test Data.

8. References

- UFC 3-220-01 Geotechnical Engineering.
- UFGS Guide Specifications For Construction
- Final Submittal Geotechnical Exploration, Fort Isabel Detention Center, Kitchen and Wastewater Treatment Plant Improvements, Los Fresnos, Cameron County, Texas, prepared by Geotechnology, Inc., March 13, 2019

FORT WORTH DISTRICT
JULY 2023

APPENDIX I:
SITE LAYOUT

Locations of the planned ICE Secure Housing Unit, and Kitchen and WWTP Improvements Project Drilled Previously

APPENDIX II:
BORING LOCATION PLAN, BORING LOGS, & LABORATORY TESTING DATA

KITCHEN AND WWTP IMPROVEMENT SITES

FINAL SUBMITTAL
GEOTECHNICAL EXPLORATION
PORT ISABEL DETENTION CENTER
KITCHEN AND WASTEWATER TREATMENT PLANT IMPROVEMENTS
U.S. IMMIGRATION AND CUSTOMS ENFORCEMENT (ICE)
LOS FRESNOS, CAMERON COUNTY, TEXAS

IDIQ CONTRACT NO. W9126G15D007 / W912P419F0002
ETEGRA PROJECT NO. 6006

Prepared for:

ETEGRA, INC.
OLIVETTE, MISSOURI

Prepared by:

GEOTECHNOLOGY, INC.
ERLANGER, KENTUCKY

Date:

MARCH 13, 2019

Geotechnology Project No.:

J033011.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

APPENDIX B – EXPLORATION PLANS

Site Location Map

Boring and Site Plans

Date Printed: 2/14/2019 1:34 PM Path: Drawing4.dwg

NOTE: BASE MAP FROM GOOGLE EARTH

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: Port Isabel Detention Center
Kitchen and WWTP Improvements

Location: Los Fresnos, Texas

Title: SITE LOCATION MAP

Client: Etegra, Inc.

Date: 2/14/2019

Project No.: J033011.01

Sheet No.: 1

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NOTE: BASE MAP FROM "TOPOGRAPHIC SURVEY, PORT ISABEL DETENTION CENTER, SHEET 1 OF 3" PREPARED BY OLSSON AND DATED FEB 1, 2019

EXISTING LIFT STATION

W-4
El. 17.1

PROPOSED LIFT STATION

K-1
El. 17.4

K-2
El. 17.8

K-3
El. 17.9

K-4
El. 18.6

K-5
El. 17.9

PROPOSED
KITCHEN
FF=EL. 22.29

ABANDONED BUILDINGS
TO BE DEMOLISHED

EXISTING
KITCHEN
FF=EL. 22.29

K-6
El. 18.7

K-7
El. 17.7

0 20 40 80
SCALE: 1" = 40'

INDICATES TEST BORING LOCATIONS

Date: 2/20/2019 Project No.: J033011.01 Sheet No.: 2	Title: BORING AND SITE PLAN KITCHEN AND LIFT STATION AREAS Client: Etegra, Inc.	Project: PIDC Kitchen and WWTP Improvements Location: Los Fresnos, Texas

GEOTECHNOLOGY INC
FROM THE GROUND UP

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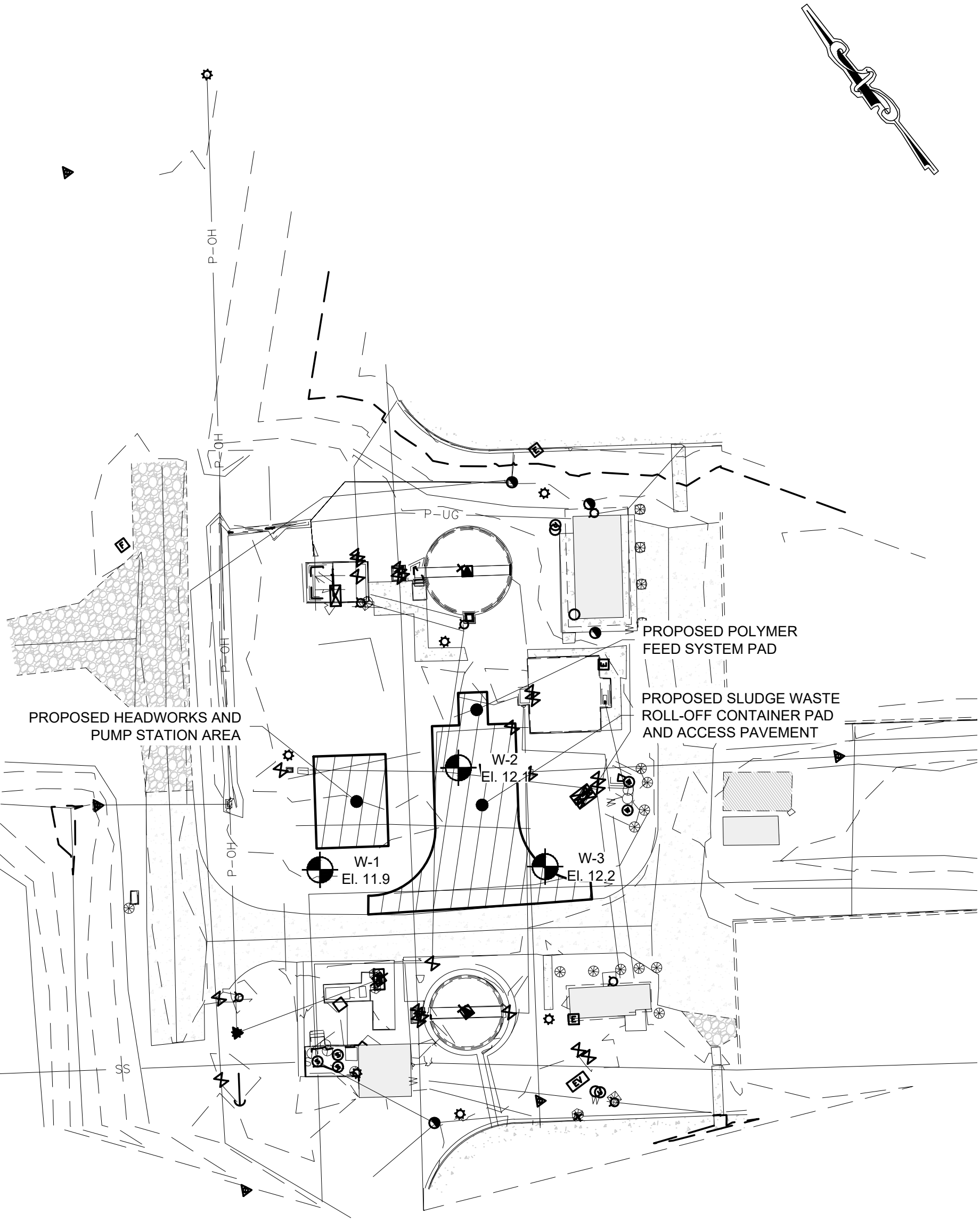
Date Printed: 2/21/2019 1:11 PM Path: \\10.0.10.12\network storage\data\projects\J033\J033011.01-Port Isabel Texas Facility\Draw\J033011.01-Boring Plan.dwg

Date Printed: 2/20/2019 1:40 PM Path: \\10.0.10.12\network storage\data\projects\J033\J033011.01-Port Isabel Texas Facility\Draw\J033011.01-Boring Plan.dwg



NOTE: BASE MAP BY OLSSON AND PROVIDED ON FEB 12, 2019

0 15 30 60
SCALE: 1" = 30'

INDICATES TEST BORING LOCATIONS

Date: 2/20/2019 Project No.: J033011.01 Sheet No.: 3	Title: BORING AND SITE PLAN WWTP AREA Client: Etegra, Inc.	Project: PIDC Kitchen and WWTP Improvements Location: Los Fresnos, Texas	GEOTECHNOLOGY INC FROM THE GROUND UP

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APPENDIX C – EXPLORATION INFORMATION

Boring Logs

Soil Classification Sheet

DRILLING LOG

DIVISION	CELRB
INSTALLATION	Port Isabel Detention Center
SHEET	1 OF 2 SHEETS
1. PROJECT	PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station)	Kitchen N 16,584,602.1 E 1,365,192.8
3. DRILLING AGENCY	ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*	K-1
5. NAME OF DRILLER	Rich Herman
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	26.5
8. DEPTH DRILLED INTO ROCK	0.0
9. TOTAL DEPTH OF HOLE	26.5
10. SIZE AND TYPE OF BIT	3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*	NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL	CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED 10 / UNDISTURBED 0
14. TOTAL NUMBER CORE BOXES	0
15. ELEVATION GROUND WATER	8.3
16. DATE HOLE	STARTED 1/10/2019 / COMPLETED 1/10/2019
17. ELEVATION TOP OF HOLE	+17.4
18. TOTAL CORE RECOVERY FOR BORING	N/A %
19. GEOLOGIST	W. Losekamp

ELEVATION (a)	DEPTH (b)	LEGEND (c)	CLASSIFICATION OF MATERIALS (Description) (d)	% CORE RECOVERY (e)	BOX OR SAMPLE NO. (f)	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* (g)
+17.4	0.0		Dark brown moist medium stiff to stiff LEAN CLAY, some organics (possible old plow zone).	67	SS-1 0.0 1.5	3-3-2
+15.4	2.0		Brown moist medium stiff sandy LEAN CLAY.	56	SS-2 2.0 3.5	2-1-3
				50	SS-3 5.0 6.5	2-2-4
+10.4	7.0		Brown wet very loose silty fine SAND.	0	SS-4 7.5 9.0	1-1-1 Groundwater at 9.0 feet during drilling. Groundwater at 9.1 feet 5 hours after completion.
+7.9	9.5		Brown wet loose SILT.	0	SS-5 10.0 11.5	3-5-5
+5.4	12.0		Brown wet very loose silty fine SAND.	100	SS-6 12.5 14.0	3-1-3

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 17.4

Hole No. K-1

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+2.9	14.5		Brown wet very loose silty fine SAND. *(continued)*			
			Brown wet medium dense silty fine SAND.	89	SS-7 15.0 16.5	5-6-6
+0.4	17.0					
			Brown wet very loose clayey fine SAND.	100	SS-8 17.5 19.0	3-2-1
-2.1	19.5					
			Brown very moist stiff FAT CLAY.	100	SS-9 20.0 21.5	2-2-4
				100	SS-10 25.0 26.5	3-3-3 Groundwater at 20.4 feet at completion. Boring backfilled with auger cuttings.
-9.1	26.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): Kitchen N 16,584,479.9 E 1,365,368.3
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: K-2
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 26.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 26.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 10; UNDISTURBED 0
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER: 7.2
16. DATE HOLE: STARTED 1/10/2019; COMPLETED 1/10/2019
17. ELEVATION TOP OF HOLE: +17.8
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
+17.8	0.0		Dark brown moist stiff LEAN CLAY, some organics (possible old plow zone) (CL).	50	SS-1 0.0 1.5	3-5-7
+15.8	2.0		Brown moist stiff LEAN CLAY (CL)			
				67	SS-2 2.5 4.0	4-6-7
+13.3	4.5		Brown, trace gray moist very stiff FAT CLAY, trace sand (CH).			
				83	SS-3 5.0 6.5	5-6-9
				78	SS-4 7.5 9.0	1-1-2
+8.3	9.5		Brown, trace gray moist stiff FAT CLAY (CH).			
				100	SS-5 10.0 11.5	2-2-2 Groundwater at 10.6 feet, 1.75 hours after completion.
				100	SS-6 12.5 14.0	1-3-3

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 17.8

Hole No. K-2

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
+3.3	14.5		Brown, trace gray moist stiff FAT CLAY (CH). *(continued)*			
			Brown wet soft clayey SILT.	100	SS-7 15.0 16.5	1-1-4
-0.7	18.5			100	SS-8A SS-8B 17.5 19.0	1-2-3
			Brown and gray moist stiff FAT CLAY.			
				100	SS-9 20.0 21.5	3-4-5 Groundwater at 20.3 feet during drilling.
				100	SS-10 25.0 26.5	2-1-3 Groundwater at 24.8 feet at completion. Boring backfilled with auger cuttings.
-8.7	26.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): Kitchen N 16,584,546.7 E 1,365,154.7
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: K-3
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 26.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 26.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 9; UNDISTURBED 1
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 1/10/2019; COMPLETED 1/10/2019
17. ELEVATION TOP OF HOLE: +17.9
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+17.9	0.0		Dark brown moist stiff LEAN CLAY, trace roots and organics (possible old plow zone).	61	SS-1 0.0 1.5	2-2-3
+15.9	2.0		Brown moist medium stiff LEAN CLAY.			
				56	SS-2 2.5 4.0	1-1-2
+13.4	4.5		Brown moist loose sandy SILT.			
				21	PT-3 5.0 7.0	
+10.4	7.5		Brown wet very loose clayey SILT, trace carbonate.	100	SS-4 7.5 9.0	WOH/12"-3 Groundwater at 8.5 feet during drilling.
+8.4	9.5		Brown wet loose SILT.			
				78	SS-5 10.0 11.5	2-3-3
+5.9	12.0		Brown moist stiff FAT CLAY, trace silt and lean clay layers.			
				78	SS-6 12.5 14.0	1-1-1

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 17.9

Hole No. K-3

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Brown moist stiff FAT CLAY, trace silt and lean clay layers. *(continued)*			
				83	SS-7 15.0 16.5	1-1-4
+0.9	17.0					
			Brown wet very soft clayey SILT, trace laminations.			
				100	SS-8 17.5 19.0	WOH-1-1
-1.6	19.5					
			Gray moist stiff FAT CLAY.			
				100	SS-9 20.0 21.5	2-2-3 Groundwater at 20.0 feet at completion.
				100	SS-10 25.0 26.5	3-4-4 Boring dry 3 hours 40 minutes after completion. Boring backfilled with auger cuttings. Bottom of test boring at 26.5 feet.
-8.6	26.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): Kitchen N 16,584,456.1 E 1,365,203.3
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: K-4
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 31.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 31.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 10; UNDISTURBED 1
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER: 8.1
16. DATE HOLE: STARTED 1/10/2019; COMPLETED 1/10/2019
17. ELEVATION TOP OF HOLE: +18.6
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+18.6	0.0		Dark brown slightly moist to moist very stiff LEAN CLAY (possible old plow zone) (CL).	50	SS-1 0.0 1.5	4-5-4 Bulk sample collected from 0.0'-5.0'
+16.6	2.0		Brown moist very stiff LEAN CLAY with carbonate deposits (CL).			
				67	PT-2 2.5 4.5	
				78	SS-3 5.0 6.5	4-5-5
+11.6	7.0		Brown moist stiff FAT CLAY (CH).			
				0	SS-4 7.5 9.0	1-1-1
				100	SS-5 10.0 11.5	2-2-2 Groundwater at 10.5 feet, 5 hours 10 minutes after completion.
+6.6	12.0		Brown moist stiff to very stiff FAT CLAY (CH).			
				100	SS-6 12.5 14.0	2-3-4

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 18.6

Hole No. K-4

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Brown moist stiff to very stiff FAT CLAY (CH). *(continued)*			
				100	SS-7 15.0 16.5	2-3-4
+1.6	17.0					
			Brown wet very soft clayey SILT (ML).			
				100	SS-8 17.5 19.0	WOH-2-3
-0.9	19.5					
			Gray very moist medium stiff to stiff FAT CLAY (CH).			
				100	SS-9 20.0 21.5	2-2-3 Groundwater at 21.0 feet during drilling.
-4.9	23.5					
			Brownish gray moist stiff FAT CLAY (CH).			
				100	SS-10 25.0 26.5	1-2-2 Groundwater at 26.8 feet at completion.
-9.9	28.5					
			Brown very moist medium stiff to stiff FAT CLAY with some lean clay and clayey silt layers.			
				100	SS-11 30.0 31.5	1-1-2 Boring backfilled with auger cuttings. Bottom of test boring at 31.5 feet.
-12.9	31.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): Kitchen N 16,584,375.7 E 1,365,310.0
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: K-5
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 31.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 31.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 10 UNDISTURBED 1
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 1/10/2019 COMPLETED 1/10/2019
17. ELEVATION TOP OF HOLE: +17.9
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+17.9	0.0		Dark brown moist very stiff LEAN CLAY with organics, trace roots (possible old plow zone) (CL).	100	SS-1 0.0 1.5	3-4-5
+15.9	2.0		Brown slightly moist very stiff LEAN CLAY with carbonate deposits (CL).	50	SS-2 2.5 4.0	5-10-10
+13.4	4.5		Light brown to brown moist very stiff FAT CLAY (CH).	33	PT-3 5.0 7.0	
+10.7	7.2		Brown moist stiff FAT CLAY (CH).	100	SS-4 7.5 9.0	1-1-2
+8.4	9.5		Brown, trace gray moist medium stiff to stiff FAT CLAY (CH).	100	SS-5 10.0 11.5	2-1-4
+5.9	12.0		Brown and gray moist stiff FAT CLAY (CH).	100	SS-6 12.5 14.0	2-2-2

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 17.9

Hole No. K-5

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
+3.4	14.5		Brown and gray moist stiff FAT CLAY (CH). *(continued)*			
			Brown wet very soft clayey SILT (ML).	89	SS-7 15.0 16.5	WOH-1-1
				100	SS-8 17.5 19.0	WOH-1-1 Groundwater at 18.7 feet during drilling.
-1.6	19.5		Gray moist stiff FAT CLAY (CH).	100	SS-9 20.0 21.5	1-2-3
				100	SS-10 25.0 26.5	1-2-5 Groundwater at 29.5 feet at completion.
-10.6	28.5		Gray very moist soft to medium stiff FAT CLAY with clayey silt layers.	100	SS-11 30.0 31.5	1-1-1 Boring dry 10 minutes after completion. Boring backfilled with auger cuttings. Bottom of boring at 31.5 feet.
-13.6	31.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): Kitchen N 16,584,340.7 E 1,365,127.5
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: K-6
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 31.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 31.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 9; UNDISTURBED 2
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER: 6.3
16. DATE HOLE: STARTED 1/11/2019; COMPLETED 1/11/2019
17. ELEVATION TOP OF HOLE: +18.7
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+18.7	0.0		Dark brown moist very stiff LEAN CLAY, trace roots (possible old plow zone) (CL).	56	SS-1 0.0 1.5	3-5-3
+16.7	2.0		Brown moist medium stiff to stiff LEAN CLAY (CL).	50	SS-2 2.5 4.0	1-1-3
+12.7	6.0		Brown moist stiff FAT CLAY (CH).	100	SS-3A SS-3B 5.0 6.5	3-4-5
+9.2	9.5		Brown very moist medium stiff LEAN CLAY (CL).	89	SS-4 7.5 9.0	1-1-1
				33	PT-5 10.0 12.0	
				83	PT-6 12.5 14.5	Groundwater at 12.4 feet, 3.5 hours after completion. Groundwater at 13.8 feet during drilling.

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE 18.7

Hole No. K-6

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Brown very moist medium stiff LEAN CLAY (CL). *(continued)*			
				100	SS-7 15.0 16.5	2-2-2
+1.7	17.0					
			Brown wet very soft clayey SILT (ML).			
				100	SS-8 17.5 19.0	WOH/18"
-0.8	19.5					
			Brown very moist stiff FAT CLAY (CH).			
				100	SS-9 20.0 21.5	WOH-2-3
				100	SS-10 25.0 26.5	2-1-3 Groundwater at 28.1 feet at completion.
-8.8	27.5					
			Gray, trace brown moist stiff FAT CLAY.			
				61	SS-11 30.0 31.5	2-2-4 Boring backfilled with auger cuttings. Bottom of boring at 31.5 feet.
-12.8	31.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): Kitchen N 16,584,273.3 E 1,365,213.4
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: K-7
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 31.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 31.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 11; UNDISTURBED 0
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 1/11/2019; COMPLETED 1/11/2019
17. ELEVATION TOP OF HOLE: +17.7
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+17.7	0.0		Dark brown moist very stiff LEAN CLAY, trace roots (topsoil) (possible old plow zone) (CL).	50	SS-1 0.0 1.5	2-2-5
+15.7	2.0		Brown moist very stiff LEAN CLAY with carbonate deposits (CL).	56	SS-2 2.5 4.0	2-3-7
+13.2	4.5		Brown moist very stiff FAT CLAY with carbonate deposits (CH).	72	SS-3 5.0 6.5	3-4-4
+10.7	7.0		Brown very moist medium stiff LEAN CLAY (CL).	100	SS-4 7.5 9.0	1-1-2
+8.2	9.5		Brown moist stiff FAT CLAY (CH).	100	SS-5 10.0 11.5	1-2-4
				100	SS-6 12.5 14.0	2-1-2

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 17.7

Hole No. K-7

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+3.2	14.5		Brown moist stiff FAT CLAY (CH). *(continued)*			
			Brown wet very soft clayey SILT (ML).	100	SS-7 15.0 16.5	1-1-1
-0.3	18.0			100	SS-8 17.5 19.0	WOH/12"-2
			Gray and brown very moist stiff FAT CLAY (CH).			
				100	SS-9 20.0 21.5	2-2-3
-4.8	22.5					
			Gray and brown wet medium stiff LEAN CLAY with sand (CL).	100	SS-10 25.0 26.5	1-1-2 Groundwater at 25.8 feet at completion.
-10.3	28.0					
			Brown and gray wet medium stiff LEAN CLAY with a little fine sand (CL).	100	SS-11 30.0 31.5	WOH-1-1 Boring dry 5 minutes after completion. Boring backfilled with augur cuttings. Bottom of boring at 31.5 feet.
-13.8	31.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): WWTP N 16,583,012.1 E 1,366,767.8
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: W-1
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 31.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 31.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 10; UNDISTURBED 1
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER: 6.5
16. DATE HOLE: STARTED 1/9/2019; COMPLETED 1/9/2019
17. ELEVATION TOP OF HOLE: +11.9
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+11.8	0.1		TOPSOIL (1 inch)	100	SS-1 0.0 1.5	3-3-4 Bulk sample collected from 0.0'-2.5'
			Dark brown moist stiff FAT CLAY, trace roots (possible old plow zone).			
+9.9	2.0		Gray to dark gray moist stiff FAT CLAY.			
				44	SS-2 2.5 4.0	1-1-2
				11	SS-3 5.0 6.5	WOH/12"-2 Groundwater at 5.4 feet, 21 hours 15 minutes after completion.
+4.9	7.0		Gray, trace brown moist soft FAT CLAY.			
				100	SS-4A SS-4B 7.5 9.0	4-3-2
+3.4	8.5		Brown very moist soft LEAN CLAY.			
+2.4	9.5		Brown moist stiff to very stiff FAT CLAY.			
				100	SS-5 10.0 11.5	2-2-2
				100	SS-6 12.5 14.0	2-7-8

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE 11.9

Hole No. W-1

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g
-2.6	14.5		Brown moist stiff to very stiff FAT CLAY. *(continued)*			
			Brown moist stiff FAT CLAY (CH).	13	PT-7 15.0 17.0	
				100	SS-8 17.5 19.0	1-2-3
-7.6	19.5					
			Brownish gray wet soft clayey SILT to lean clay.	100	SS-9 20.0 21.5	2-4-3 Groundwater at 20.5 feet at completion.
-11.6	23.5					
			Brown very moist medium stiff FAT CLAY.	83	SS-10A SS-10B 25.0 26.5	2-2-1
-14.1	26.0					
			Brown and gray wet very loose sandy SILT (ML).	100	SS-11A SS-11B 30.0 31.5	WOH/12"-1 Boring backfilled with auger cuttings and bentonite chips. Bottom of boring at 31.5 feet.
-18.6	30.5					
			Brownish gray very moist medium stiff to stiff LEAN CLAY.			
-19.6	31.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT	PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station)	WWTP N 16,583,011.9 E 1,366,818.2
3. DRILLING AGENCY	ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*	W-2
5. NAME OF DRILLER	Rich Herman
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	21.5
8. DEPTH DRILLED INTO ROCK	0.0
9. TOTAL DEPTH OF HOLE	21.5
10. SIZE AND TYPE OF BIT	3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*	NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL	CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED 8 / UNDISTURBED 1
14. TOTAL NUMBER CORE BOXES	0
15. ELEVATION GROUND WATER	6.9
16. DATE HOLE	STARTED 1/8/2019 / COMPLETED 1/8/2019
17. ELEVATION TOP OF HOLE	+12.1
18. TOTAL CORE RECOVERY FOR BORING	N/A %
19. GEOLOGIST	W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+12.0	0.1		TOPSOIL (1 inch)	100	SS-1 0.0 1.5	4-4-4
			Dark gray moist stiff FAT CLAY, trace gravel, trace roots (possible old plow zone).			
+10.1	2.0		Gray moist stiff FAT CLAY.			
				78	SS-2 2.5 4.0	1-1-2
				100	SS-3 5.0 6.5	WOH-1-1 Groundwater at 5.2 feet, 16.5 hours after completion.
+5.1	7.0		Brown and gray moist medium stiff FAT CLAY, trace gravel (CH).			
				46	PT-4 7.5 9.5	
+2.4	9.7		Brown wet very soft clayey SILT (ML).			
				56	SS-5 10.0 11.5	WOH-1-1
+0.1	12.0		Brownish gray moist stiff FAT CLAY, trace sand.			
				100	SS-6 12.5 14.0	WOH-2-2 Groundwater at 12.5 feet during drilling.

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 12.1

Hole No. W-2

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
			Brownish gray moist stiff FAT CLAY, trace sand. *(continued)*			
				100	SS-7 15.0 16.5	1-2-3 Groundwater 15.6 feet at completion.
				100	SS-8A SS-8B 17.5 19.0	3-1-1
-6.4	18.5					
			Brown wet very soft clayey SILT.			
-7.4	19.5					
			Gray very moist medium stiff LEAN CLAY with sand (CL).			
				100	SS-9 20.0 21.5	2-2-2 Boring backfilled with bentonite chips and cuttings. Bottom of boring at 21.5 feet.
-9.4	21.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): WWTP N 16,582,973.3 E 1,366,821.2
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: W-3
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 21.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 21.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 9; UNDISTURBED 0
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER: 2.0
16. DATE HOLE: STARTED 1/8/2019; COMPLETED 1/8/2019
17. ELEVATION TOP OF HOLE: +12.2
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+12.1	0.1		TOPSOIL (1 inch)	100	SS-1 0.0 1.5	2-4-6
			Gray, trace brown moist stiff FAT CLAY, trace roots (possible old plow zone).			
+10.2	2.0		Gray moist stiff FAT CLAY.			
				50	SS-2 2.5 4.0	1-1-2
				100	SS-3 5.0 6.5	1-2-2
				100	SS-4 7.5 9.0	1-3-3
+2.2	10.0		Gray to dark gray very moist medium stiff FAT CLAY.	100	SS-5 10.0 11.5	1-2-1
+0.2	12.0		Gray wet stiff to very stiff FAT CLAY.			
				100	SS-6 12.5 14.0	2-3-4 Groundwater at 12.5 feet during drilling. Groundwater at 14.2 feet at completion and at 10 minutes after completion.

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 12.2

Hole No. W-3

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
-2.3	14.5		Gray wet stiff to very stiff FAT CLAY. *(continued)*			
			Gray moist stiff to very stiff FAT CLAY, trace sand.	100	SS-7 15.0 16.5	3-4-5
-4.8	17.0					
			Gray very moist medium stiff FAT CLAY.	100	SS-8 17.5 19.0	2-2-2
-7.3	19.5					
			Gray, trace brown wet medium stiff sandy LEAN CLAY (CL).	100	SS-9 20.0 21.5	2-2-4 Boring backfilled with bentonite chips and cuttings. Bottom of boring at 21.5 feet.
-9.3	21.5					

DRILLING LOG

DIVISION: CELRB

INSTALLATION: Port Isabel Detention Center

SHEET 1 OF 2 SHEETS

1. PROJECT: PIDC Kitchen/WWTP Improvements
2. LOCATION (Coordinates or Station): Lift Station N 16,584,689.8 E 1,365,431.1
3. DRILLING AGENCY: ETTL Engineers & Consultants, Inc.
4. HOLE NO. *(As shown on drawing title and file number)*: W-4
5. NAME OF DRILLER: Rich Herman
6. DIRECTION OF HOLE: ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN: 31.5
8. DEPTH DRILLED INTO ROCK: 0.0
9. TOTAL DEPTH OF HOLE: 31.5
10. SIZE AND TYPE OF BIT: 3 1/4" ID HSA
11. DATUM FOR ELEVATION SHOWN *(TBM or MSL)*: NAVD 88
12. MANUFACTURER'S DESIGNATION OF DRILL: CME 55
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN: DISTURBED 10; UNDISTURBED 1
14. TOTAL NUMBER CORE BOXES: 0
15. ELEVATION GROUND WATER:
16. DATE HOLE: STARTED 1/9/2019; COMPLETED 1/9/2019
17. ELEVATION TOP OF HOLE: +17.1
18. TOTAL CORE RECOVERY FOR BORING: N/A %
19. GEOLOGIST: W. Losekamp

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+17.1	0.0		Brown moist stiff LEAN CLAY, trace roots (possible old plow zone) (CL).	50	SS-1 0.0 1.5	3-4-4 Bulk sample collected from 0.0'-5.0'
+15.1	2.0		Light brown moist very stiff LEAN CLAY, trace sand (CL).			
				100	SS-2 2.5 4.0	5-5-4
				75	PT-3 5.0 7.0	
+9.8	7.3		Brown moist stiff FAT CLAY, trace sand (CH).			
				100	SS-4 7.5 9.0	2-3-6
				100	SS-5 10.0 11.5	1-4-5
+5.1	12.0		Brown, trace black moist stiff to very stiff FAT CLAY (CH).			
				100	SS-6 12.5 14.0	2-3-5

DRILLING LOG (Cont Sheet)

ELEVATION TOP OF HOLE: 17.1

Hole No. W-4

PROJECT: PIDC Kitchen/WWTP Improvements

INSTALLATION: Port Isabel Detention Center

SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOV-ERY e	BOX OR SAMPLE NO. f	REMARKS *(Drilling time, water loss, depth weathering, etc., if significant)* g
+2.6	14.5					
			Brown trace gray moist medium stiff FAT CLAY (CH).	44	SS-7 15.0 16.5	1-2-4 Groundwater at 15.2 feet during drilling.
+0.1	17.0					
			Brown wet very soft clayey SILT (ML).	100	SS-8A SS-8B 17.5 19.0	2-1-4
-1.4	18.5					
			Brown and gray moist stiff FAT CLAY.			
-2.4	19.5					
			Gray moist very stiff FAT CLAY, trace sand.	100	SS-9 20.0 21.5	5-7-8 Groundwater at 20.6 feet at completion.
				100	SS-10 25.0 26.5	2-3-4
-11.4	28.5					
			Gray and brown moist medium stiff to stiff FAT CLAY.	100	SS-11 30.0 31.5	1-2-2 Boring dry at 17.8 hours after completion. Boring backfilled with cuttings and bentonite chips. Bottom of boring at 31.5 feet.
-14.4	31.5					

GEOTECHNOLOGY INC
FROM THE GROUND UP

SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density

Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties

Descriptive Term	Percent
Trace	1 – 10
Little	11 – 20
Some	21 – 35
And	36 – 50

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

APPENDIX D – LABORATORY TEST RESULTS

Tabulations of Laboratory Tests

Laboratory Test Forms

DEPARTMENT OF THE ARMY
ENGINEER RESEARCH AND DEVELOPMENT CENTER, CORPS OF ENGINEERS
GEOTECHNICAL AND STRUCTURES LABORATORY
WATERWAYS EXPERIMENT STATION, 3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

August 15, 2017

Reply to the Attention of:
Concrete and Materials Branch

Matthew Barker
Geotechnology, Inc.
1398 Cox Avenue
Erlanger, KY 41018

Dear Mr. Barker:

In reference to your check no. 129549, dated July 11, 2017 and audit agreement, dated July 10, 2017, an audit based on your AASHTO Accreditation was performed on documents submitted by your laboratory. We examined the AMRL On-site Assessment Report No. 20833, dated December 16, 2015, the CCRL Inspection Report No. T-100, dated December 21, 2016 and the AASHTO accreditation certificate effective August 14, 2017. Your Quality System meets the requirements of the U.S. Army Corps of Engineers. The material test methods that you are validated to perform for the U.S. Army Corps of Engineers were determined from the inspection reports from AASHTO and are as follows:

Aggregate Tests: ASTM C117, C127, C128, C136, C40, C88, C123, C131, C142, C535, C566, C702, C1077, C1260, and E329.

Concrete Tests: ASTM C31, C39, C138, C143, C172, C173, C231, C1064, C42, C78, C157, C192, C293, C511, C617, C803, C1074, C1077, C1231, and E329.

Rock Tests: ASTM D4644, D5731, and D7012.

Soil Tests: ASTM D421, D422, D698, D1140, D1557, D1883, D2166, D2216, D2435, D2487, D2488, D2850, D2974, D3740, D4318, D4546, D4767, D5084, D6938, and E329.

We will add your laboratory to the list of commercial laboratories qualified to conduct material tests for the U.S. Army Corps of Engineers; see the Materials Testing Center homepage at http://www.erdc.usace.army.mil/Media/FactSheets/FactSheetArticleView/tabid/9254/Article/476661/materials-testing-center.aspx. All Corps offices will be notified of this decision and will have the opportunity to use your services. Geotechnology, Inc., Erlanger, KY will remain on our list of laboratories qualified to conduct material tests until **August 14, 2019** two (2) years from the date of the audit. In the event your AASHTO accreditation is suspended in whole or part, Geotechnology, Inc., Erlanger, KY is to notify the Materials Testing Center immediately to perform a re-evaluation of your laboratory's validation. Failure to notify the Materials Testing Center will result in immediate suspension of your U.S. Army Corps of Engineers validation.

Sincerely,

Alfred B. Crawley, PE
Director
Materials Testing Center

Copy Furnished:
Doug Comer, Louisville District

GEOTECHNOLOGY INC
FROM THE GROUND UP

GEOTECHNICAL EXPLORATION
PIDC KITCHEN/WWTP IMPROVEMENTS
LOS FRESNOS, TEXAS
J033011.01

TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth (ft.) From	Depth (ft.) To	Moisture Content (%)	Dry Unit Weight (pcf)	Atterberg Limits (%) LL	Atterberg Limits (%) PL	Atterberg Limits (%) PI	Gradation Analysis (%) Gravel	Gradation Analysis (%) Sand	Gradation Analysis (%) Silt	Gradation Analysis (%) Clay	USCS Classification	Unconfined Compressive Strength (psf)	Resistivity (ohm-cm)	Redox Potential (mV)	pH	Sulfides (mg/kg)	Chlorides (mg/kg)	Sulfates (mg/kg)
K-1	SS-1	0.0	1.5	23.0																
K-1	SS-2	2.5	4.0	17.1																
K-1	SS-4	7.5	9.0	26.6																
K-1	SS-6	12.5	14.0	25.2																
K-1	SS-8	17.5	19.0	33.7																
K-1	SS-9	20.0	21.5	32.6																
K-1	SS-10	25.0	26.5	27.5																
K-2	SS-1	0.0	1.5	14.8																
K-2	SS-2	2.5	4.0	14.5																
K-2	SS-4	7.5	9.0	21.7																
K-2	SS-6	12.5	14.0	30.9																
K-2	SS-7	15.0	16.5	31.6																
K-2	SS-9	20.0	21.5	36.1																
K-2	SS-10	25.0	26.5	28.7																
K-3	SS-1	0.0	1.5	16.3																
K-3	SS-2	2.5	4.0	17.2																
K-3	PT-3	5.0	5.4	16.5					0.0	17.3	82.7									
K-3	SS-4	7.5	9.0	27.1																
K-3	SS-5	10.0	11.5	31.0																
K-3	SS-7	15.0	16.5	36.8																
K-3	SS-8	17.5	19.0	28.9																
K-3	SS-9	20.0	21.5	32.1																
K-4	SS-1	0.0	1.5	17.7		37	23	14					CL							
K-4	BUCKET	0.0	5.0	11.9		38	18	20					CL		300	186	8.08	ND	1500	970
K-4	PT-2A	2.8	3.3	15.6	107.1	36	19	17					CL	4,710						
K-4	PT-2B	3.3	3.5	14.7	112.5															
K-4	SS-3	5.0	6.5	16.5		40	20	20					CL							
K-4	SS-5	10.0	11.5	28.9		52	20	32					CH							
K-4	SS-6	12.5	14.0	27.5		63	22	41					CH							
K-4	SS-7	15.0	16.5	31.9																
K-4	SS-8	17.5	19.0	31.8		29	26	3	0.0	1.8	98.2		ML							
K-4	SS-9	20.0	21.5	33.7		53	18	35					CH							
K-4	SS-10	25.0	26.5	30.2		56	20	36					CH							
K-4	SS-11	30.0	31.5	31.0																
K-5	SS-1	0.0	1.5	15.9		41	19	22					CL							
K-5	SS-2	2.5	4.0	13.5		39	17	22					CL							
K-5	PT-3A	5.1	5.3	10.4	97.8															
K-5	PT-3B	5.3	5.7			53	21	32					CH							
K-5	SS-4	7.5	9.0	28.3		51	20	31					CH							
K-5	SS-5	10.0	11.5	30.8		57	25	32					CH							
K-5	SS-6	12.5	14.0	31.4		69	23	46					CH							

TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth (ft.) From	Depth (ft.) To	Moisture Content (%)	Dry Unit Weight (pcf)	Atterberg Limits (%) LL	PL	PI	Gradation Analysis (%) Gravel	Sand	Silt	Clay	USCS Classification	Unconfined Compressive Strength (psf)	Resistivity (ohm-cm)	Redox Potential (mV)	pH	Sulfides (mg/kg)	Chlorides (mg/kg)	Sulfates (mg/kg)
K-5	SS-7 & 8	15.0	19.0	32.2		30	25	5	0.1	2.5	97.4		ML							
K-5	SS-9	20.0	21.5	34.0		78	26	52					CH							
K-5	SS-10	25.0	26.5	31.2																
K-5	SS-11	30.0	31.5	29.7																
K-6	SS-1	0.0	1.5	17.5		39	20	19					CL							
K-6	SS-2	2.5	4.0	19.3		39	19	20					CL							
K-6	SS-3A	5.0	6.0	20.4		41	19	22					CL							
K-6	SS-4	7.5	9.0	26.1		51	21	30					CH							
K-6	PT-5A	10.0	10.2	30.9	91.6															
K-6	PT-5B	10.2	10.7			42	24	18	0.3	1.2	55.4	43.1	CL							
K-6	PT-6A	12.5	13.0	31.0	92.1									1,420						
K-6	SS-7	15.0	16.5	32.6																
K-6	SS-8	17.5	19.0	30.5		32	25	7	0.5	1.6	97.9		ML							
K-6	SS-9	20.0	21.5	37.7		75	22	53					CH							
K-6	SS-10	25.0	26.5	29.4																
K-6	SS-11	30.0	31.5	27.8																
K-7	SS-1	0.0	1.5	22.3		45	19	26					CL							
K-7	SS-2	2.5	4.0	17.5		43	20	23					CL							
K-7	SS-3	5.0	6.5	23.2		57	20	37					CH							
K-7	SS-4	7.5	9.0	32.0		47	19	28					CL							
K-7	SS-5	10.0	11.5	29.8		51	19	32					CH							
K-7	SS-6	12.5	14.0	32.6		67	26	41					CH							
K-7	SS-7	15.0	16.5	33.2		31	25	6	0.0	5.2	94.8		ML							
K-7	SS-8	17.5	19.0	34.4		50	20	30					CH							
K-7	SS-9	20.0	21.5	32.8																
K-7	SS-10	25.0	26.5	27.0		35	15	20	0.0	28.4	71.6		CL							
K-7	SS-11	30.0	31.5	28.5		35	23	12	0.0	12.8	87.2		CL							
W-1	SS-1	0.0	1.5	25.6																
W-1	BUCKET	0.0	2.5	25.4		62	20	42					CH		1200	196	8.22	ND	110	1900
W-1	SS-2	2.5	4.0	28.6																
W-1	SS-3	5.0	6.5	23.2																
W-1	SS-4A	7.5	8.5	31.0																
W-1	SS-4B	8.5	9.0	25.3																
W-1	SS-5	10.0	11.5	32.1																
W-1	SS-6	12.5	14.0	29.1																
W-1	PT-7	15.0	17.0	31.5		64	22	42					CH							
W-1	SS-8	17.5	19.0	32.1																
W-1	SS-9	20.0	21.5	26.3																
W-1	SS-10A	25.0	26.0	34.4																
W-1	SS-10B & SS-11A	26.0	30.5	32.4		26	24	2	0.0	29.7	70.3		ML							
W-1	SS-11B	30.5	31.5	32.8																

TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth (ft.) From	Depth (ft.) To	Moisture Content (%)	Dry Unit Weight (pcf)	Atterberg Limits (%) LL	Atterberg Limits (%) PL	Atterberg Limits (%) PI	Gradation Analysis (%) Gravel	Gradation Analysis (%) Sand	Gradation Analysis (%) Silt	Gradation Analysis (%) Clay	USCS Classification	Unconfined Compressive Strength (psf)	Resistivity (ohm-cm)	Redox Potential (mV)	pH	Sulfides (mg/kg)	Chlorides (mg/kg)	Sulfates (mg/kg)
W-2	SS-1	0.0	1.5	30.7																
W-2	SS-2	2.5	4.0	30.6																
W-2	SS-3	5.0	6.5	32.8																
W-2	PT-4A	7.5	8.0	32.3	90.9	75	26	49					CH	1,650						
W-2	SS-5	10.0	11.5	31.0		31	23	8	0.0	3.1	96.9		ML							
W-2	SS-6	12.5	14.0	30.0																
W-2	SS-7	15.0	16.5	30.2																
W-2	SS-8A	17.5	18.5	33.0																
W-2	SS-9	20.0	21.5	27.0		37	16	21	0.0	22.1	77.9		CL							
W-3	SS-1	0.0	1.5	24.0																
W-3	SS-2	2.5	4.0	34.9																
W-3	SS-3	5.0	6.5	29.1																
W-3	SS-4	7.5	9.0	33.0																
W-3	SS-5	10.0	11.5	35.8																
W-3	SS-6	12.5	14.0	27.3																
W-3	SS-7	15.0	16.5	30.7																
W-3	SS-8	17.5	19.0	35.2																
W-3	SS-9	20.0	21.5	27.5		29	18	11	0.0	29.9	70.1		CL							
W-4	SS-1	0.0	1.5	17.9																
W-4	BUCKET	0.0	5.0	18.2		43	18	25					CL		280	190	8.09	ND	1600	870
W-4	SS-2	2.5	4.0	14.9																
W-4	PT-3A	5.2	5.7	27.5	93.6									4,230						
W-4	PT-3B	5.7	5.9	26.1	94.1															
W-4	SS-4	7.5	9.0	29.6																
W-4	SS-5	10.0	11.5	28.8		57	24	33					CH							
W-4	SS-6	12.5	14.0	32.3		75	28	47					CH							
W-4	SS-7	15.0	16.5	30.5		57	24	33					CH							
W-4	SS-8A	17.5	18.5	35.0		34	25	9	0.3	0.8	98.9		ML							
W-4	SS-9	20.0	21.5	36.2																
W-4	SS-10	25.0	26.5	28.2																
W-4	SS-11	30.0	31.5	32.5																

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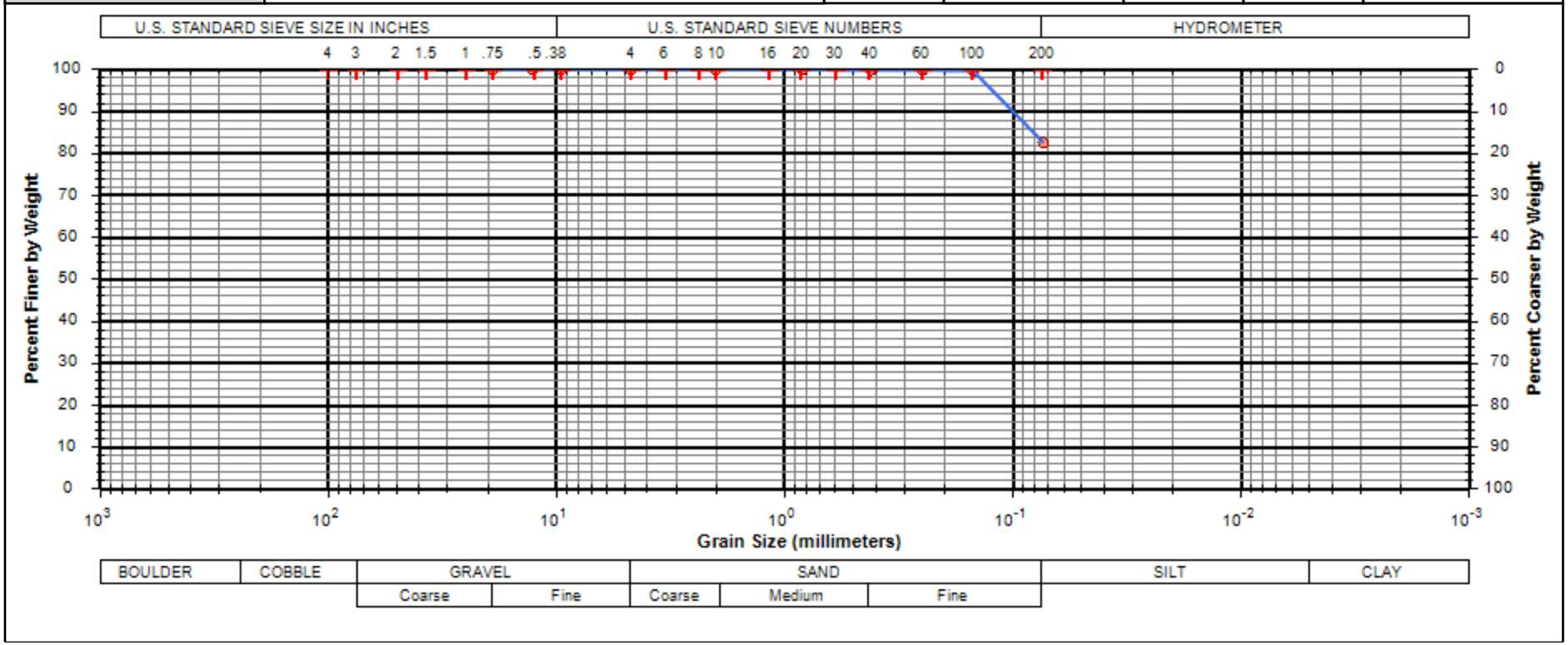
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/01/2019
Boring No.:	K-3	**Sample No.:**	PT-3	**Depth (ft.):**	5.0 - 5.4	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown moist loose sandy SILT					0.0	17.3	82.7		ML
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
										16.5



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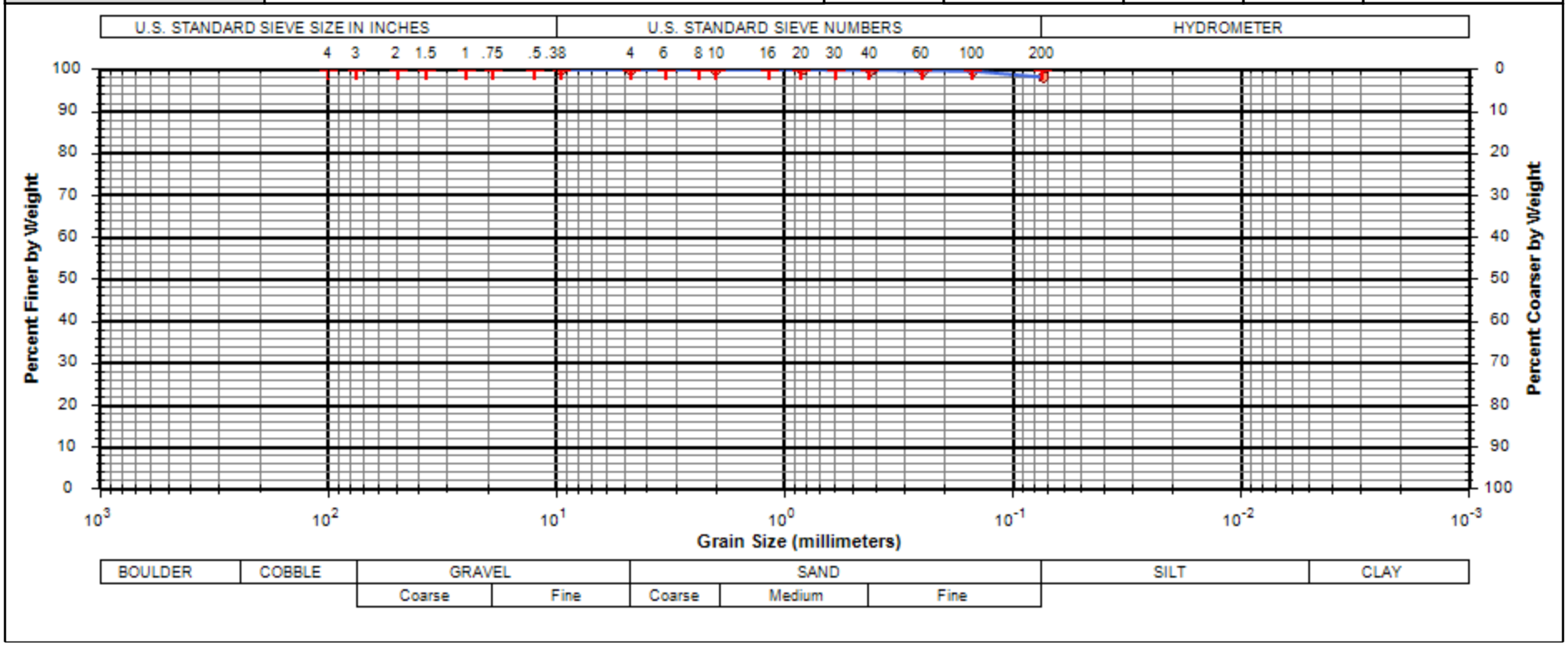
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/01/2019
Boring No.:	K-4	**Sample No.:**	SS-8	**Depth (ft.):**	17.5 - 19.0	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown wet very soft clayey SILT					0.0	1.8	98.2		ML
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						29	26	3		31.8



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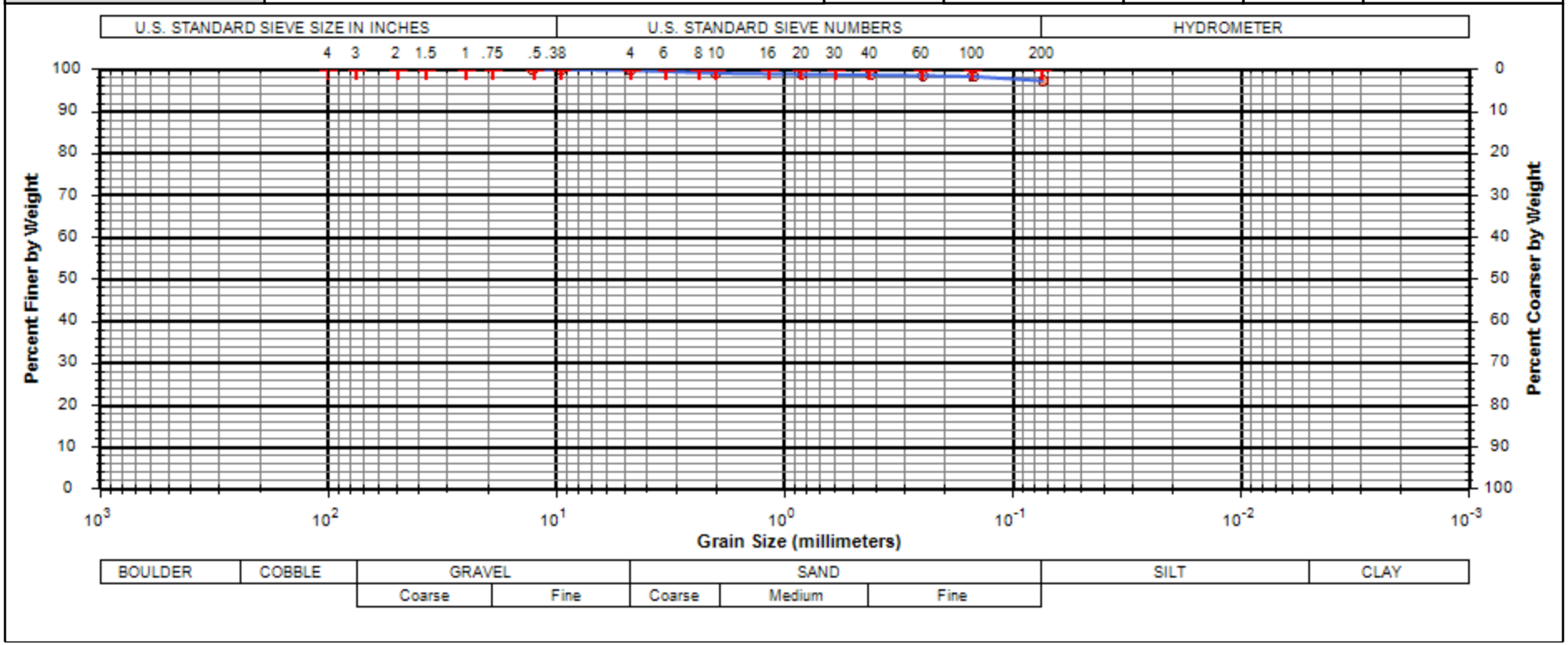
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/01/2019
Boring No.:	K-5	**Sample No.:**	SS-7 & SS-8	**Depth (ft.):**	15.0 - 19.0	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown wet very soft clayey SILT					0.1	2.5	97.4		ML
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						30	25	5		32.2



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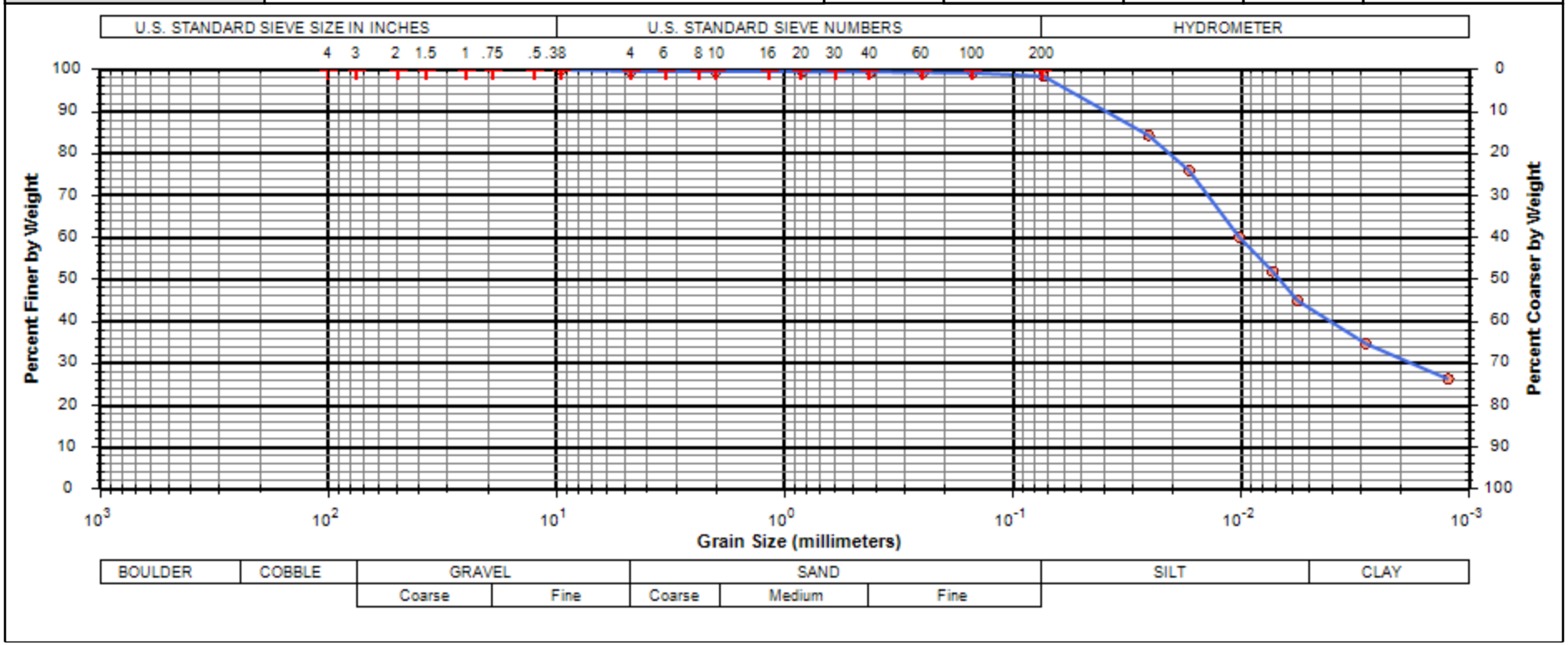
PARTICLE-SIZE ANALYSIS OF SOILS ASTM D-422

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/06/2019
Boring No.:	K-6	**Sample No.:**	PT-5B	**Depth (ft.):**	10.2 - 10.7	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**USCS**
Sample Description:	Brown very moist medium stiff LEAN CLAY					0.3	1.2	55.4	43.1	CL
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						42	24	18		30.2



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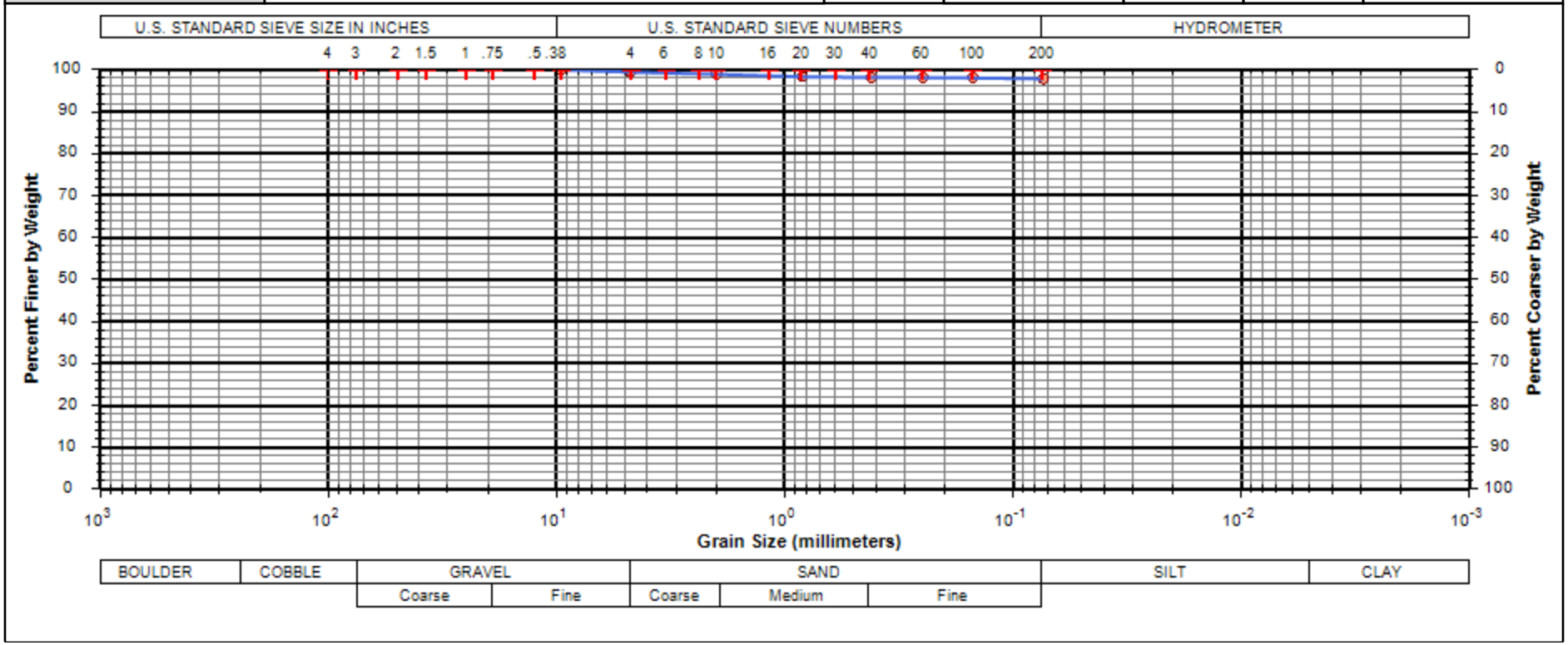
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/05/2019
Boring No.:	K-6	**Sample No.:**	SS-8	**Depth (ft.):**	17.5 - 19.0	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown wet very soft clayey SILT					0.5	1.6	97.9		ML
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						32	25	7		30.5



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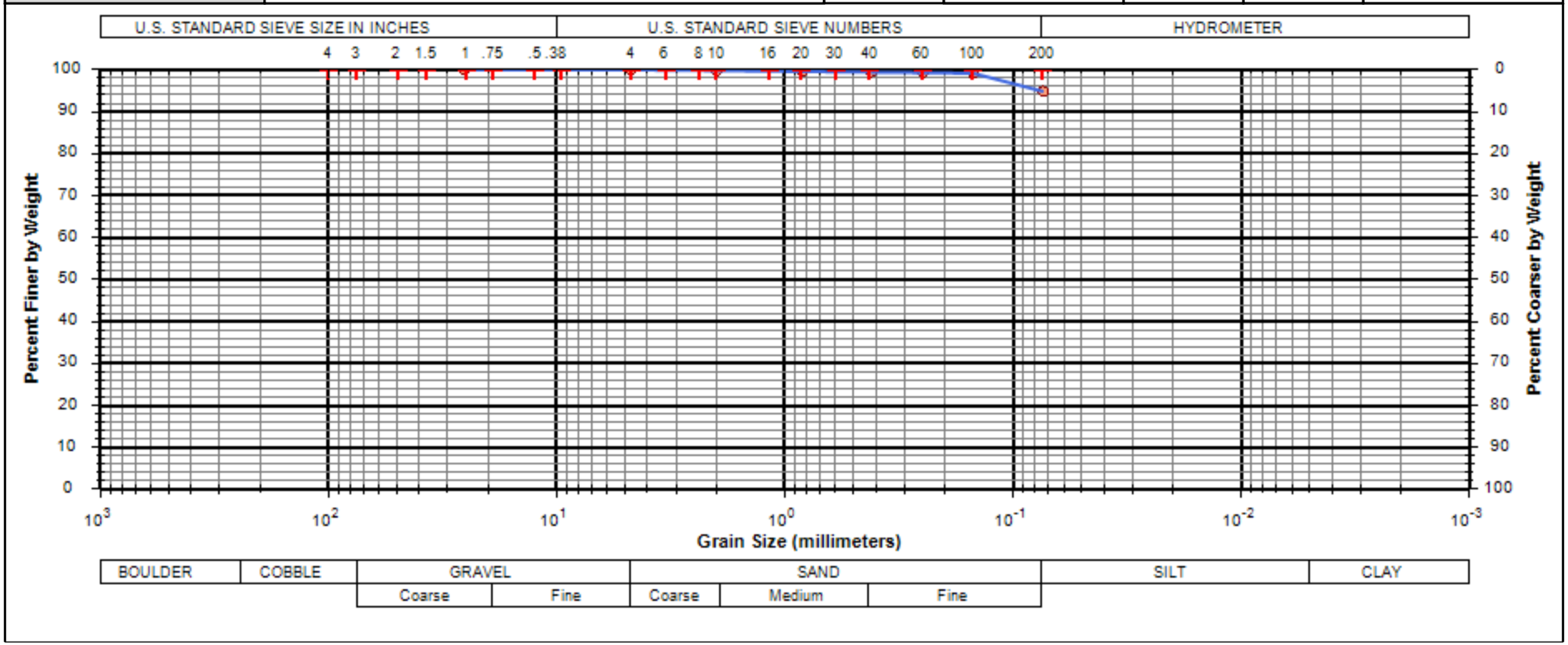
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/04/2019
Boring No.:	K-7	**Sample No.:**	SS-7	**Depth (ft.):**	15.0 - 16.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown wet very soft clayey SILT					0.0	5.2	94.8		ML
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						31	25	6		33.2



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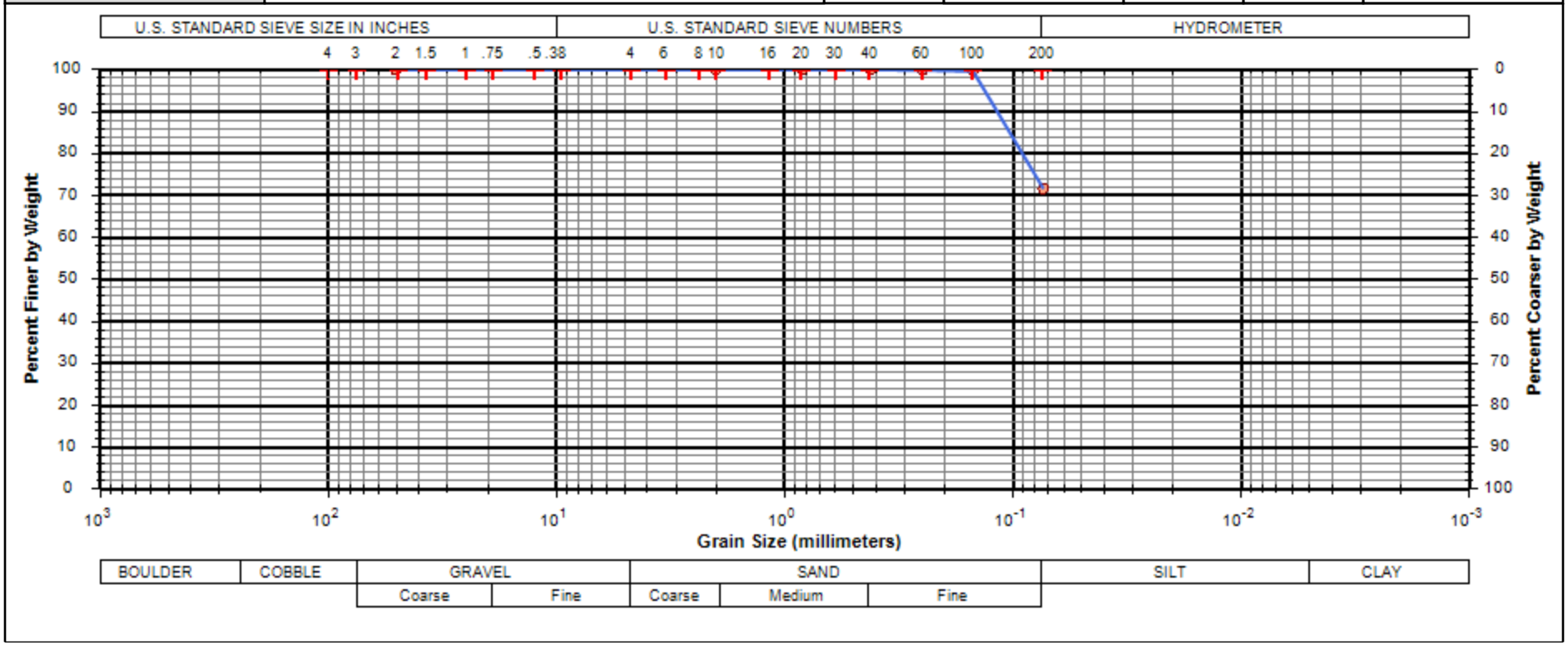
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/05/2019
Boring No.:	K-7	**Sample No.:**	SS-10	**Depth (ft.):**	25.0 - 26.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Gray and brown wet medium stiff LEAN CLAY with sand					0.0	28.4	71.6		CL
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						35	15	20		27.0



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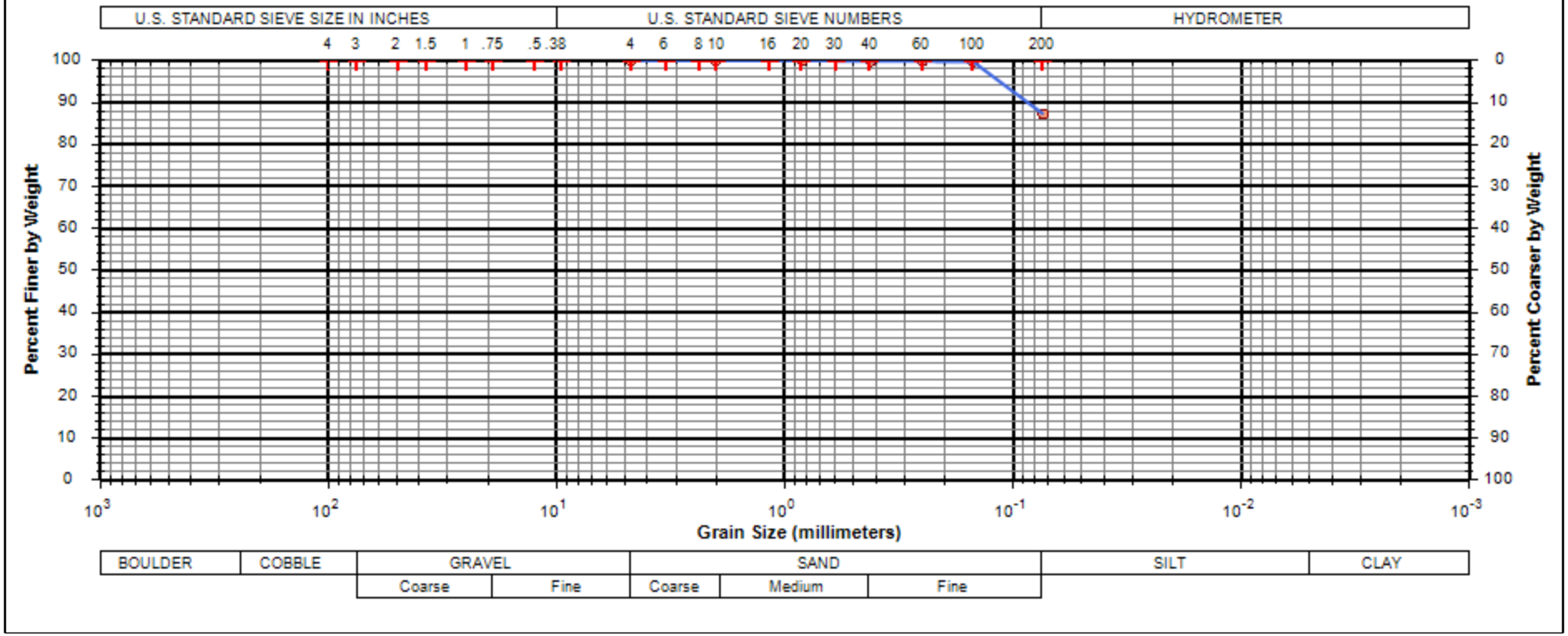
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/04/2019
Boring No.:	K-7	**Sample No.:**	SS-11	**Depth (ft.):**	30.0 - 31.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown and gray wet medium stiff LEAN CLAY with a little fine sand					0.0	12.7	87.3		CL
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						35	23	12		28.5



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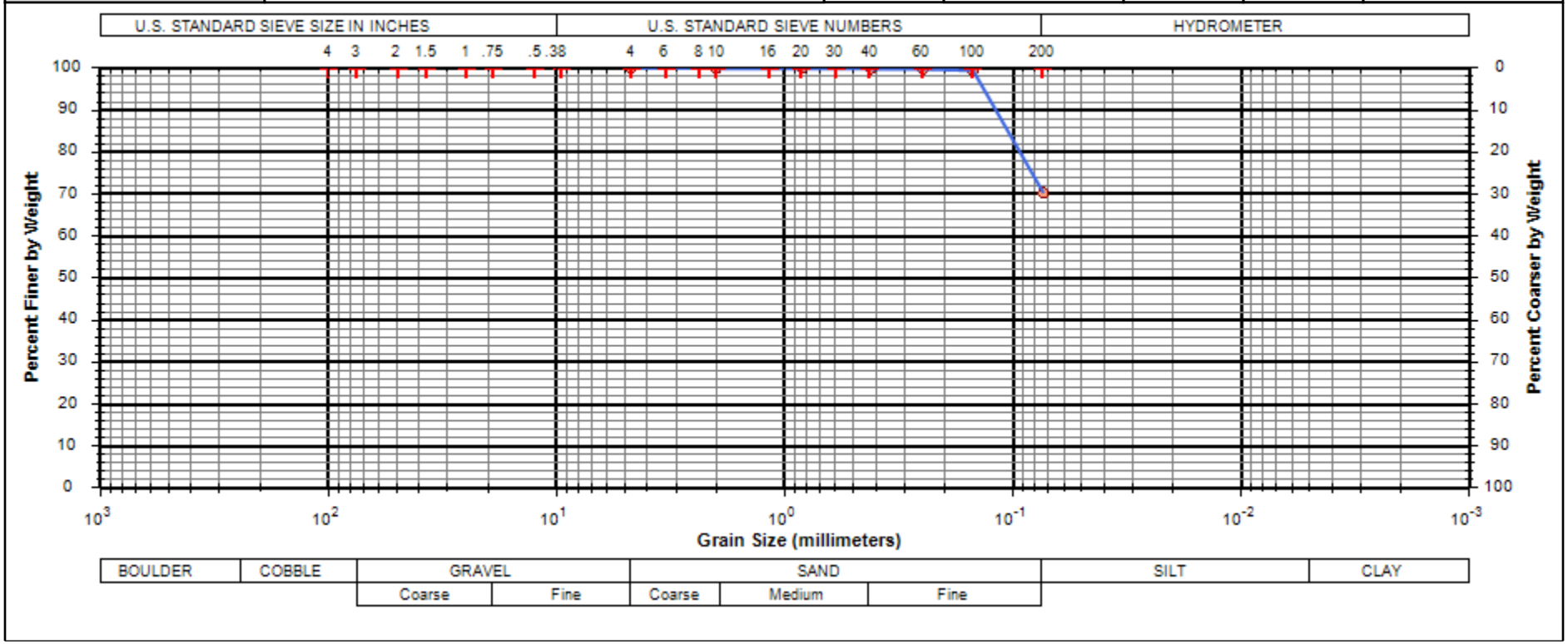
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.							**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX							**Date:**	02/06/2019
Boring No.:	W-1	**Sample No.:**	SS-10B & SS-11A	**Depth (ft.):**	26.0 - 30.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**	**USCS**
						0.0	29.7	70.3	ML

Sample Description:	Brown and gray wet very loose sandy SILT	**LL**	**PL**	**PI**	**Group Index**	**WC (%)**
		26	24	2		32.4



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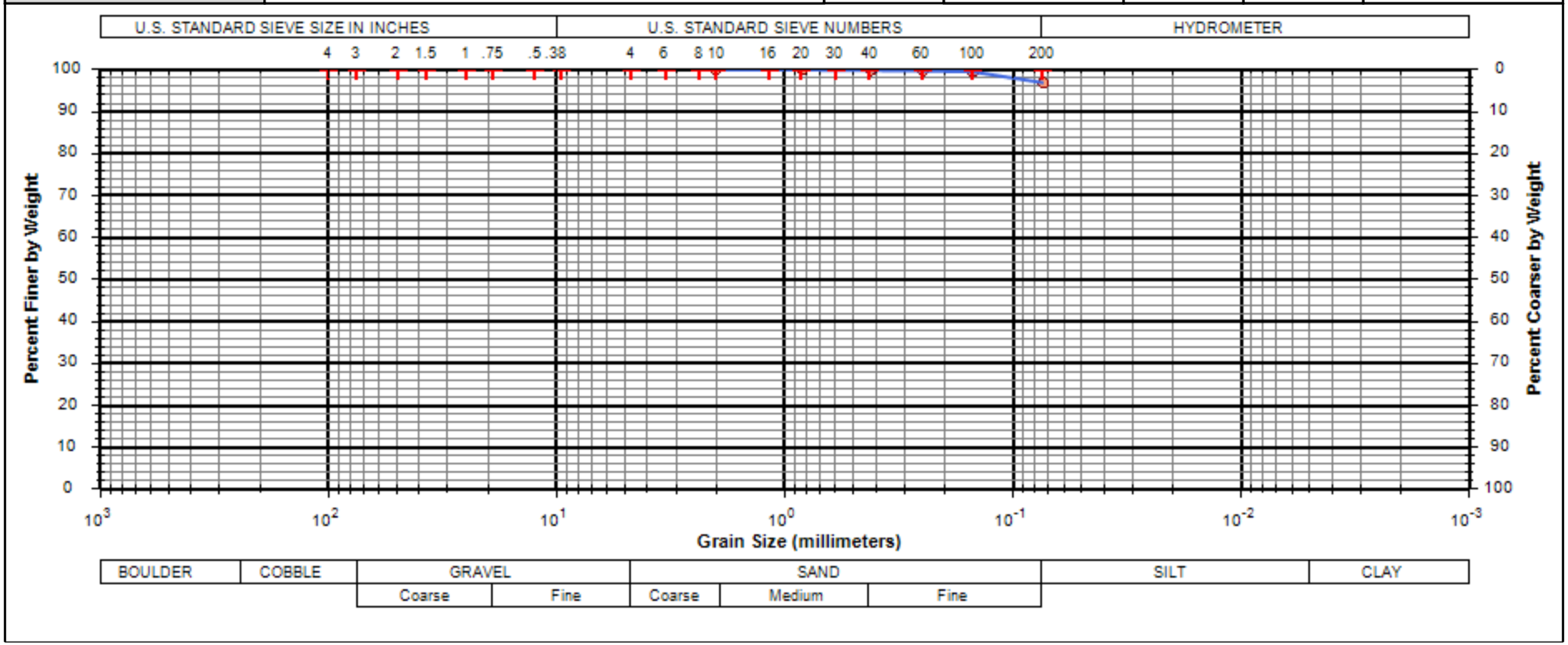
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/06/2019
Boring No.:	W-2	**Sample No.:**	SS-5	**Depth (ft.):**	10.0 - 11.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown wet very soft clayey SILT					0.0	3.1	96.9		ML
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						31	23	8		31.0



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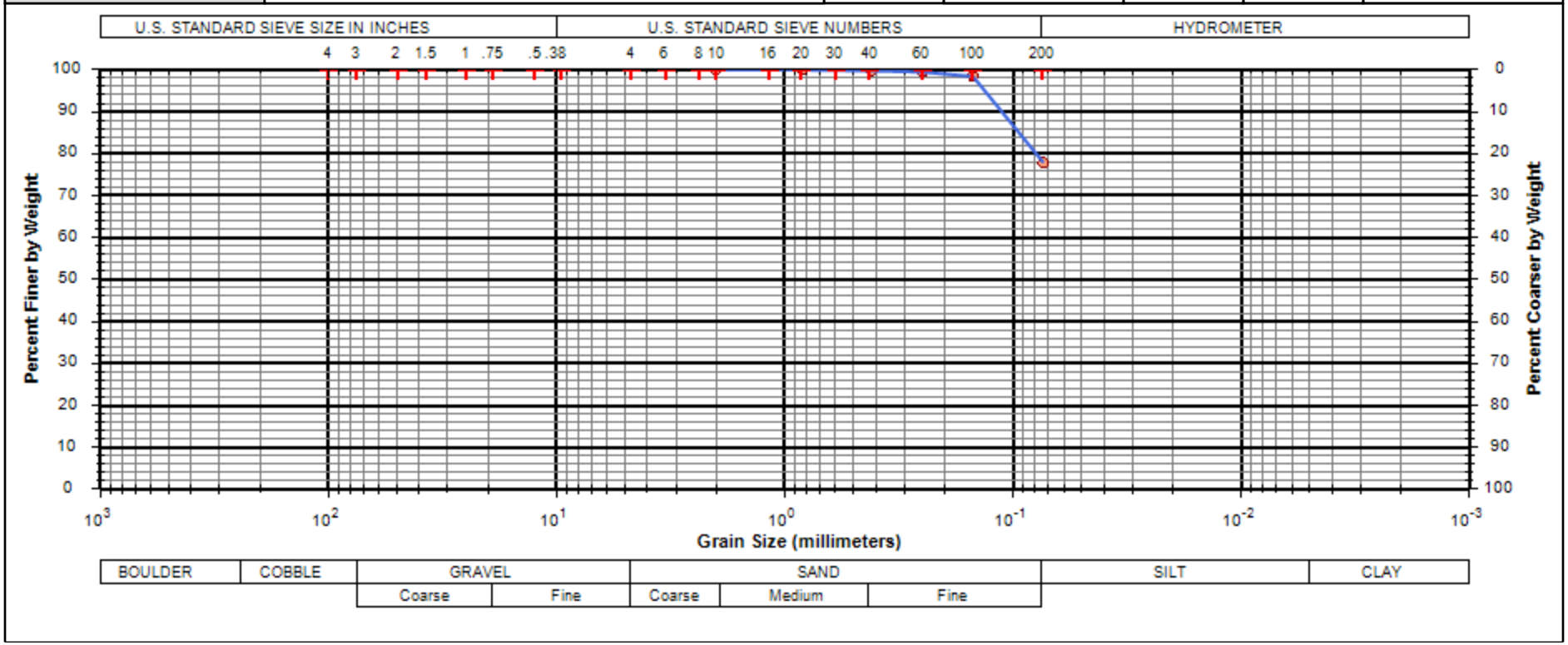
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/06/2019
Boring No.:	W-2	**Sample No.:**	SS-9	**Depth (ft.):**	20.0 - 21.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Gray very moist medium stiff LEAN CLAY with sand					0.0	22.1	77.9		CL
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						37	16	21		27.0



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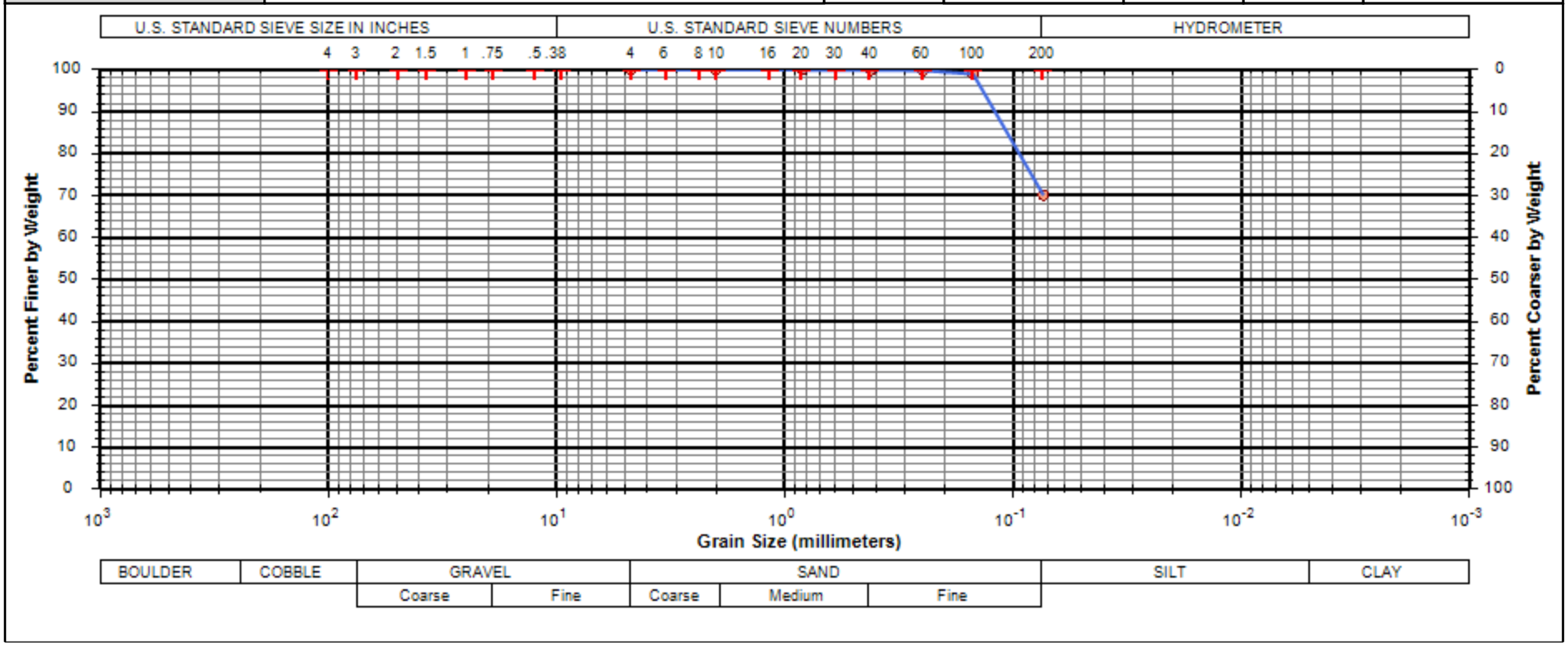
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/05/2019
Boring No.:	W-3	**Sample No.:**	SS-9	**Depth (ft.):**	20.0 - 21.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Gray, trace brown wet medium stiff sandy LEAN CLAY					0.0	29.9	70.1		CL
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						29	18	11		27.5



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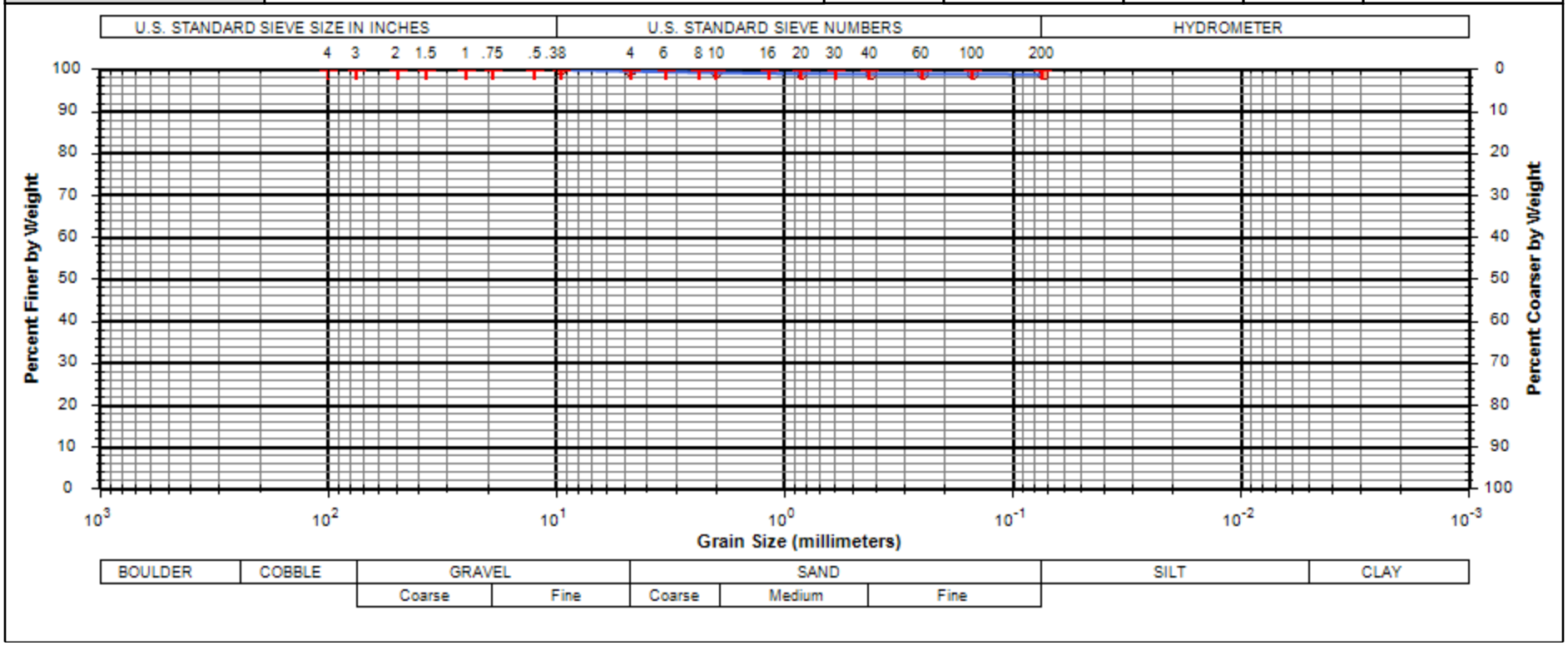
SIEVE ANALYSIS OF AGGREGATES ASTM C-136

Client:	Etegra, Inc.								**Project No.:**	J033011.01
Project:	Geotechnical Exploration, PIDC Kitchen and WWTP Improvements, Los Fresnos, TX								**Date:**	02/05/2019
Boring No.:	W-4	**Sample No.:**	SS-8A	**Depth (ft.):**	17.5 - 18.5	**Gravel (%)**	**Sand (%)**	**Silt & Clay (%)**		**USCS**
Sample Description:	Brown wet very soft clayey SILT					0.3	0.8	98.9		ML
						LL	**PL**	**PI**	**Group Index**	**WC (%)**
						34	25	9		35.0



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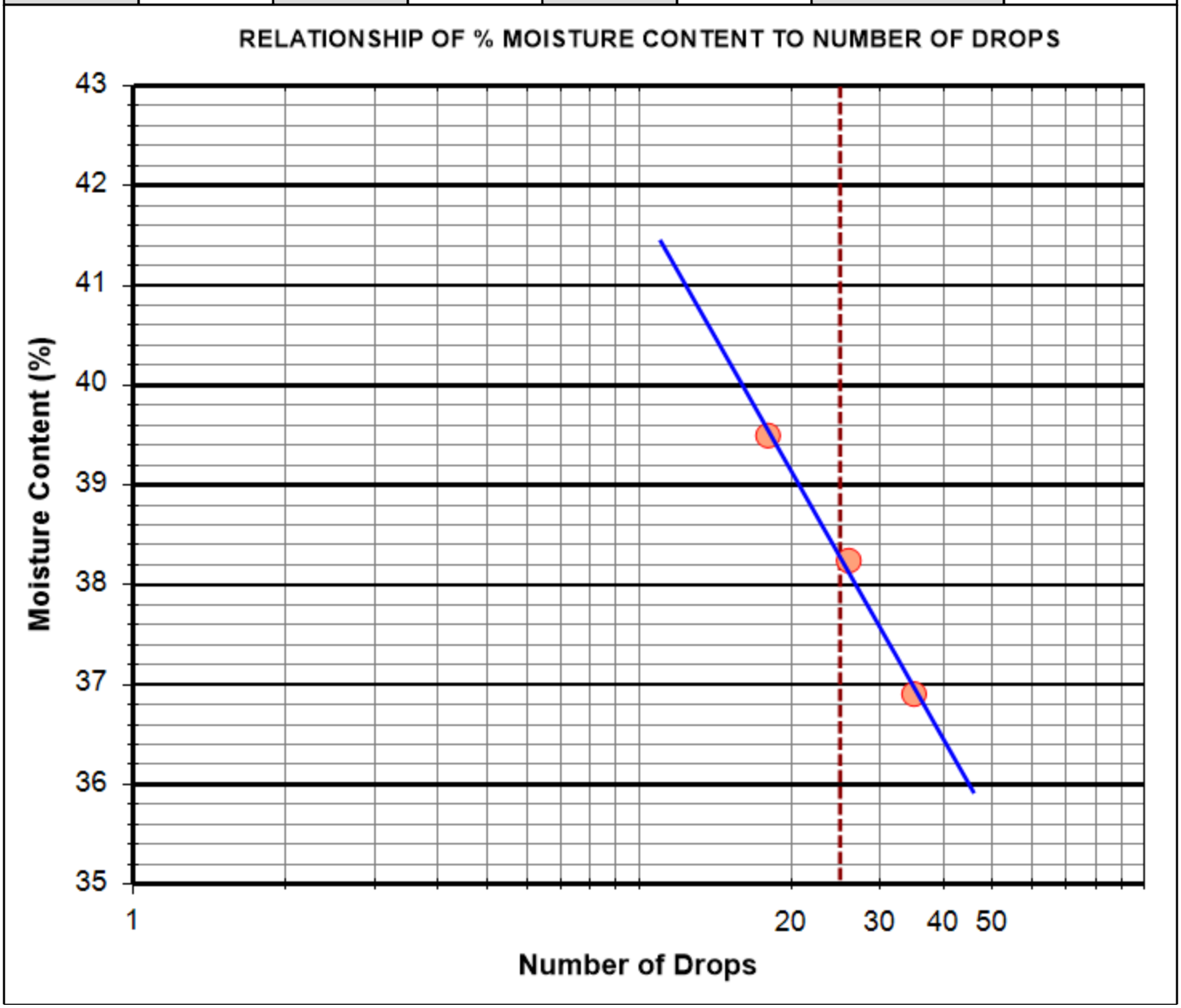
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/1/2019
Boring No.:	K-4	**Sample No.:**	Bucket	**Depth (ft.):**	0.0 - 5.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown to dark brown slightly moist very stiff LEAN CLAY					**In Situ Moisture Content:**	11.9%
Liquid Limit:	38	**Plastic Limit:**	18	**Plastic Index:**	20	**USCS:**	CL



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GEOTECHNOLOGY INC
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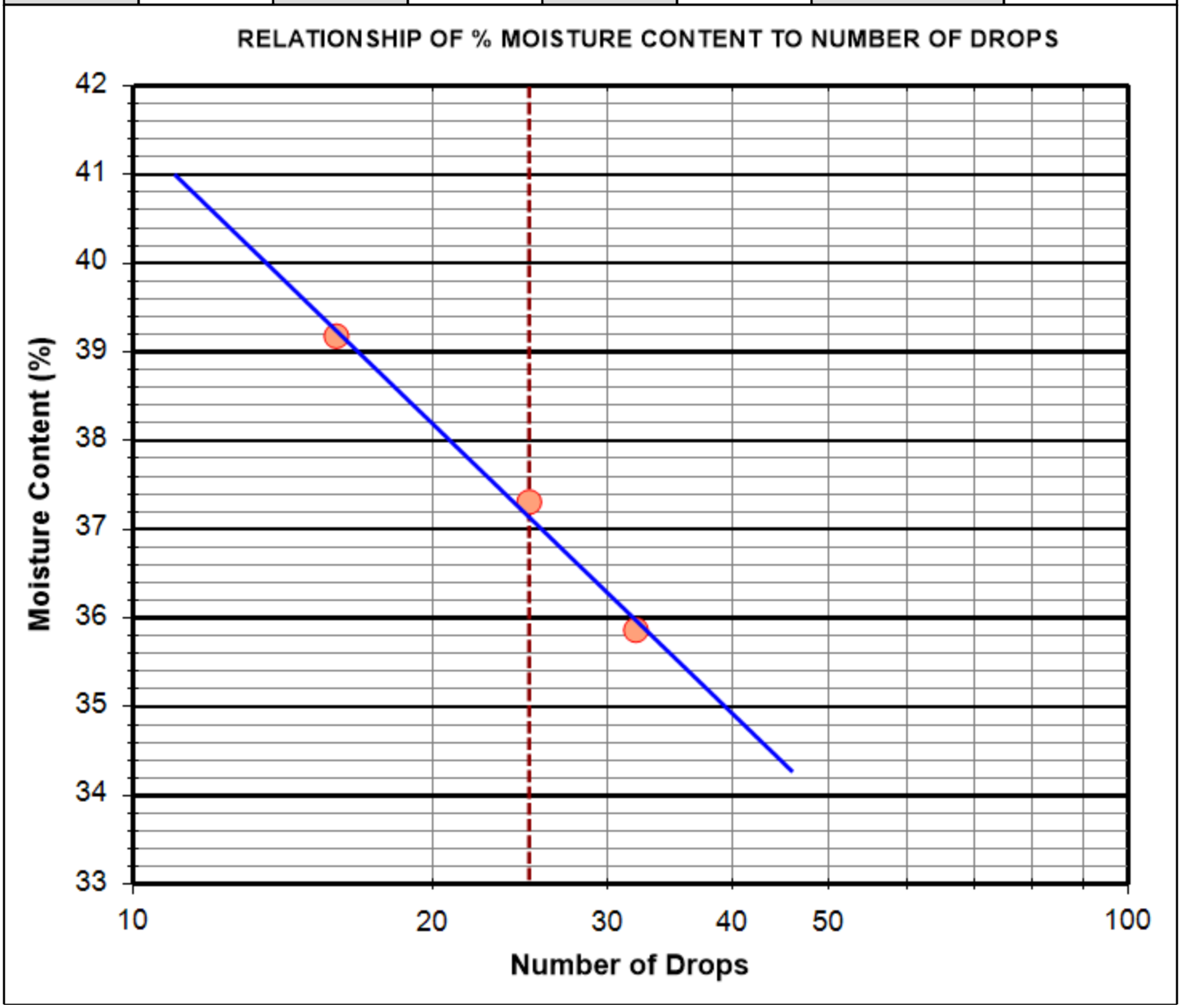
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/25/2019
Boring No.:	K-4	**Sample No.:**	SS-1	**Depth (ft.):**	0.0 - 1.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Dark brown slightly moist to moist very stiff LEAN CLAY					**In Situ Moisture Content:**	17.7%
Liquid Limit:	37	**Plastic Limit:**	23	**Plastic Index:**	14	**USCS:**	CL



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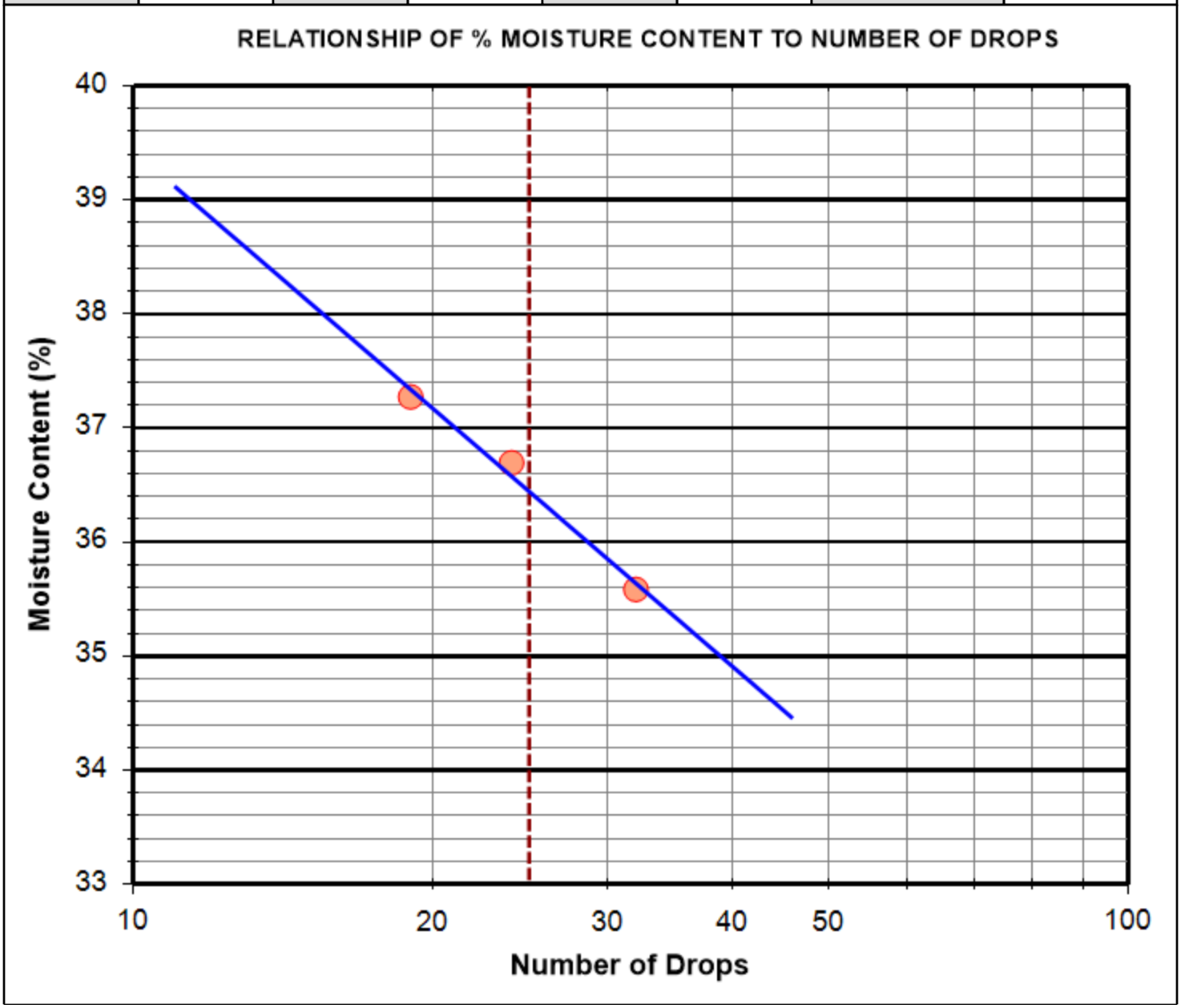
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/24/2019
Boring No.:	K-4	**Sample No.:**	PT-2A	**Depth (ft.):**	2.8 - 3.3	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist very stiff LEAN CLAY with carbonate deposits					**In Situ Moisture Content:**	15.6%
Liquid Limit:	36	**Plastic Limit:**	19	**Plastic Index:**	17	**USCS:**	CL



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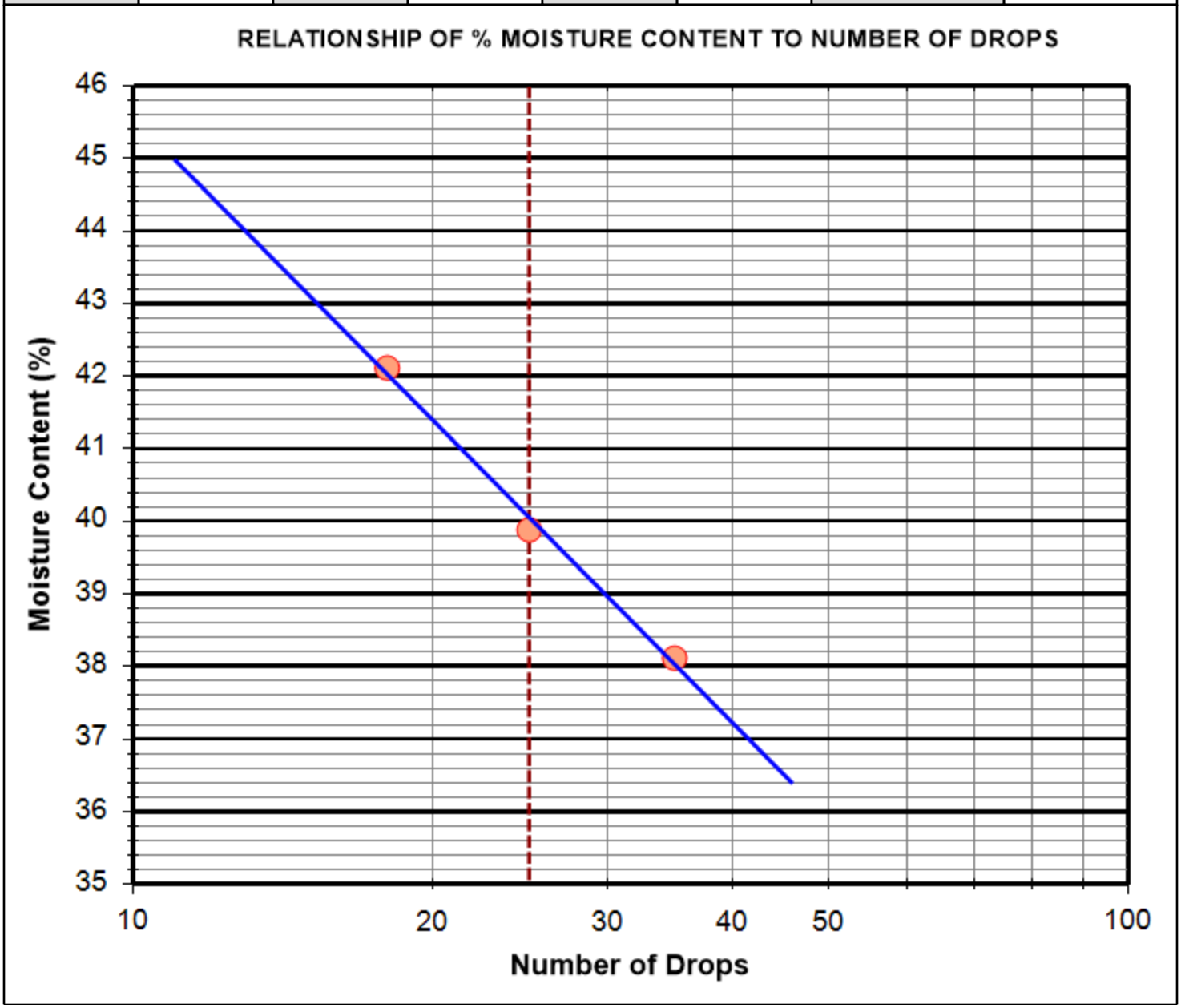
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/25/2019
Boring No.:	K-4	**Sample No.:**	SS-3	**Depth (ft.):**	5.0 - 6.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist very stiff LEAN CLAY with carbonate deposits					**In Situ Moisture Content:**	16.5%
Liquid Limit:	40	**Plastic Limit:**	20	**Plastic Index:**	20	**USCS:**	CL



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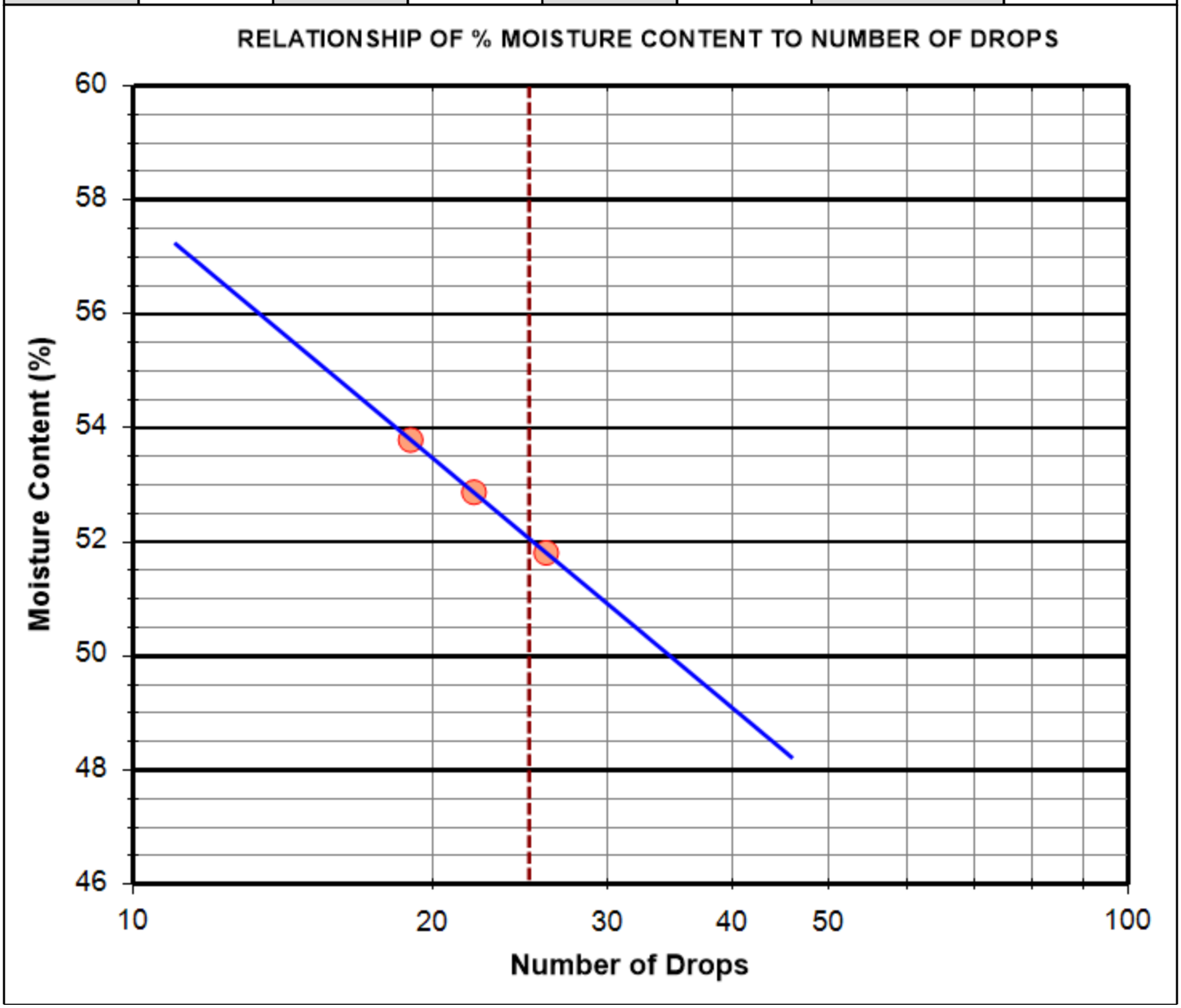
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/28/2019
Boring No.:	K-4	**Sample No.:**	SS-5	**Depth (ft.):**	10.0 - 11.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff FAT CLAY					**In Situ Moisture Content:**	28.9%
Liquid Limit:	52	**Plastic Limit:**	20	**Plastic Index:**	32	**USCS:**	CH



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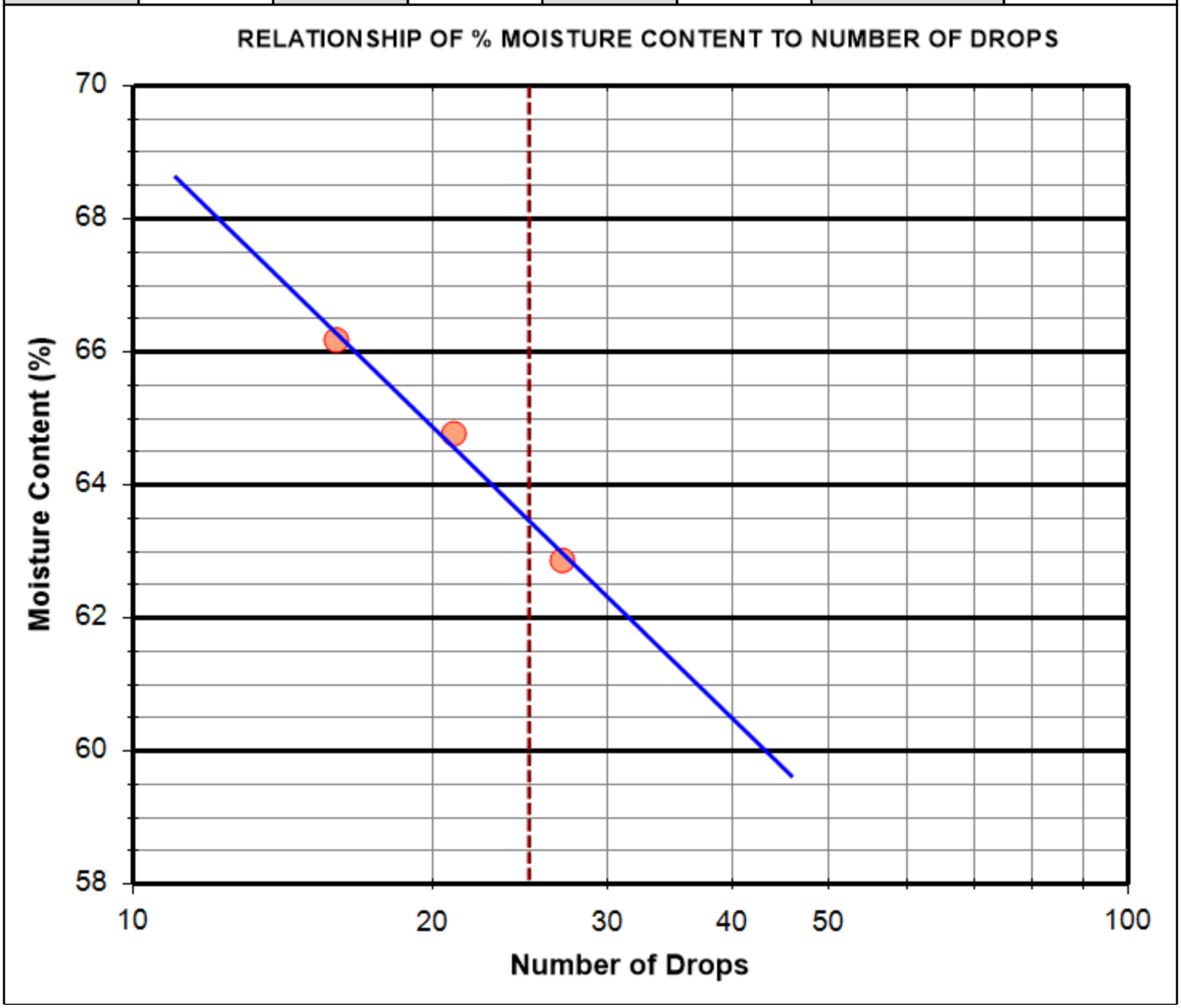
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/28/2019
Boring No.:	K-4	**Sample No.:**	SS-6	**Depth (ft.):**	12.5 - 14.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff to very stiff FAT CLAY					**In Situ Moisture Content:**	27.5%
Liquid Limit:	63	**Plastic Limit:**	22	**Plastic Index:**	41	**USCS:**	CH



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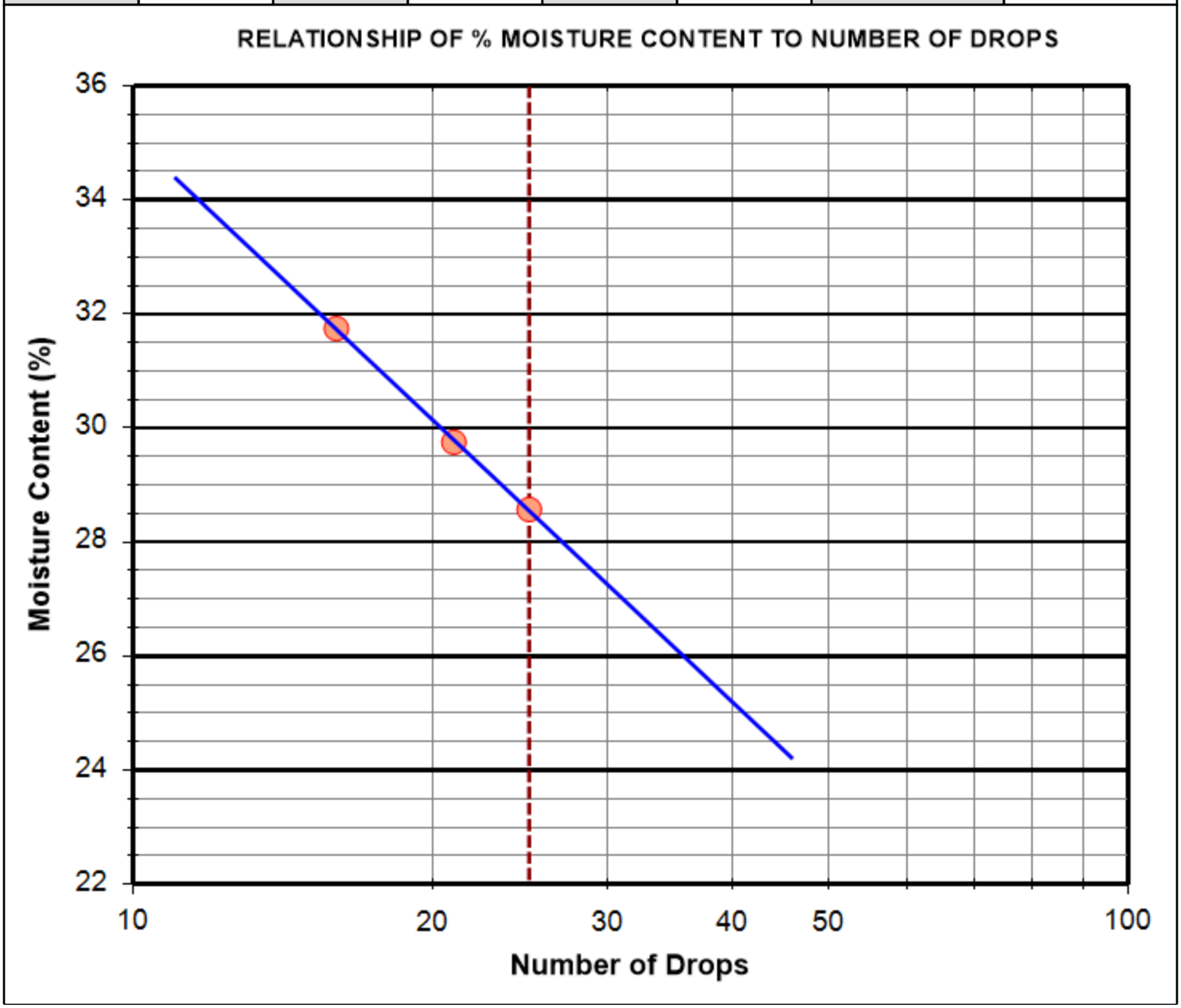
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/30/2019
Boring No.:	K-4	**Sample No.:**	SS-8	**Depth (ft.):**	17.5 - 19.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown wet very soft clayey SILT					**In Situ Moisture Content:**	31.8%
Liquid Limit:	29	**Plastic Limit:**	26	**Plastic Index:**	3	**USCS:**	ML



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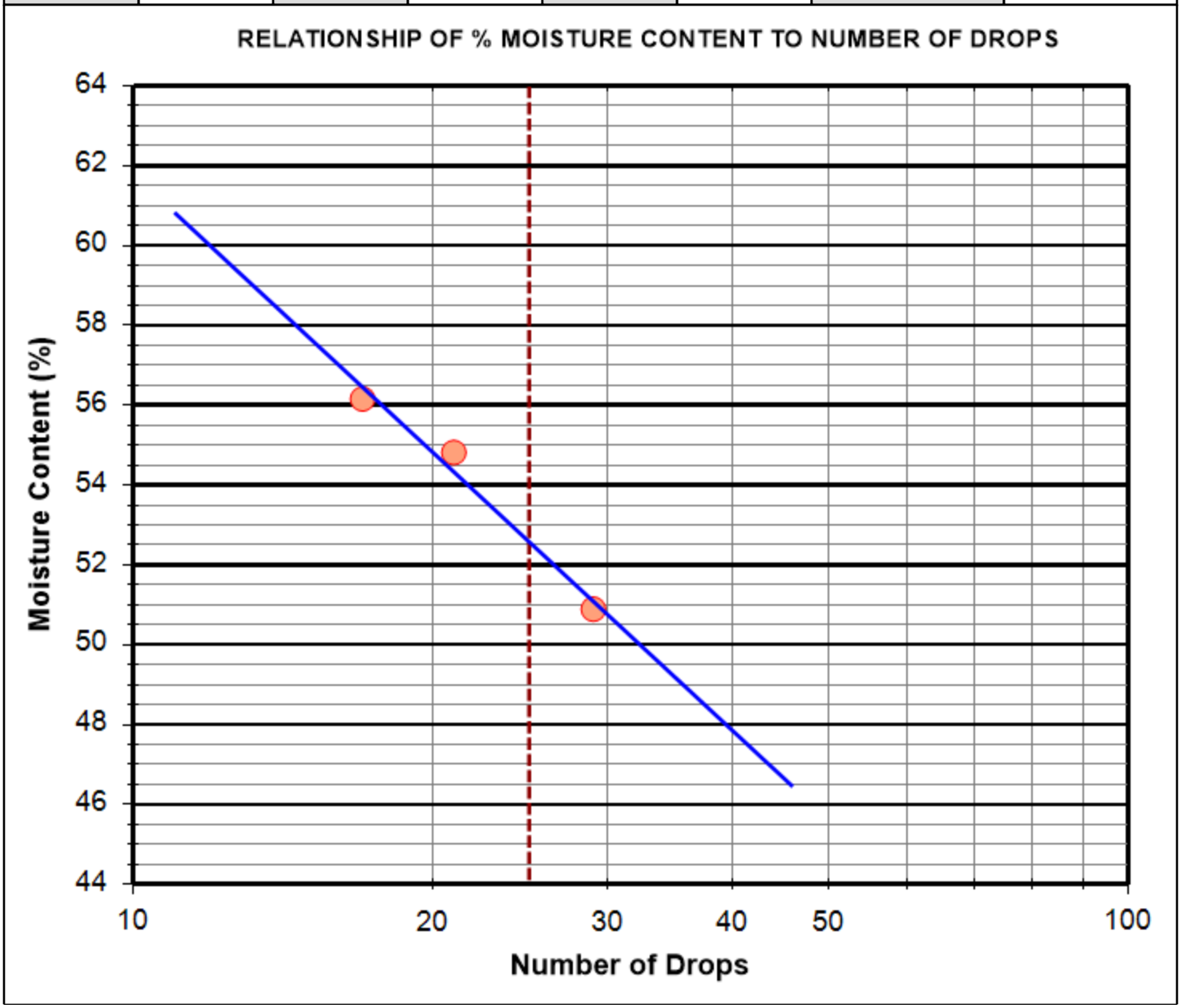
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/30/2019
Boring No.:	K-4	**Sample No.:**	SS-9	**Depth (ft.):**	20.0´ - 21.5´	**Sample Preparation Condition:**	Air Dried
Sample Description:	Gray moist medium stiff to stiff FAT CLAY					**In Situ Moisture Content:**	33.7%
Liquid Limit:	53	**Plastic Limit:**	18	**Plastic Index:**	35	**USCS:**	CH



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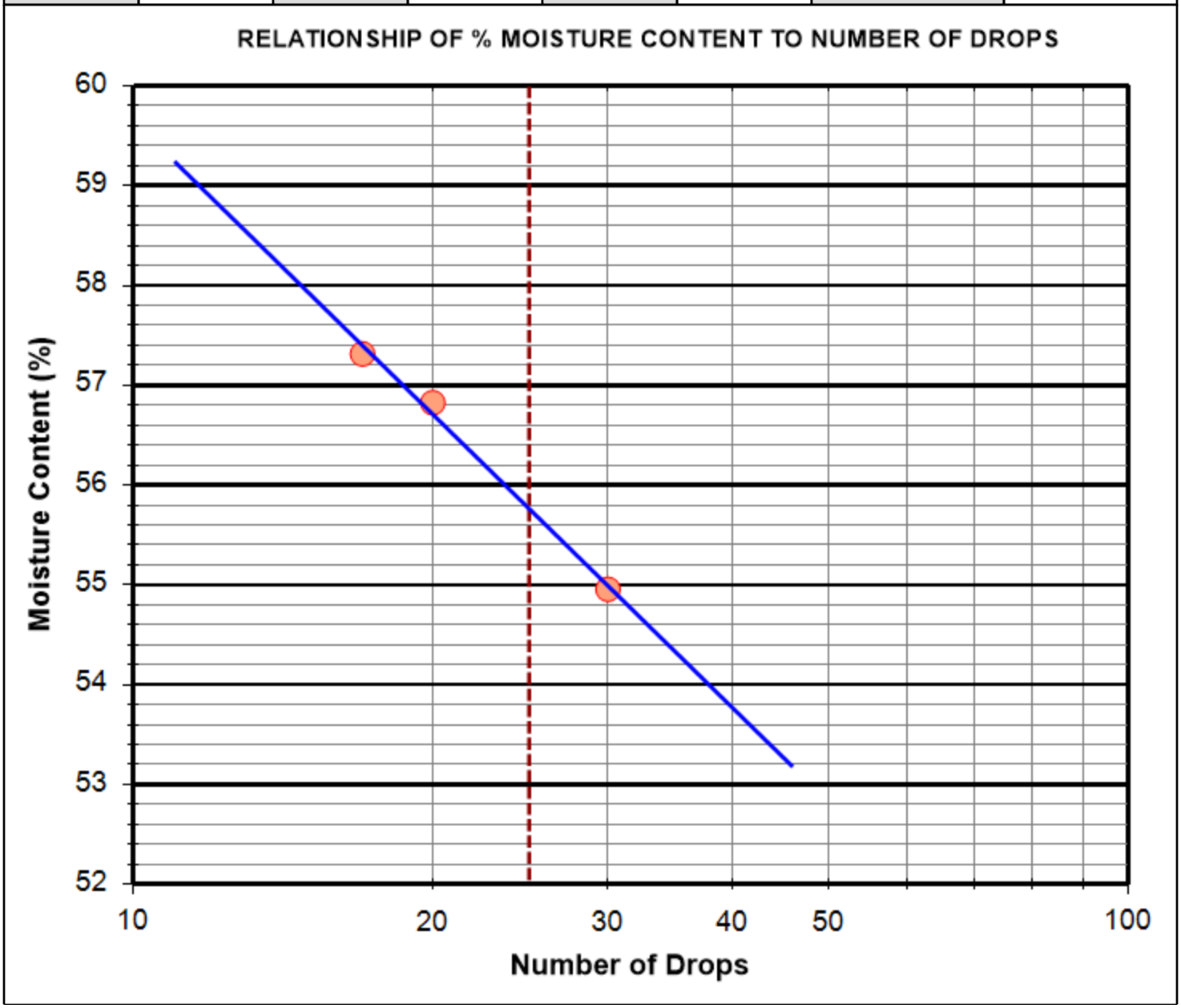
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/30/2019
Boring No.:	K-4	**Sample No.:**	SS-10	**Depth (ft.):**	25.0´ - 26.5´	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brownish gray moist stiff FAT CLAY					**In Situ Moisture Content:**	30.2%
Liquid Limit:	56	**Plastic Limit:**	20	**Plastic Index:**	36	**USCS:**	CH



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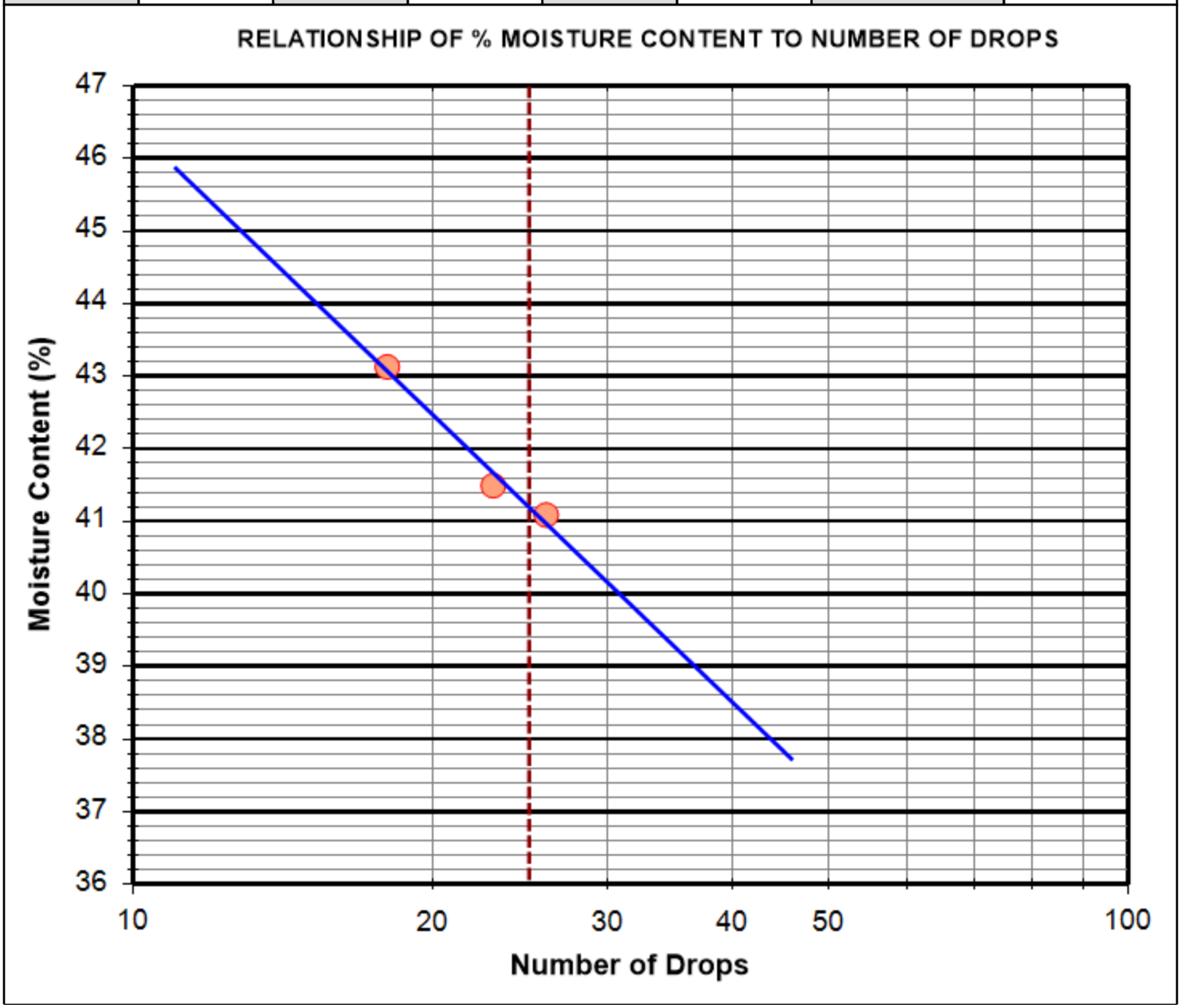
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/1/2019
Boring No.:	K-5	**Sample No.:**	SS-1	**Depth (ft.):**	0.0 - 1.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Dark brown moist very stiff LEAN CLAY with organics, trace roots					**In Situ Moisture Content:**	15.9%
Liquid Limit:	41	**Plastic Limit:**	19	**Plastic Index:**	22	**USCS:**	CL



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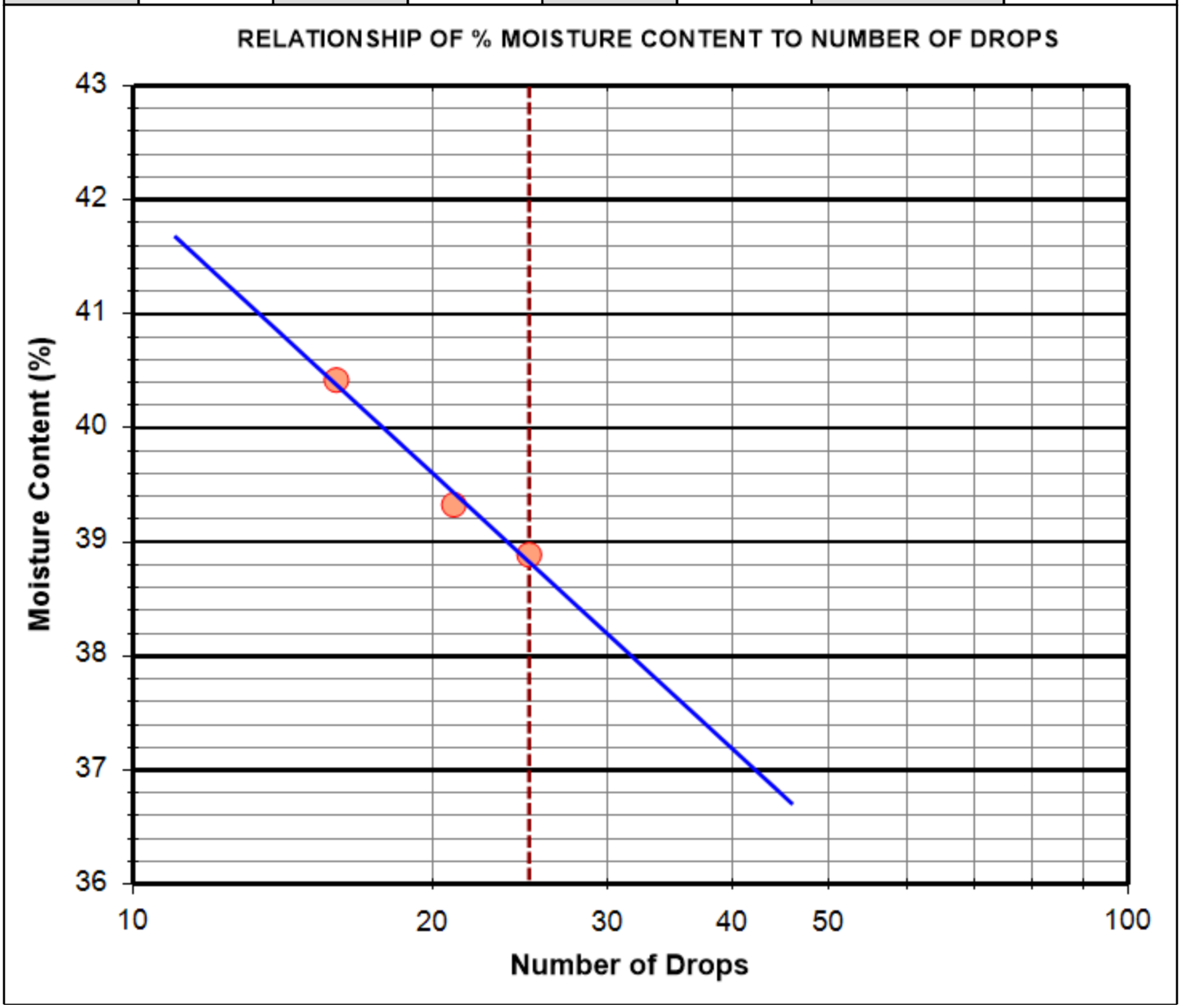
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/28/2019
Boring No.:	K-5	**Sample No.:**	SS-2	**Depth (ft.):**	2.5 - 4.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown slightly moist very stiff LEAN CLAY with carbonate deposits					**In Situ Moisture Content:**	13.5%
Liquid Limit:	39	**Plastic Limit:**	17	**Plastic Index:**	22	**USCS:**	CL



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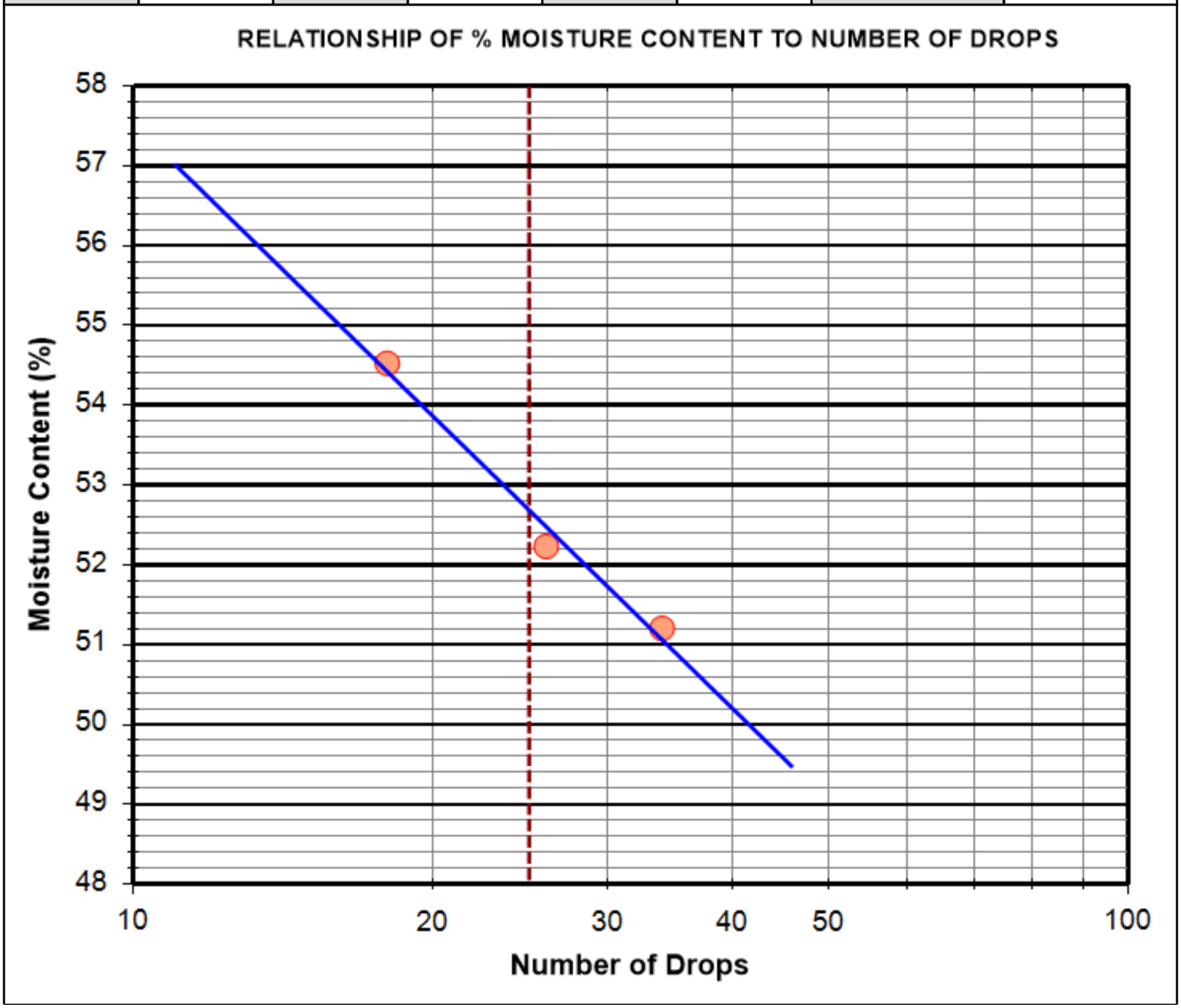
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/23/2019
Boring No.:	K-5	**Sample No.:**	PT-3B	**Depth (ft.):**	5.3 - 5.7	**Sample Preparation Condition:**	Air Dried
Sample Description:	Light brown to brown moist very stiff FAT CLAY					**In Situ Moisture Content:**	11.8%
Liquid Limit:	53	**Plastic Limit:**	21	**Plastic Index:**	32	**USCS:**	CH



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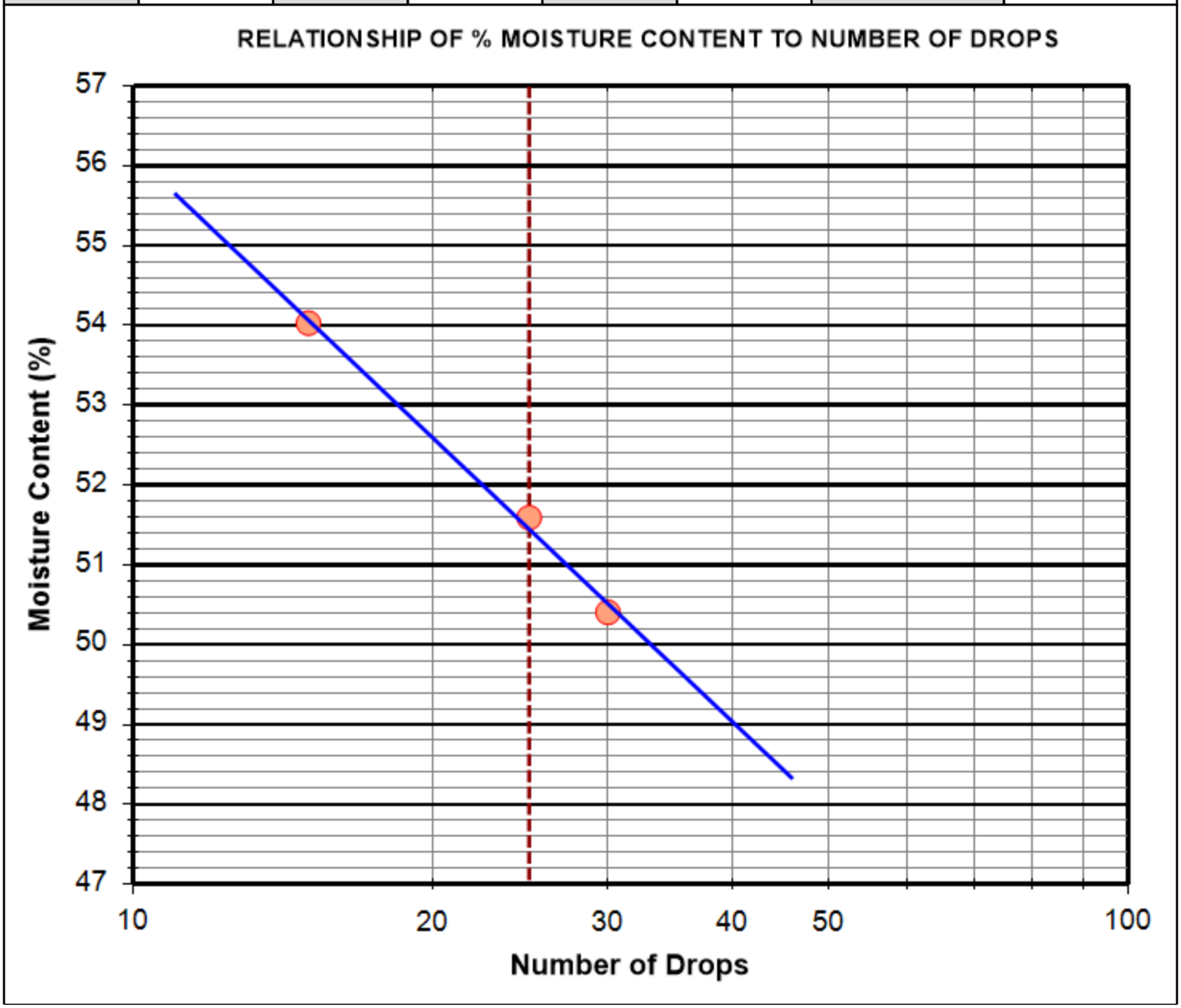
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/28/2019
Boring No.:	K-5	**Sample No.:**	SS-4	**Depth (ft.):**	7.5 - 9.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff FAT CLAY					**In Situ Moisture Content:**	28.3%
Liquid Limit:	51	**Plastic Limit:**	20	**Plastic Index:**	31	**USCS:**	CH



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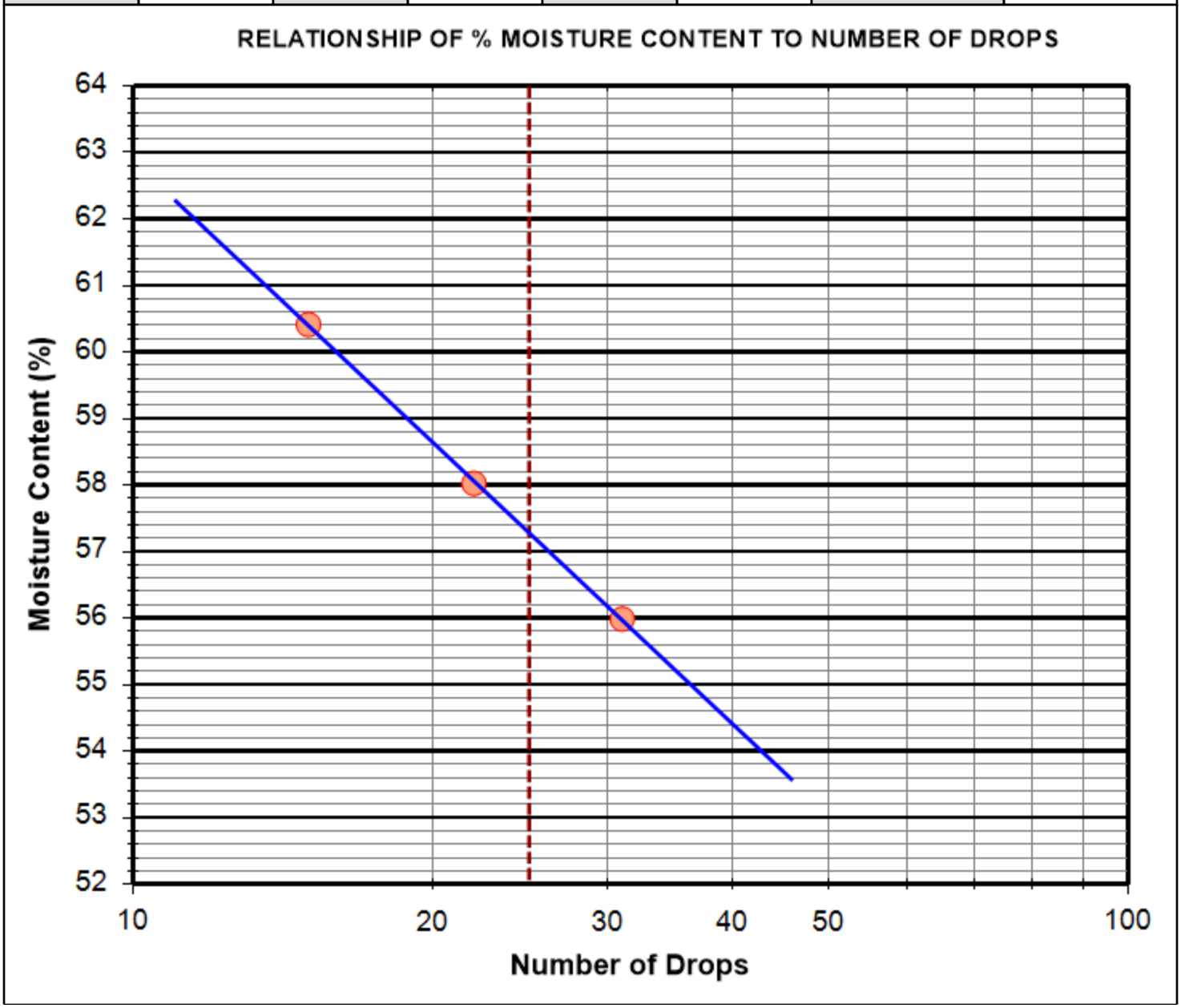
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/25/2019
Boring No.:	K-5	**Sample No.:**	SS-5	**Depth (ft.):**	10.0 - 11.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown trace gray moist medium stiff to stiff FAT CLAY					**In Situ Moisture Content:**	30.8%
Liquid Limit:	57	**Plastic Limit:**	25	**Plastic Index:**	32	**USCS:**	CH



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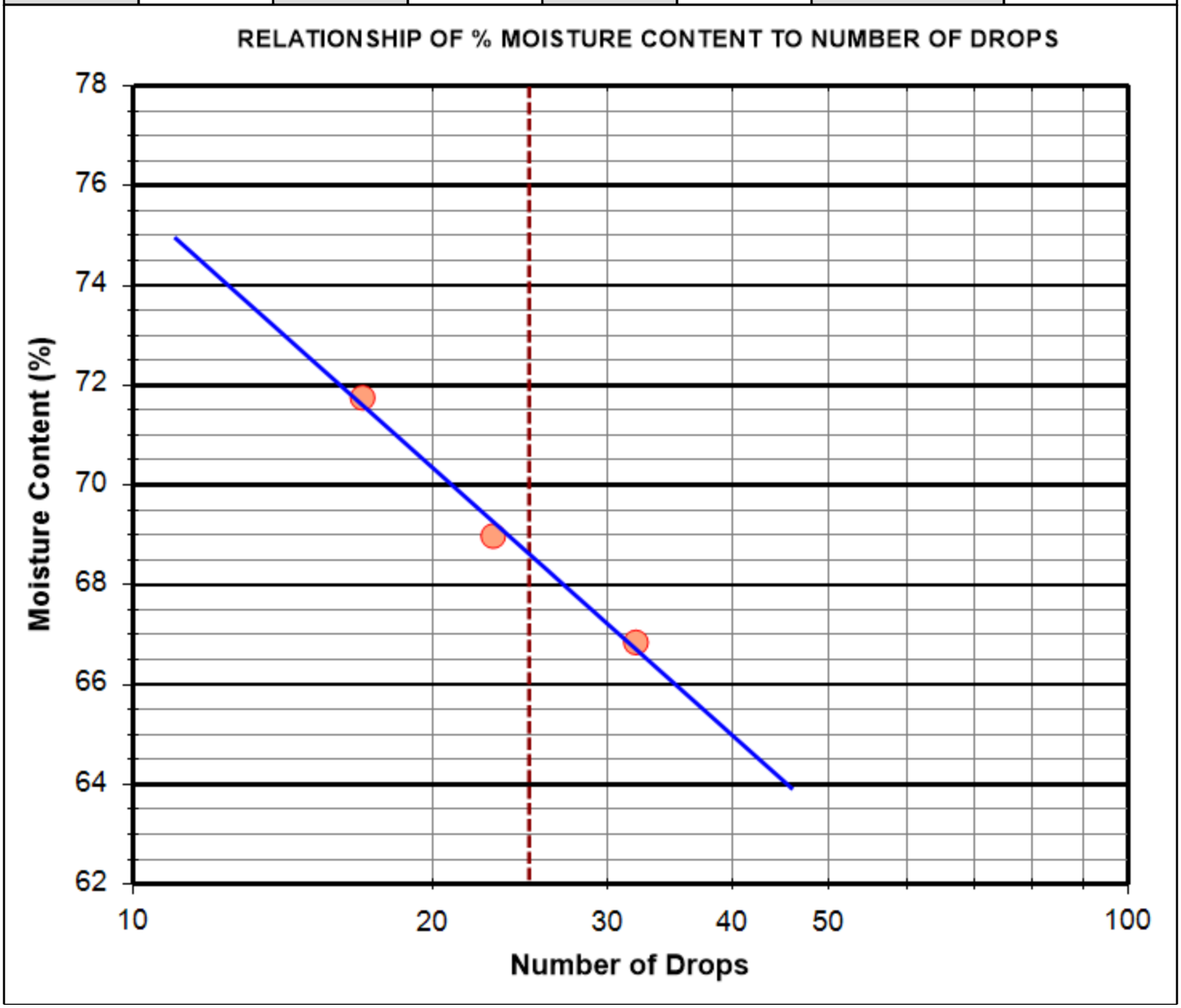
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/30/2019
Boring No.:	K-5	**Sample No.:**	SS-6	**Depth (ft.):**	12.5´ - 14.0´	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown and gray moist stiff FAT CLAY					**In Situ Moisture Content:**	31.4%
Liquid Limit:	69	**Plastic Limit:**	23	**Plastic Index:**	46	**USCS:**	CH



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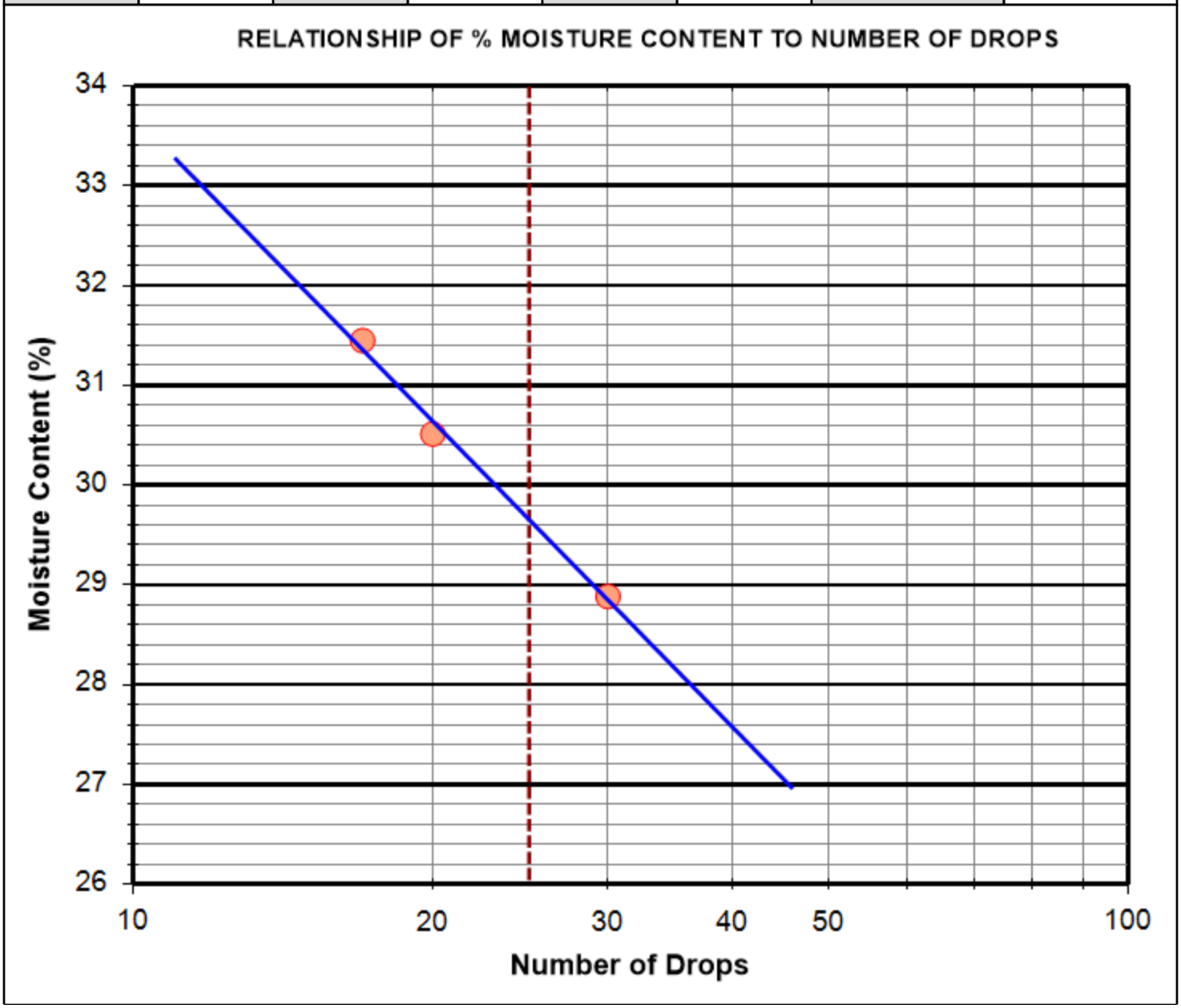
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/1/2019
Boring No.:	K-5	**Sample No.:**	SS-7 & SS-8	**Depth (ft.):**	15.0 - 19.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown wet very soft clayey SILT					**In Situ Moisture Content:**	32.2%
Liquid Limit:	30	**Plastic Limit:**	25	**Plastic Index:**	5	**USCS:**	ML



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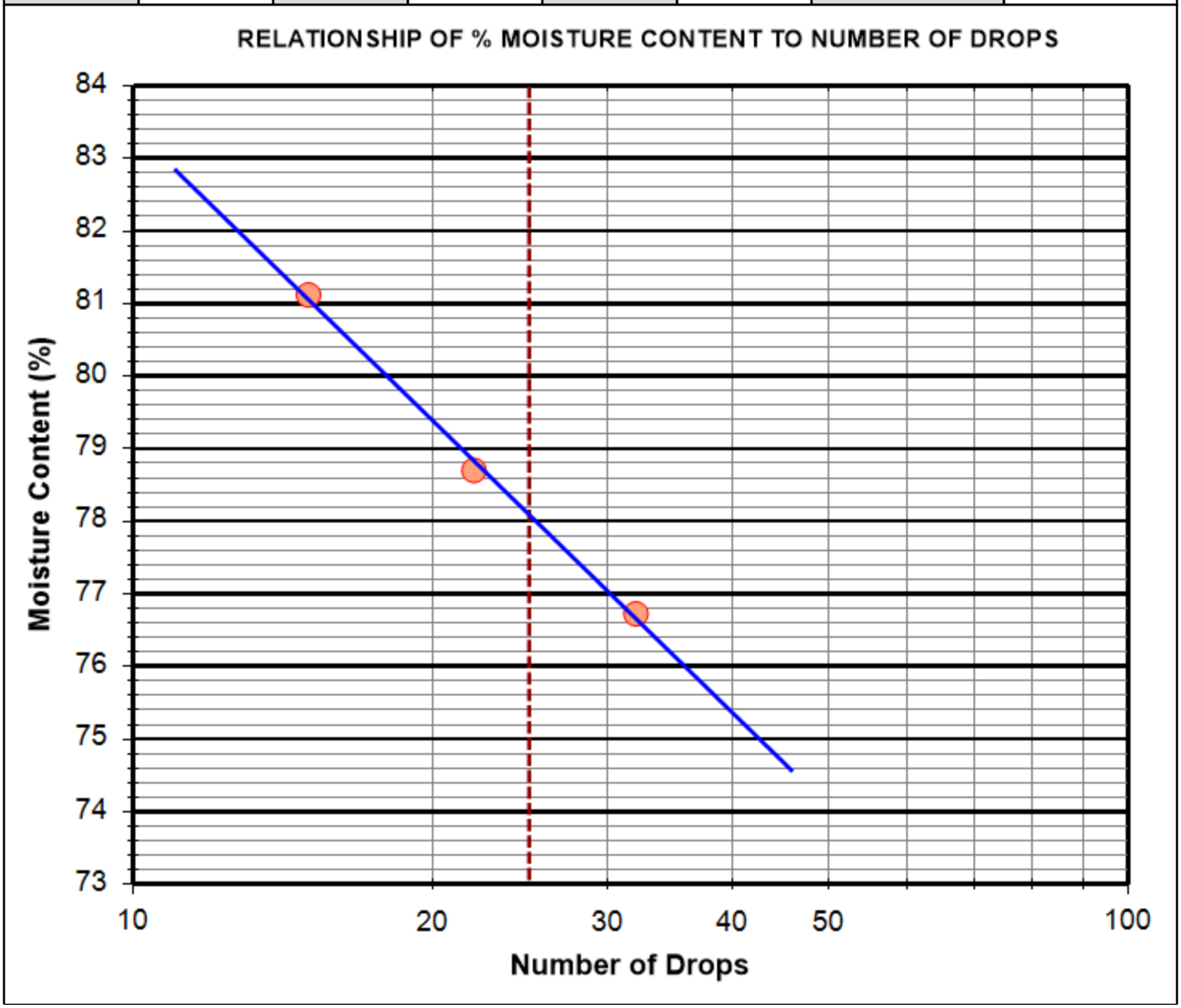
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/25/2019
Boring No.:	K-5	**Sample No.:**	SS-9	**Depth (ft.):**	20.0 - 21.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Gray moist stiff FAT CLAY					**In Situ Moisture Content:**	34.0%
Liquid Limit:	78	**Plastic Limit:**	26	**Plastic Index:**	52	**USCS:**	CH



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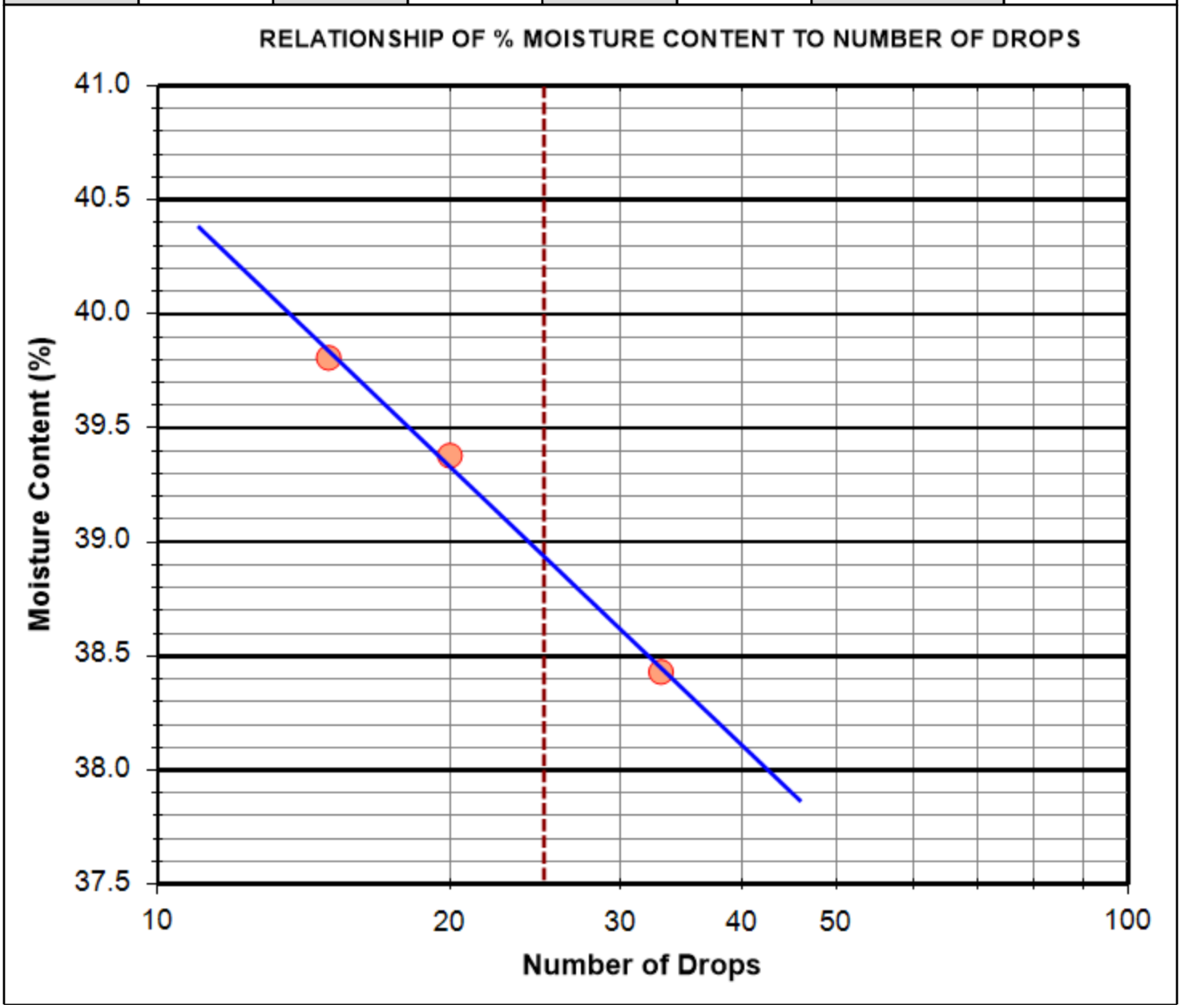
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/25/2019
Boring No.:	K-6	**Sample No.:**	SS-1	**Depth (ft.):**	0.0 - 1.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Dark brown moist very stiff LEAN CLAY, trace roots					**In Situ Moisture Content:**	17.5%
Liquid Limit:	39	**Plastic Limit:**	20	**Plastic Index:**	19	**USCS:**	CL



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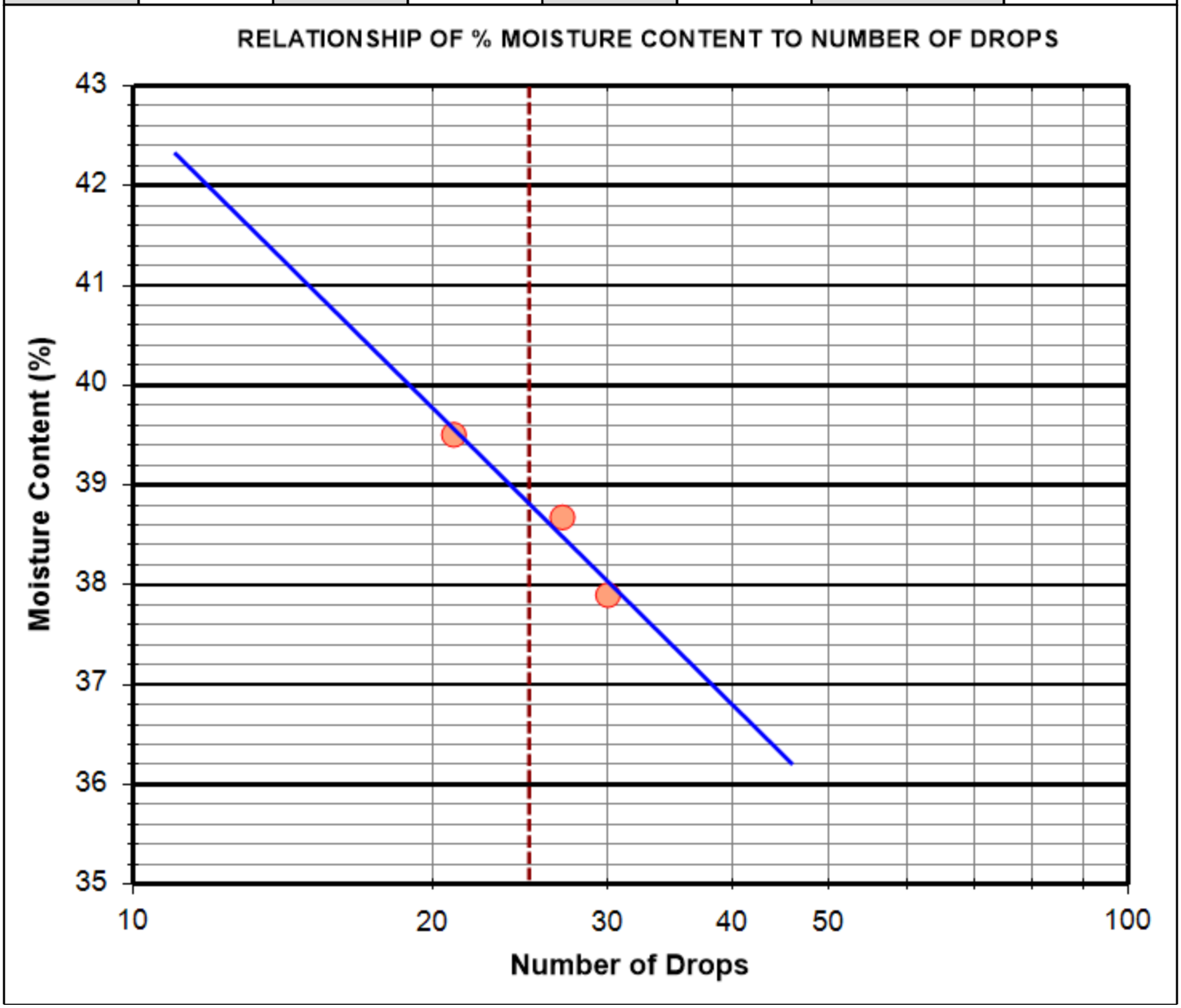
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/25/2019
Boring No.:	K-6	**Sample No.:**	SS-2	**Depth (ft.):**	2.5 - 4.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist medium stiff to stiff LEAN CLAY					**In Situ Moisture Content:**	19.3%
Liquid Limit:	39	**Plastic Limit:**	19	**Plastic Index:**	20	**USCS:**	CL



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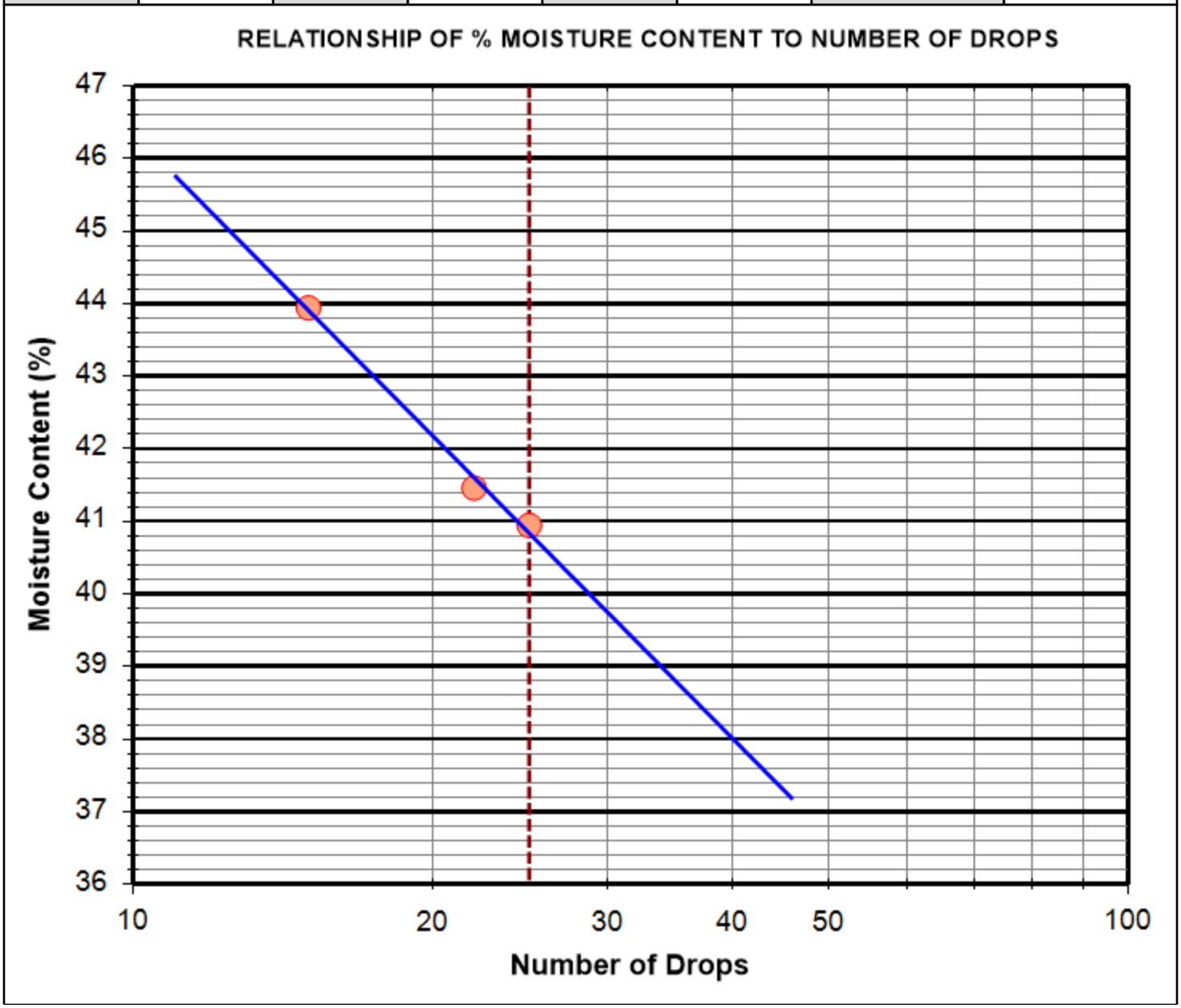
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/1/2019
Boring No.:	K-6	**Sample No.:**	SS-3A	**Depth (ft.):**	5.0 - 6.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist medium stiff to stiff LEAN CLAY					**In Situ Moisture Content:**	20.4%
Liquid Limit:	41	**Plastic Limit:**	19	**Plastic Index:**	22	**USCS:**	CL



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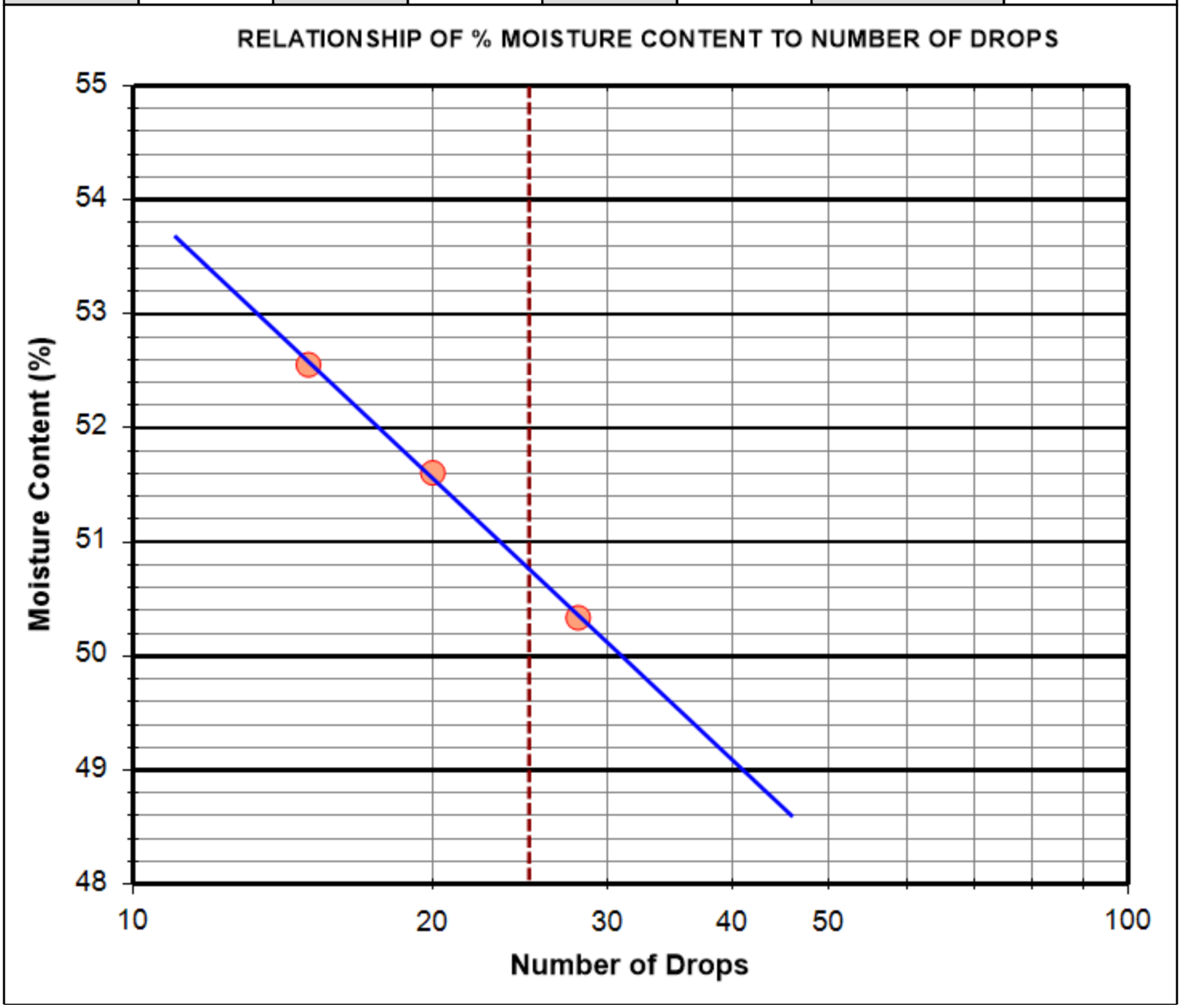
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/5/2019
Boring No.:	K-6	**Sample No.:**	SS-4	**Depth (ft.):**	7.5 - 9.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff FAT CLAY					**In Situ Moisture Content:**	26.1%
Liquid Limit:	51	**Plastic Limit:**	21	**Plastic Index:**	30	**USCS:**	CH



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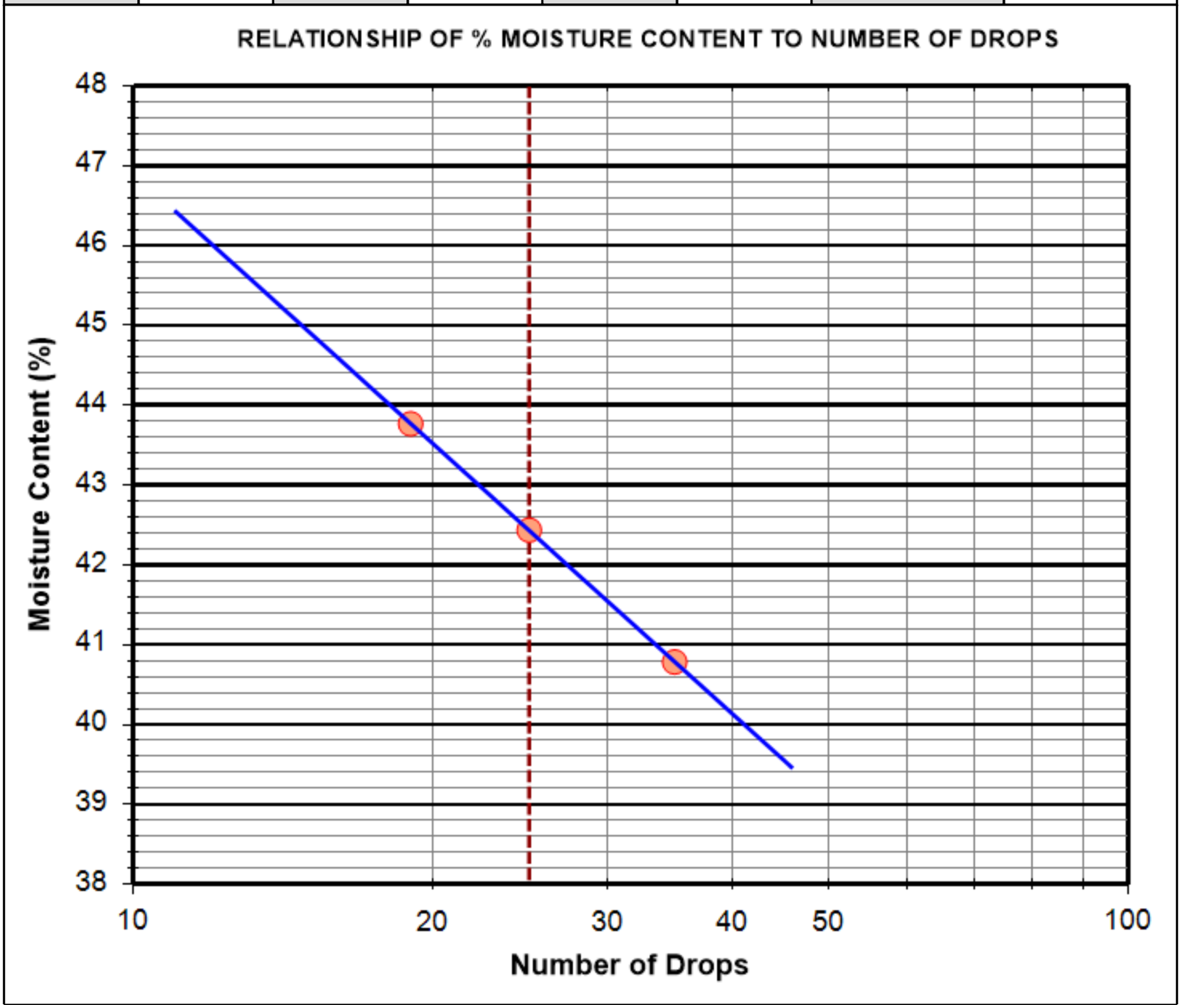
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/23/2019
Boring No.:	K-6	**Sample No.:**	PT-5B	**Depth (ft.):**	10.2 - 10.7	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown very moist medium stiff LEAN CLAY					**In Situ Moisture Content:**	30.2%
Liquid Limit:	42	**Plastic Limit:**	24	**Plastic Index:**	18	**USCS:**	CL



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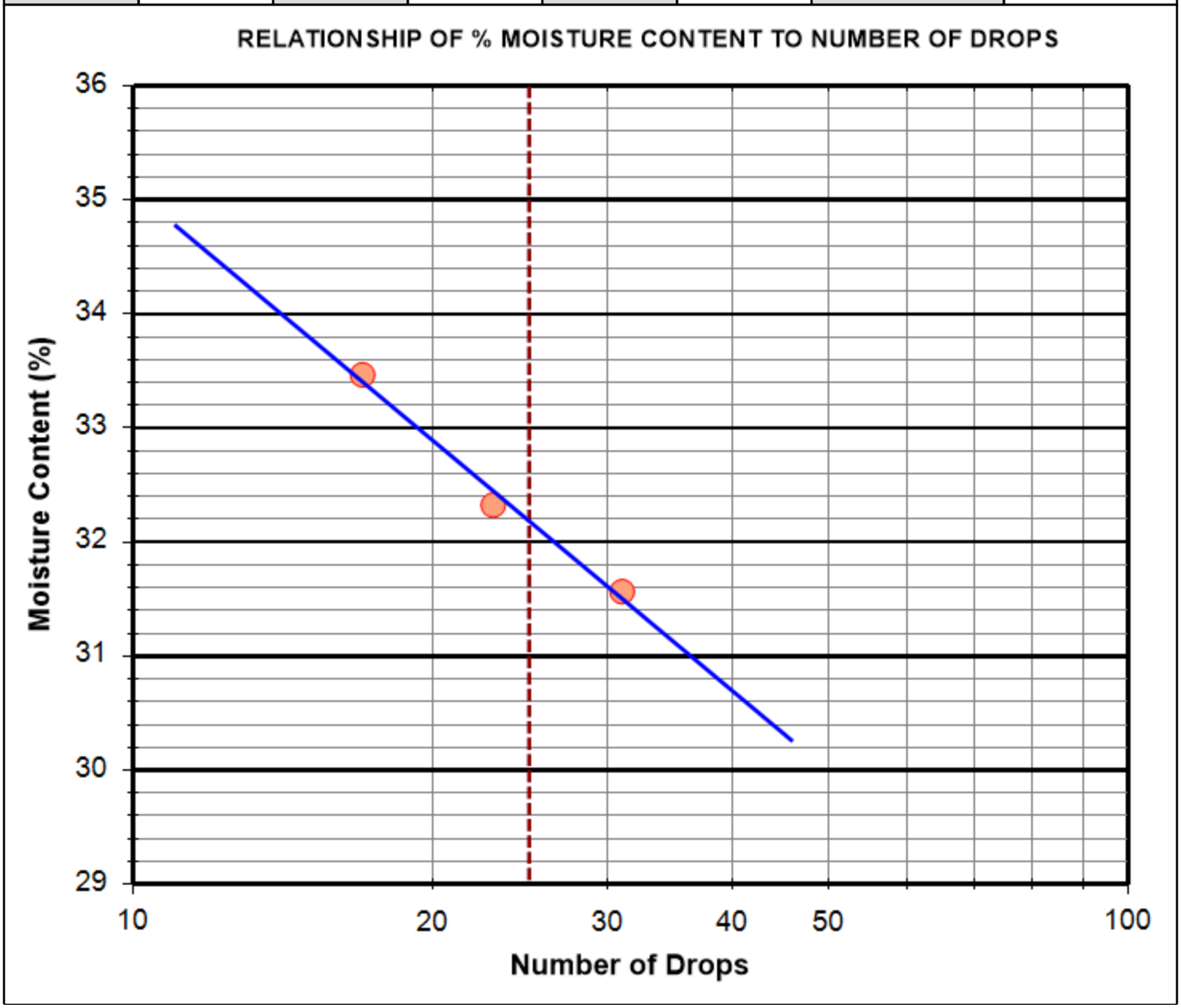
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/5/2019
Boring No.:	K-6	**Sample No.:**	SS-8	**Depth (ft.):**	17.5 - 19.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown wet very soft clayey SILT					**In Situ Moisture Content:**	30.5%
Liquid Limit:	32	**Plastic Limit:**	25	**Plastic Index:**	7	**USCS:**	ML



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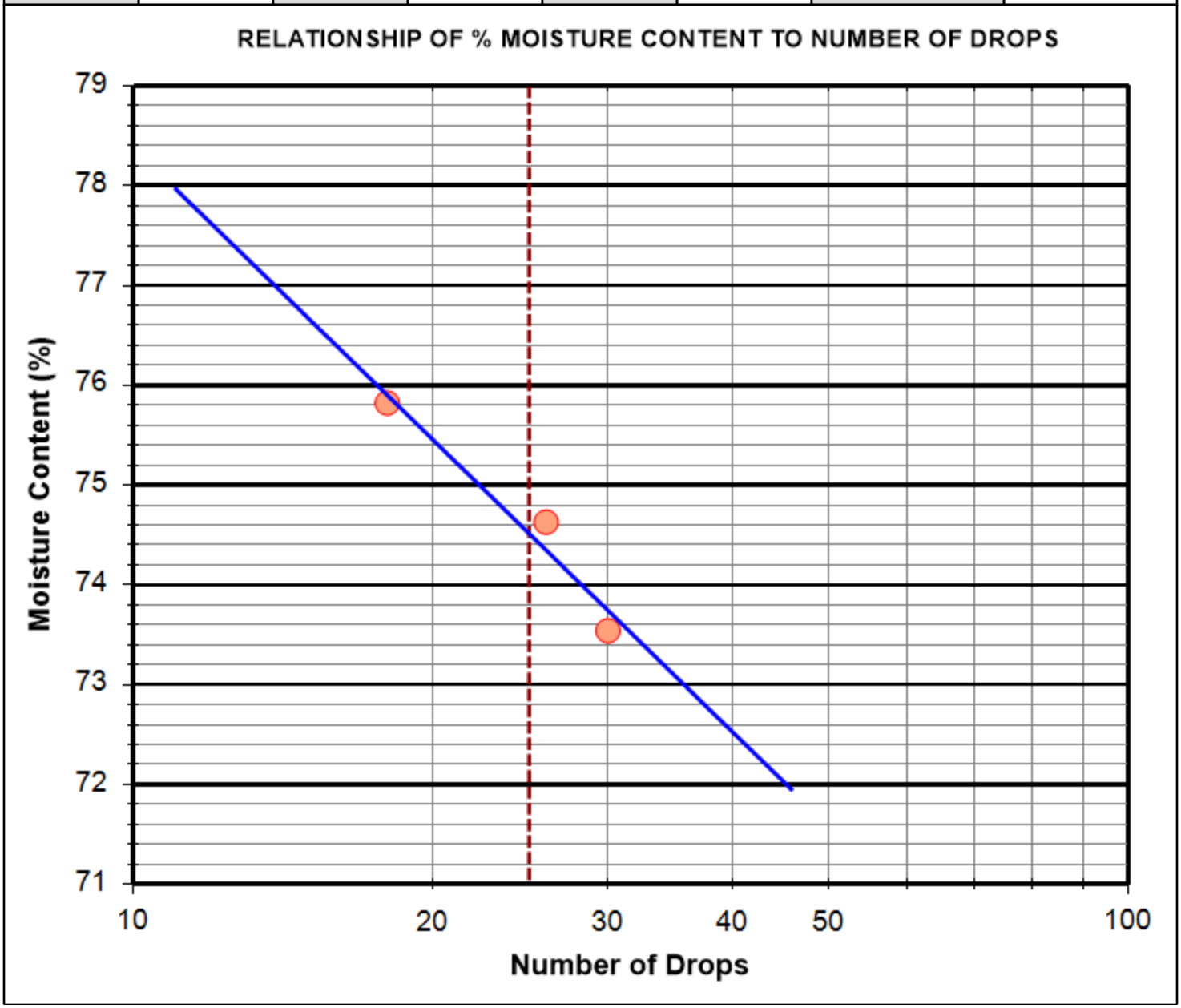
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/5/2019
Boring No.:	K-6	**Sample No.:**	SS-9	**Depth (ft.):**	20.0 - 21.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown very moist stiff FAT CLAY					**In Situ Moisture Content:**	37.7%
Liquid Limit:	75	**Plastic Limit:**	22	**Plastic Index:**	53	**USCS:**	CH



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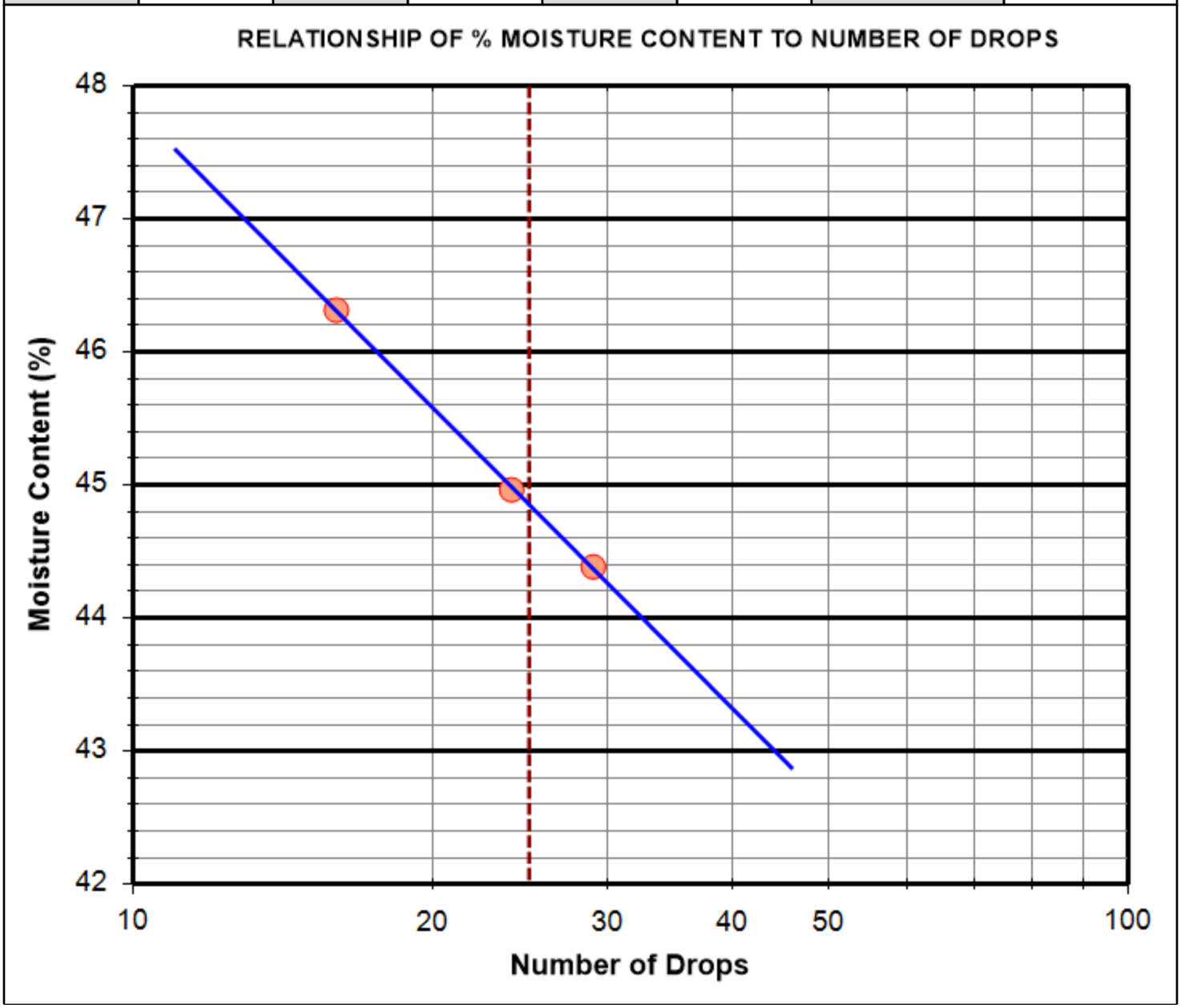
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/5/2019
Boring No.:	K-7	**Sample No.:**	SS-1	**Depth (ft.):**	0.0 - 1.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Dark brown moist very stiff LEAN CLAY, trace roots					**In Situ Moisture Content:**	22.3%
Liquid Limit:	45	**Plastic Limit:**	19	**Plastic Index:**	26	**USCS:**	CL



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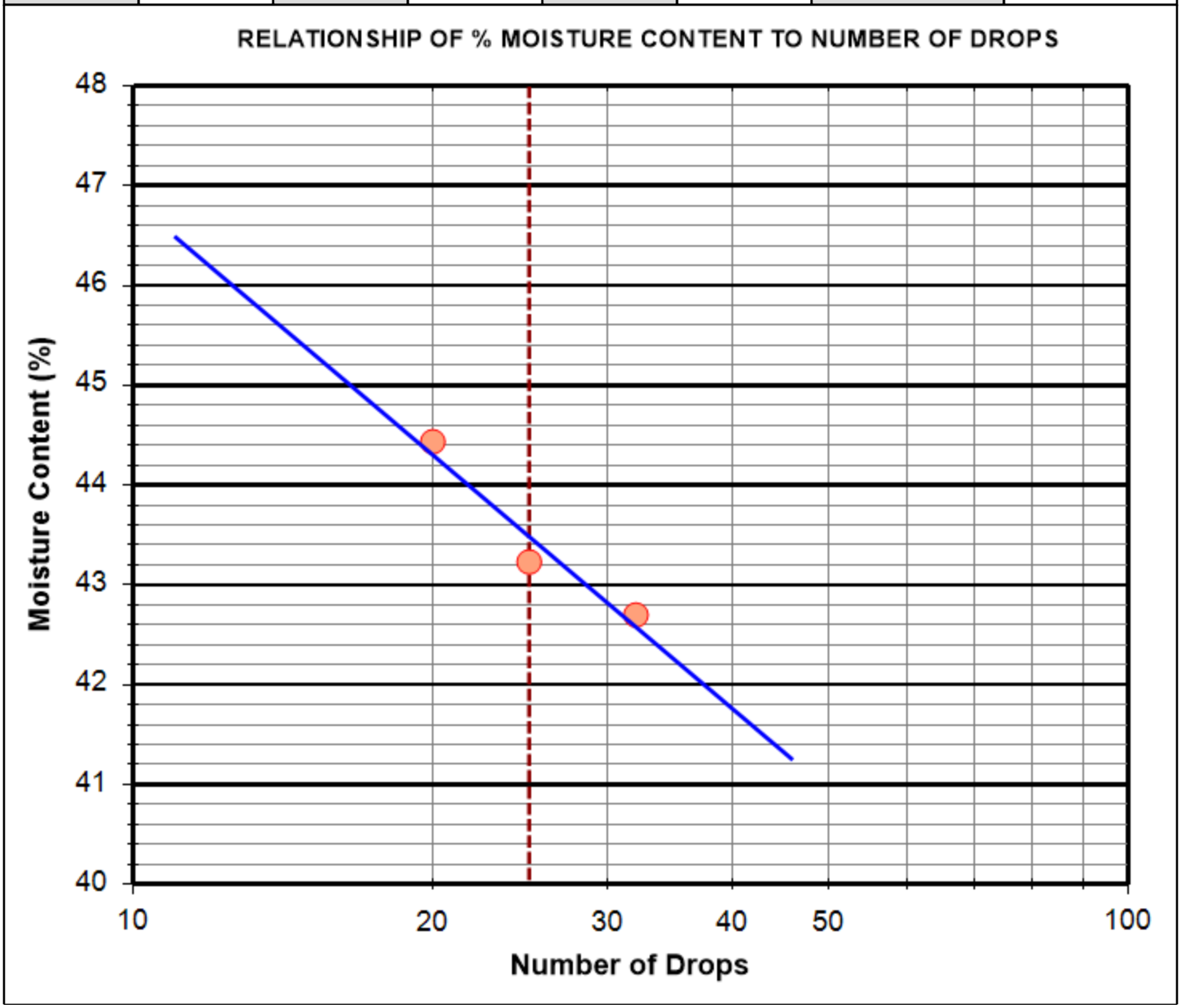
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-2	**Depth (ft.):**	2.5 - 4.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist very stiff LEAN CLAY with carbonate deposits					**In Situ Moisture Content:**	17.5%
Liquid Limit:	43	**Plastic Limit:**	20	**Plastic Index:**	23	**USCS:**	CL



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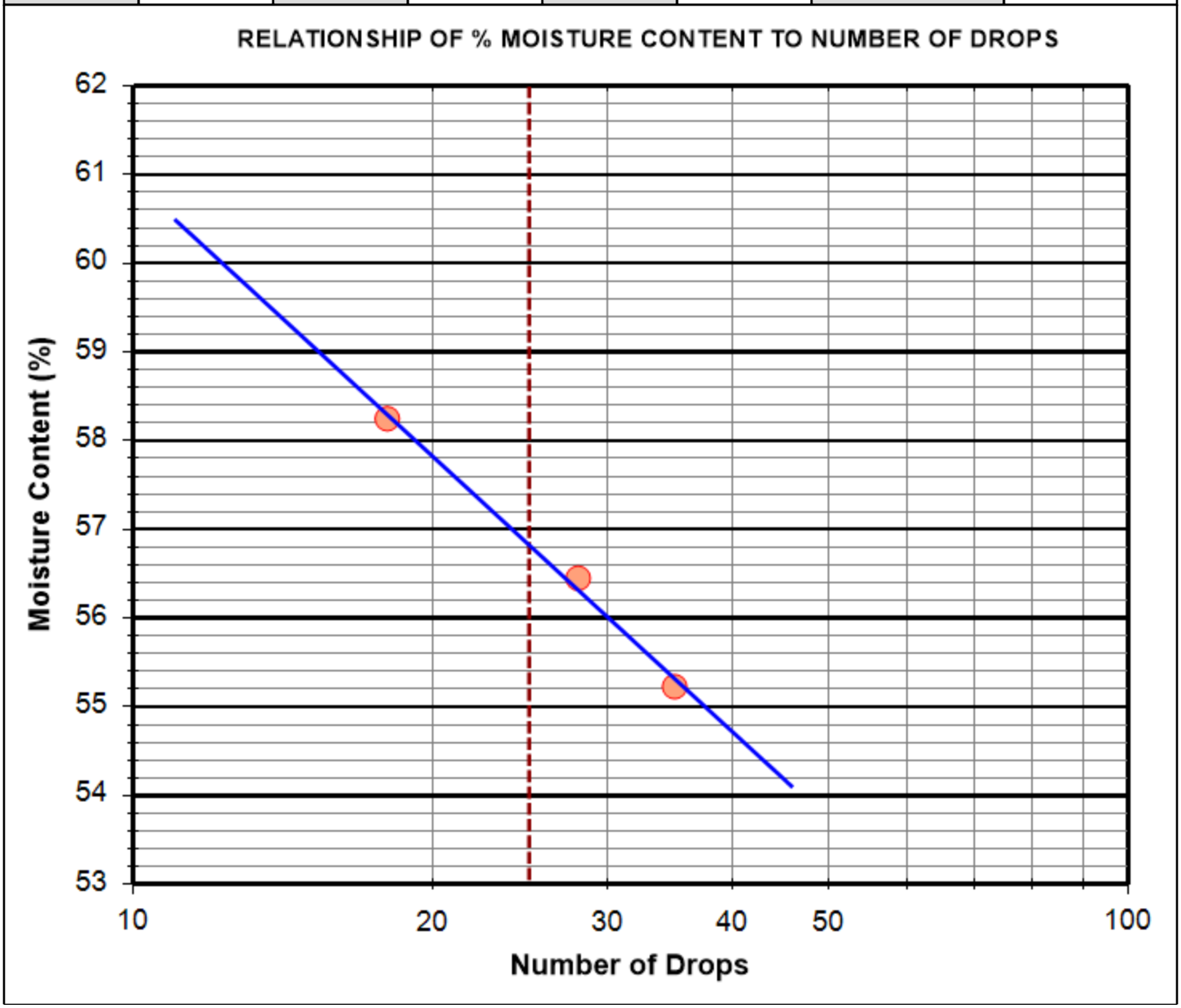
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-3	**Depth (ft.):**	5.0 - 6.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist very stiff FAT CLAY with carbonate deposits					**In Situ Moisture Content:**	23.2%
Liquid Limit:	57	**Plastic Limit:**	20	**Plastic Index:**	37	**USCS:**	CH



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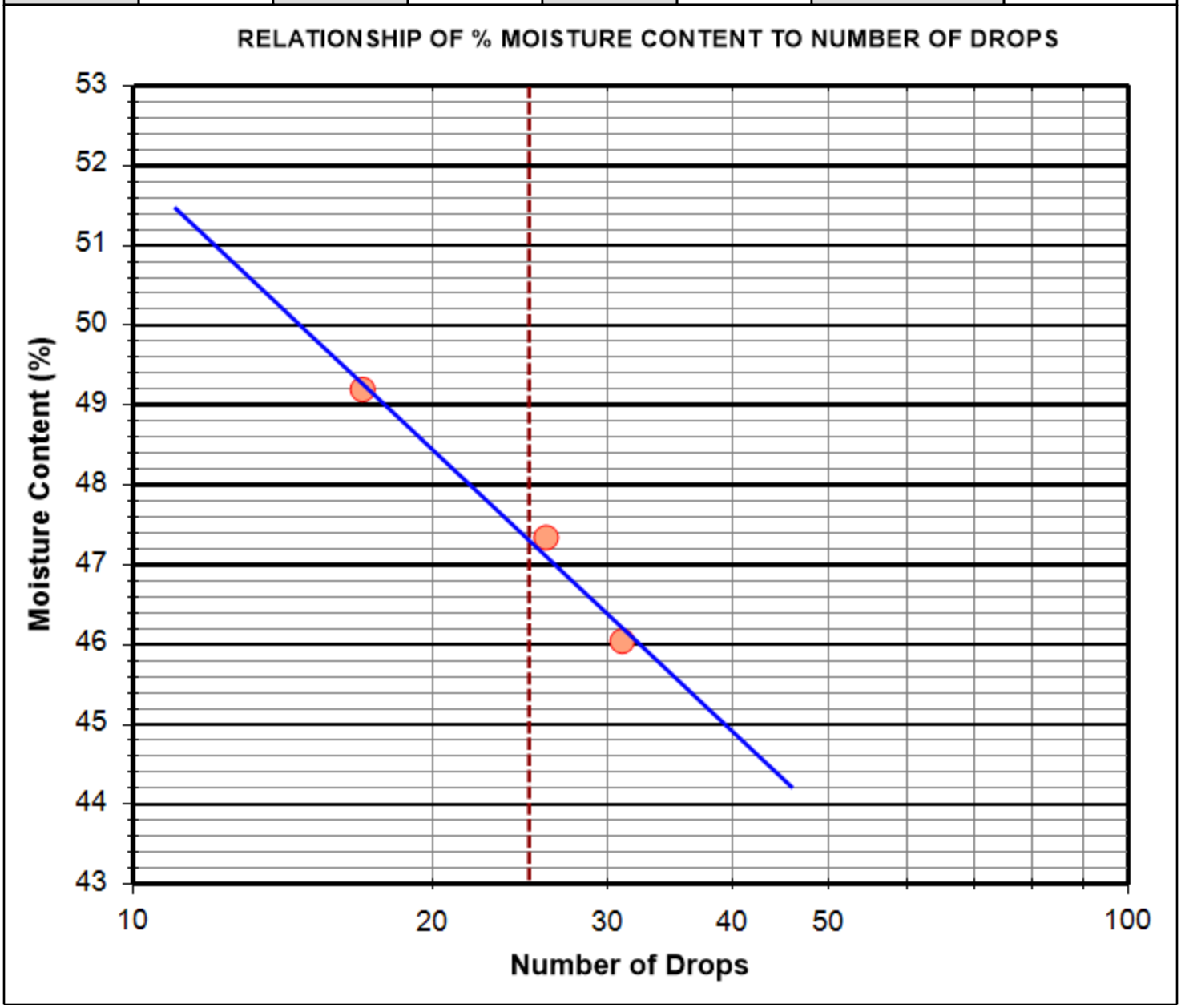
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-4	**Depth (ft.):**	7.5 - 9.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown very moist medium stiff LEAN CLAY					**In Situ Moisture Content:**	32.0%
Liquid Limit:	47	**Plastic Limit:**	19	**Plastic Index:**	28	**USCS:**	CL



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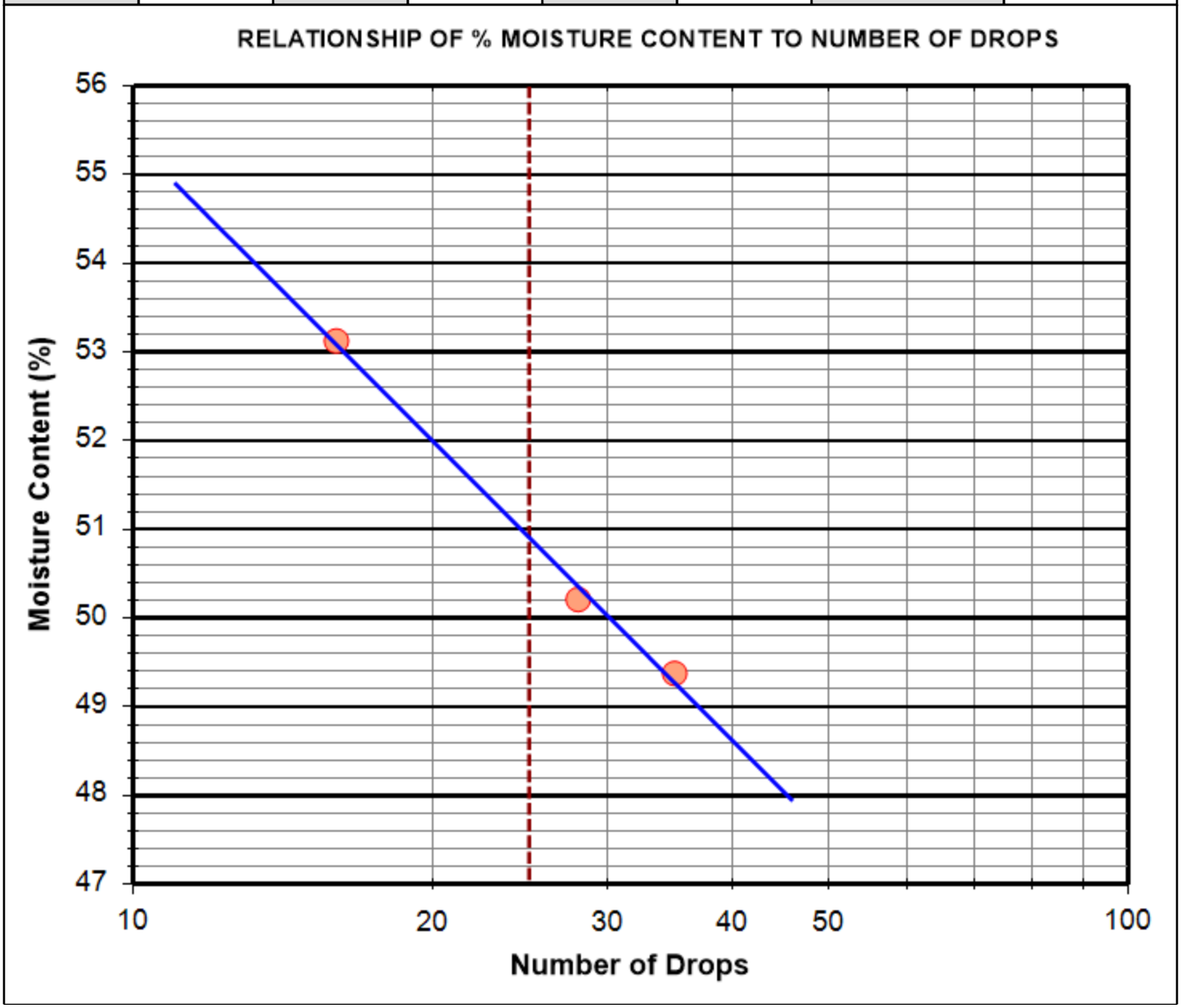
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-5	**Depth (ft.):**	10.0 - 11.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff FAT CLAY					**In Situ Moisture Content:**	29.8%
Liquid Limit:	51	**Plastic Limit:**	19	**Plastic Index:**	32	**USCS:**	CH



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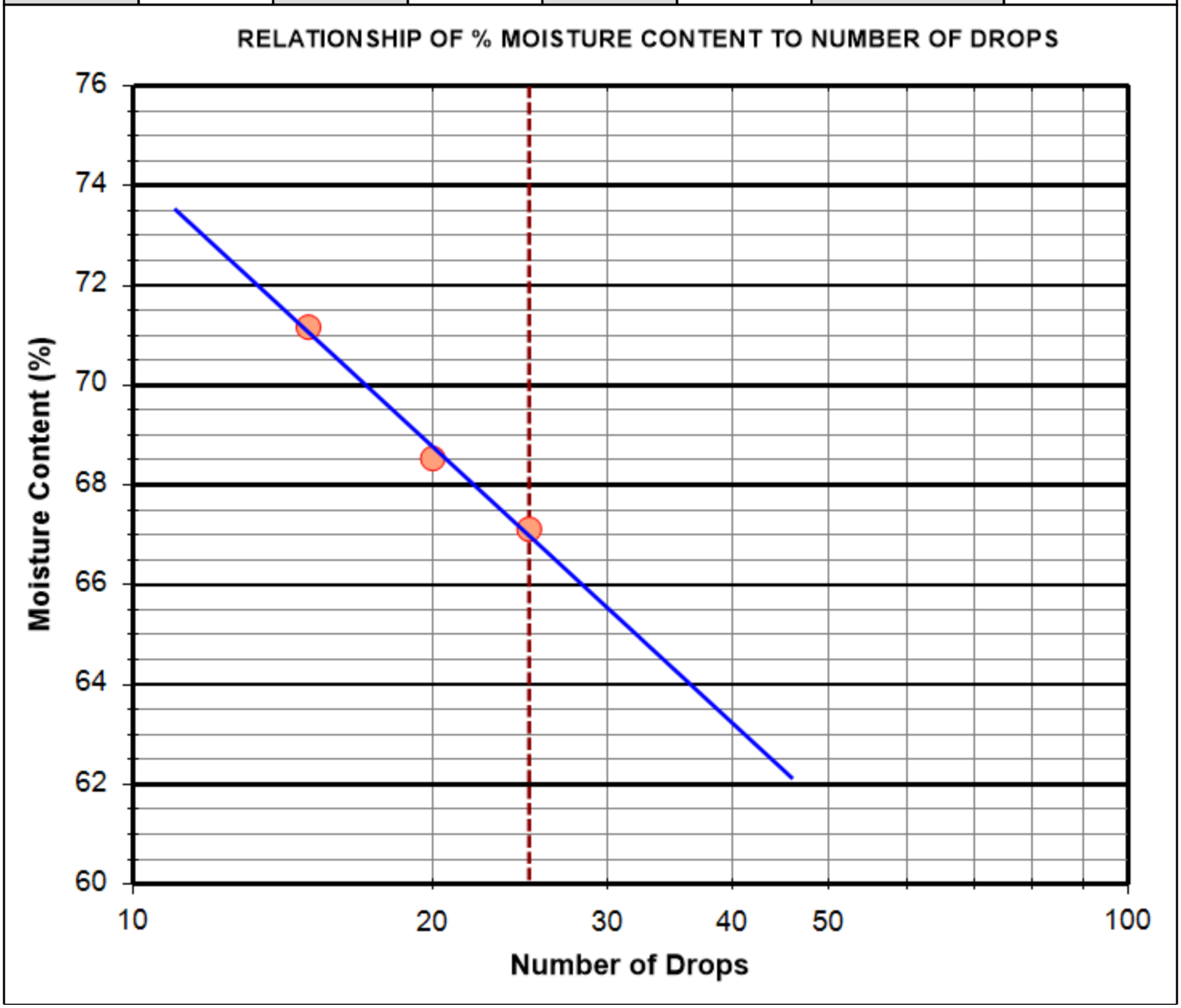
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-6	**Depth (ft.):**	12.5 - 14.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff FAT CLAY					**In Situ Moisture Content:**	32.6%
Liquid Limit:	67	**Plastic Limit:**	26	**Plastic Index:**	41	**USCS:**	CH



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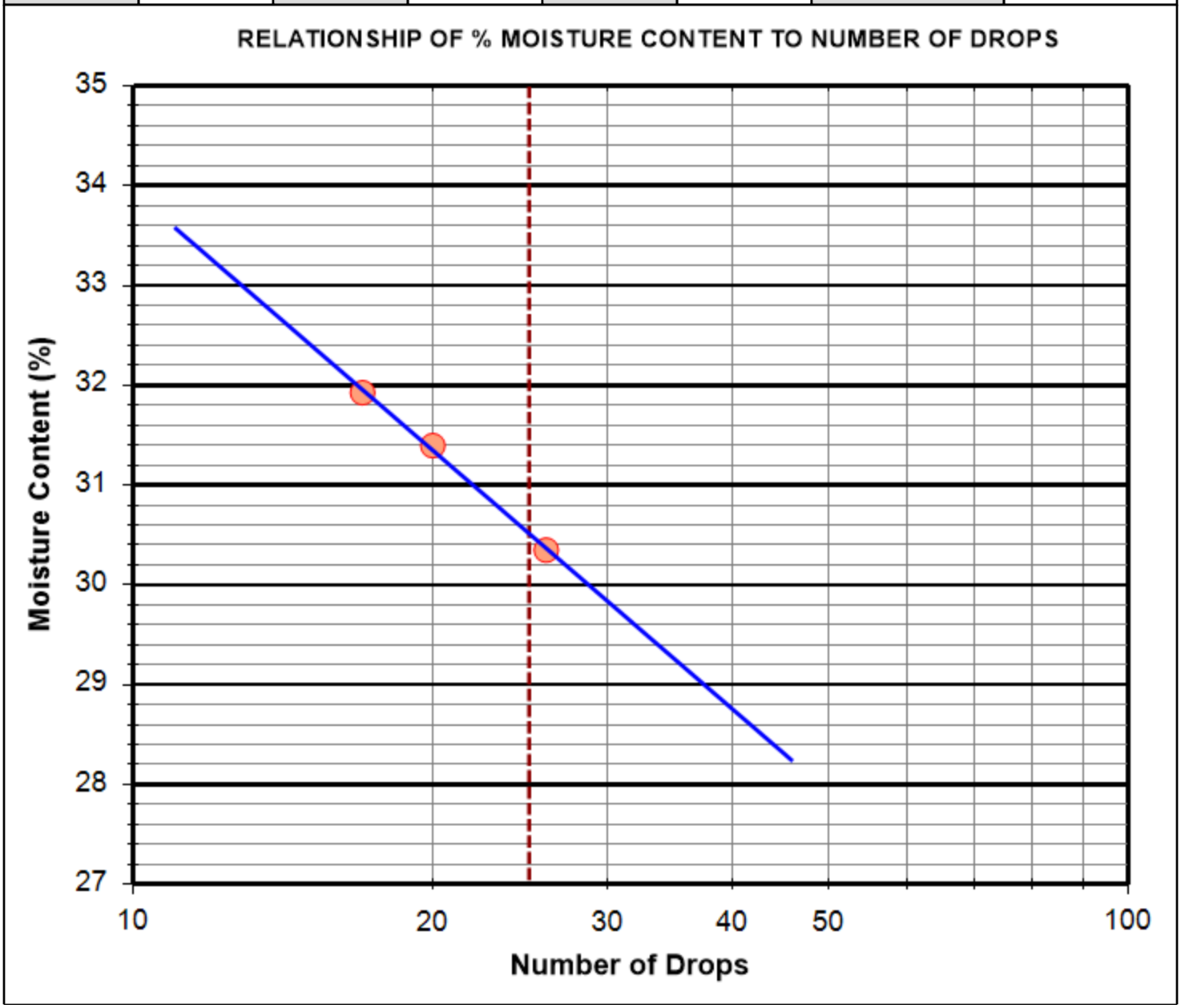
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-7	**Depth (ft.):**	15.0 - 16.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown wet very soft clayey SILT					**In Situ Moisture Content:**	33.2%
Liquid Limit:	31	**Plastic Limit:**	25	**Plastic Index:**	6	**USCS:**	ML



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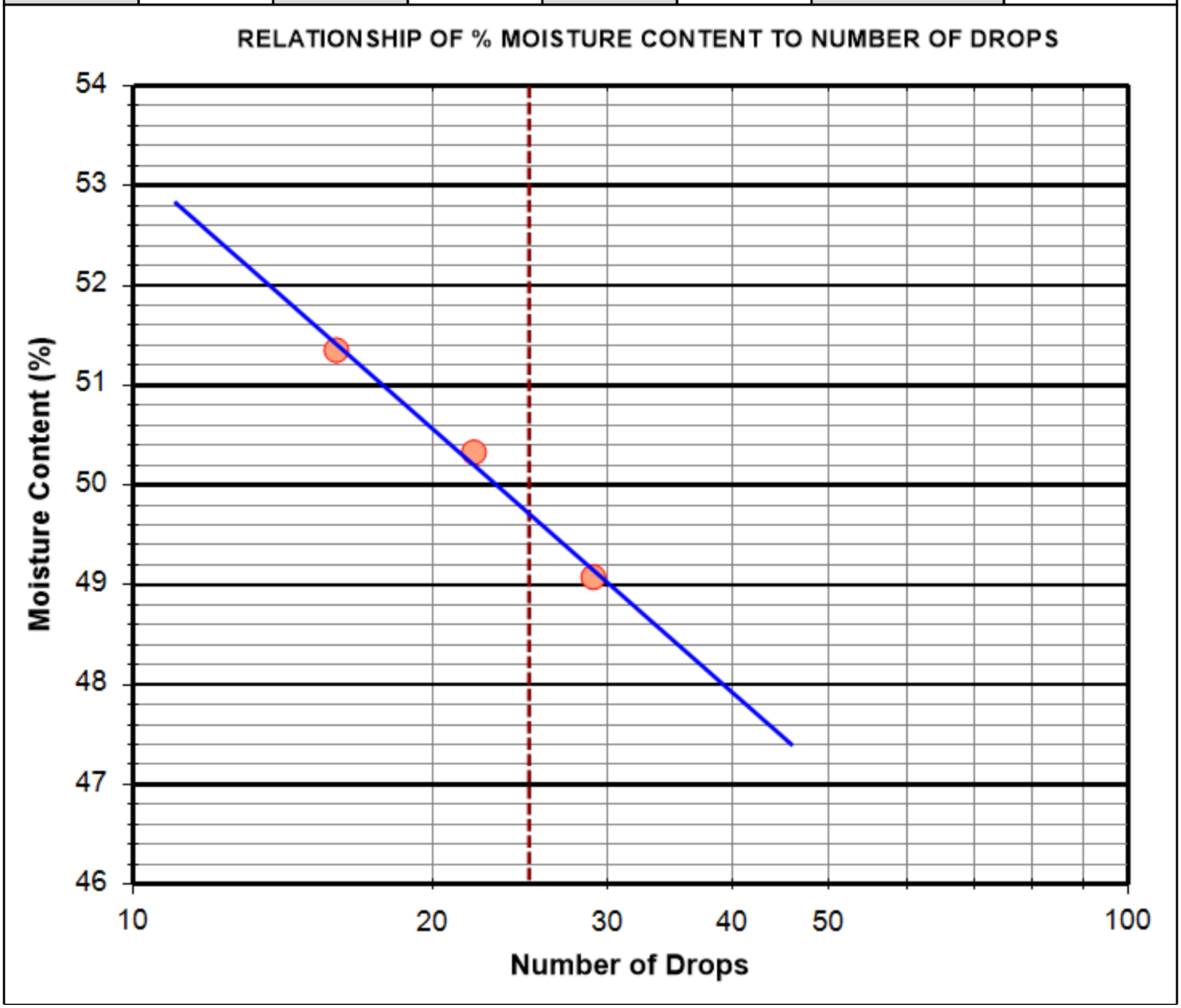
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-8	**Depth (ft.):**	18 - 19.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Gray and brown very moist stiff FAT CLAY					**In Situ Moisture Content:**	34.4%
Liquid Limit:	50	**Plastic Limit:**	20	**Plastic Index:**	30	**USCS:**	CH



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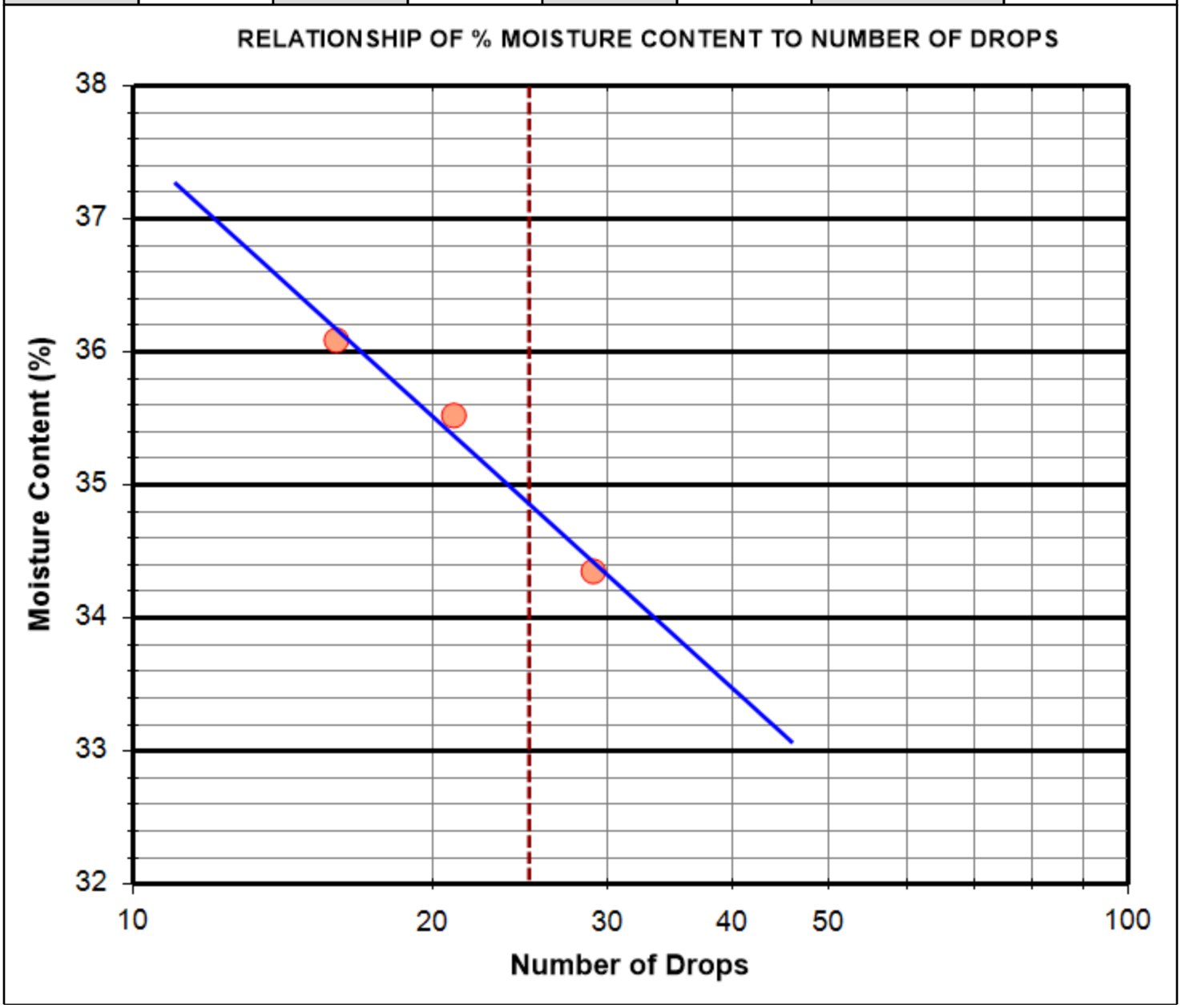
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/5/2019
Boring No.:	K-7	**Sample No.:**	SS-10	**Depth (ft.):**	25.0 - 26.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Gray and brown wet medium stiff LEAN CLAY with sand					**In Situ Moisture Content:**	27.0%
Liquid Limit:	35	**Plastic Limit:**	15	**Plastic Index:**	20	**USCS:**	CL



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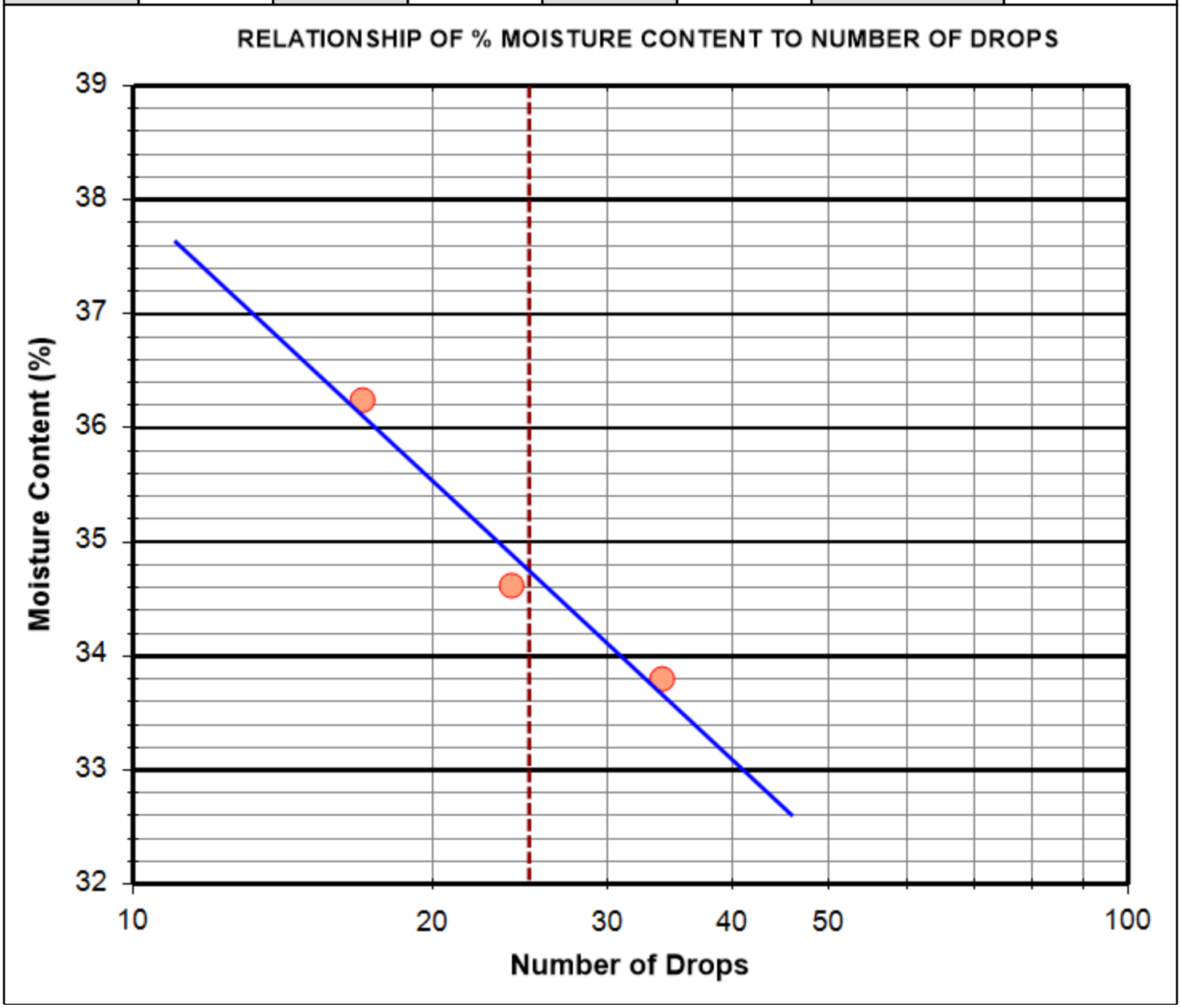
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	K-7	**Sample No.:**	SS-11	**Depth (ft.):**	30.0 - 31.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown and gray wet medium stiff LEAN CLAY with a little fine sand					**In Situ Moisture Content:**	28.5%
Liquid Limit:	35	**Plastic Limit:**	23	**Plastic Index:**	12	**USCS:**	CL



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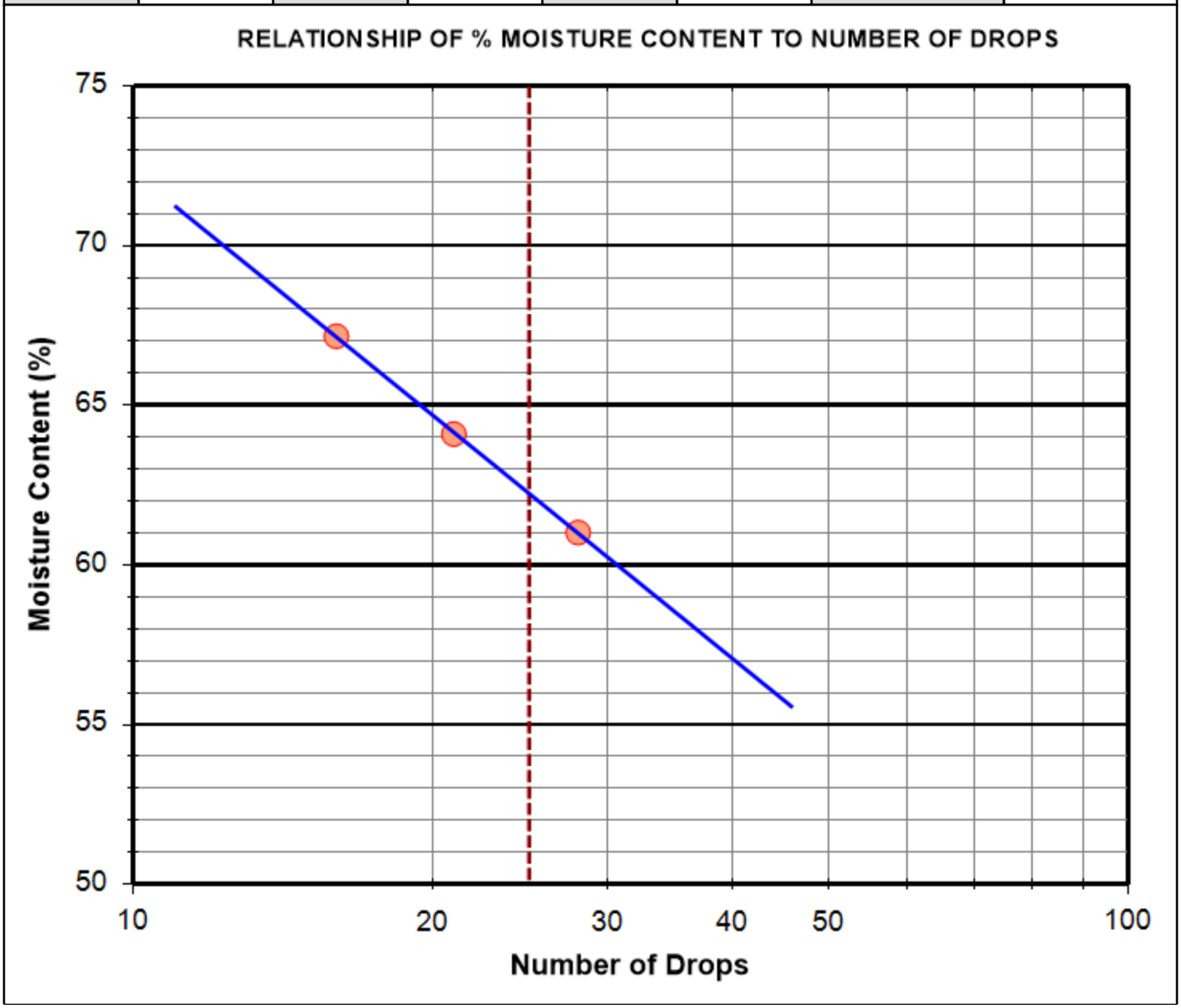
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	W-1	**Sample No.:**	Bucket	**Depth (ft.):**	0.0 - 2.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Dark brown moist stiff FAT CLAY, trace roots					**In Situ Moisture Content:**	25.4%
Liquid Limit:	62	**Plastic Limit:**	20	**Plastic Index:**	42	**USCS:**	CH



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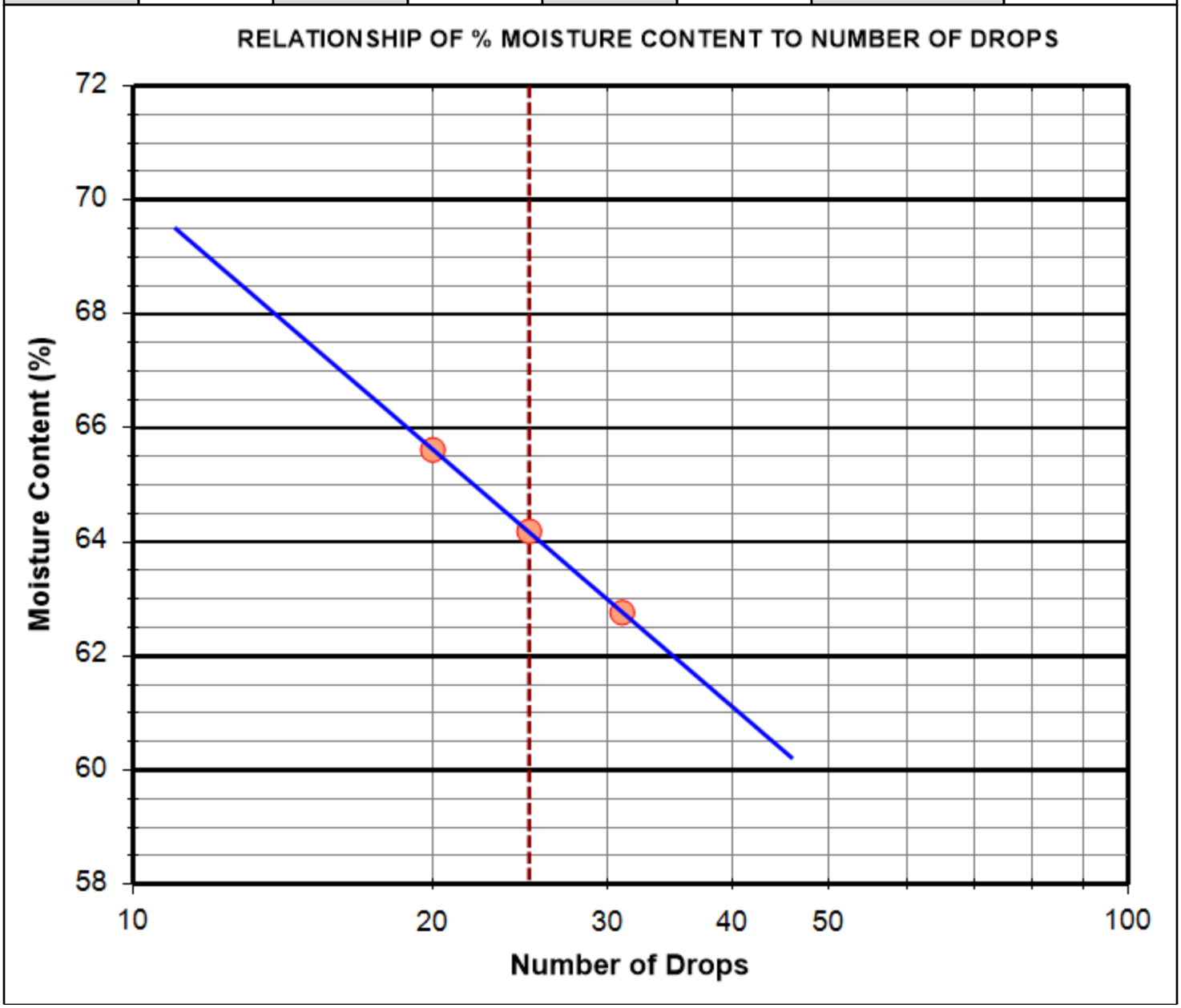
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/23/2019
Boring No.:	W-1	**Sample No.:**	PT-7	**Depth (ft.):**	15.0 - 17.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff FAT CLAY					**In Situ Moisture Content:**	31.5%
Liquid Limit:	64	**Plastic Limit:**	22	**Plastic Index:**	42	**USCS:**	CH



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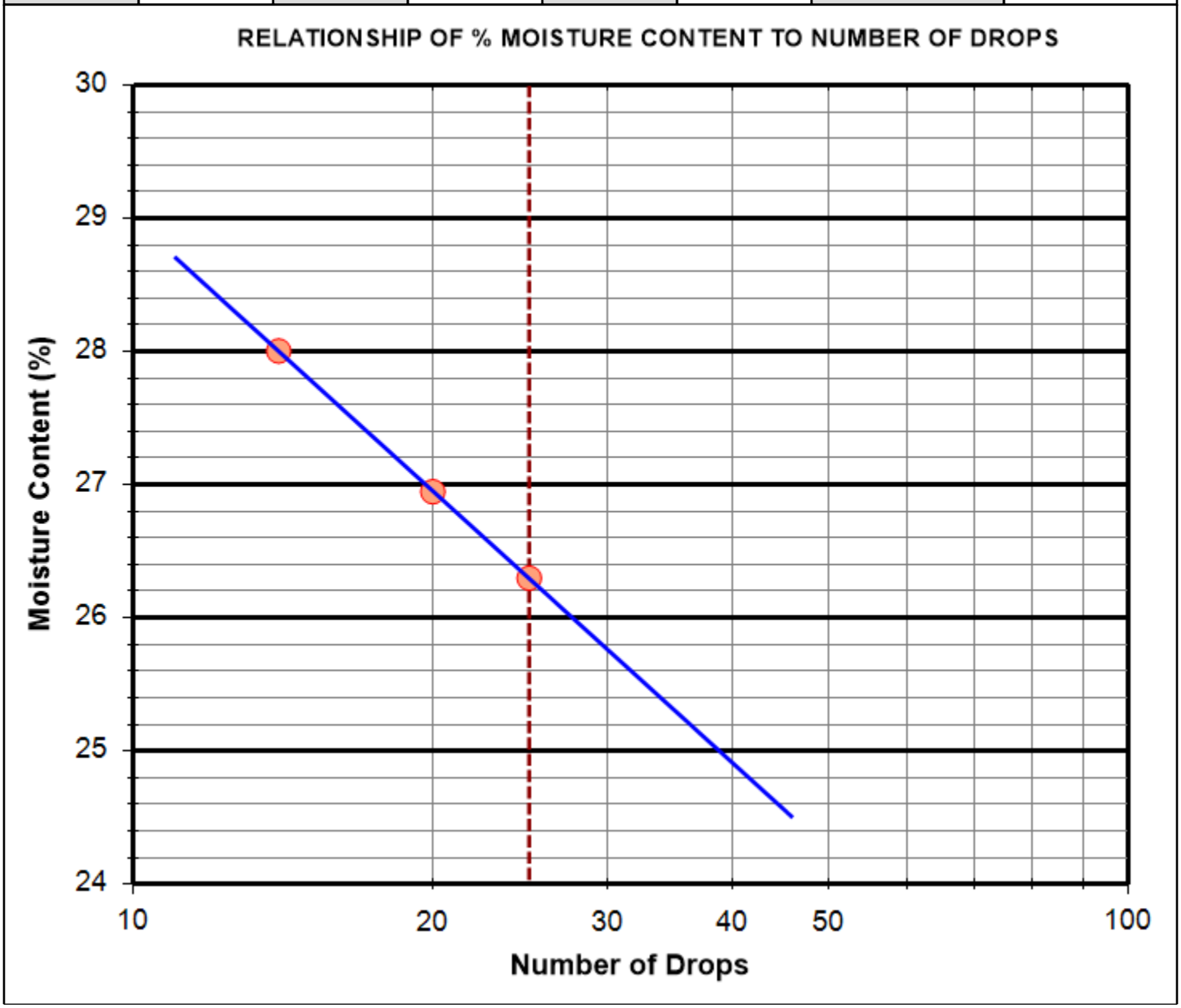
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/6/2019
Boring No.:	W-1	**Sample No.:**	SS-10B & SS-11A	**Depth (ft.):**	26.0 - 30.5	**Sample Preparation Condition:**	Air Dried

Sample Description:	Brown and gray wet very loose sandy SILT					**In Situ Moisture Content:**	32.4%
Liquid Limit:	26	**Plastic Limit:**	24	**Plastic Index:**	2	**USCS:**	ML



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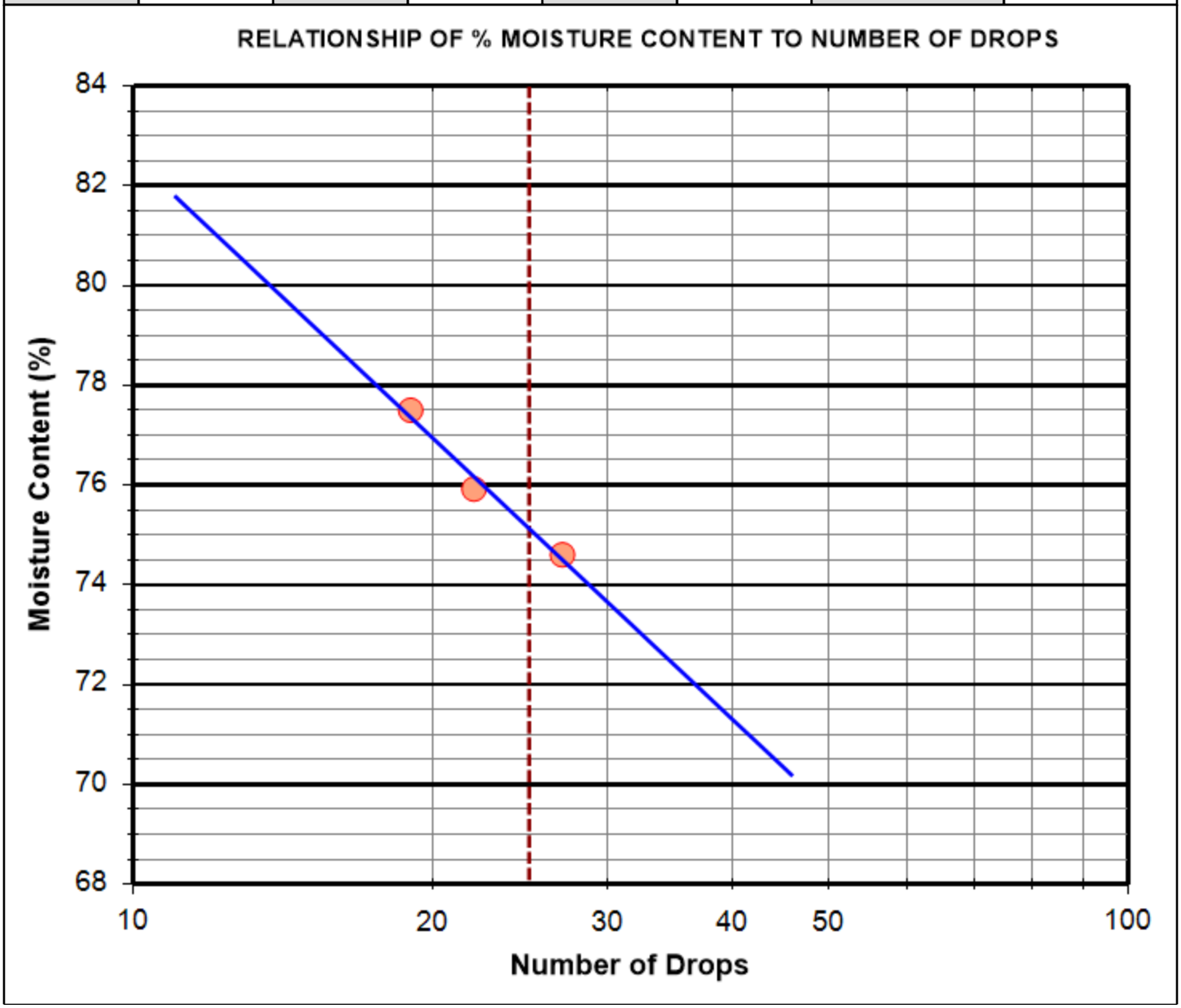
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	1/23/2019
Boring No.:	W-2	**Sample No.:**	PT-4A	**Depth (ft.):**	7.5 - 8.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown and gray moist medium stiff FAT CLAY, trace gravel					**In Situ Moisture Content:**	32.3%
Liquid Limit:	75	**Plastic Limit:**	26	**Plastic Index:**	49	**USCS:**	CH



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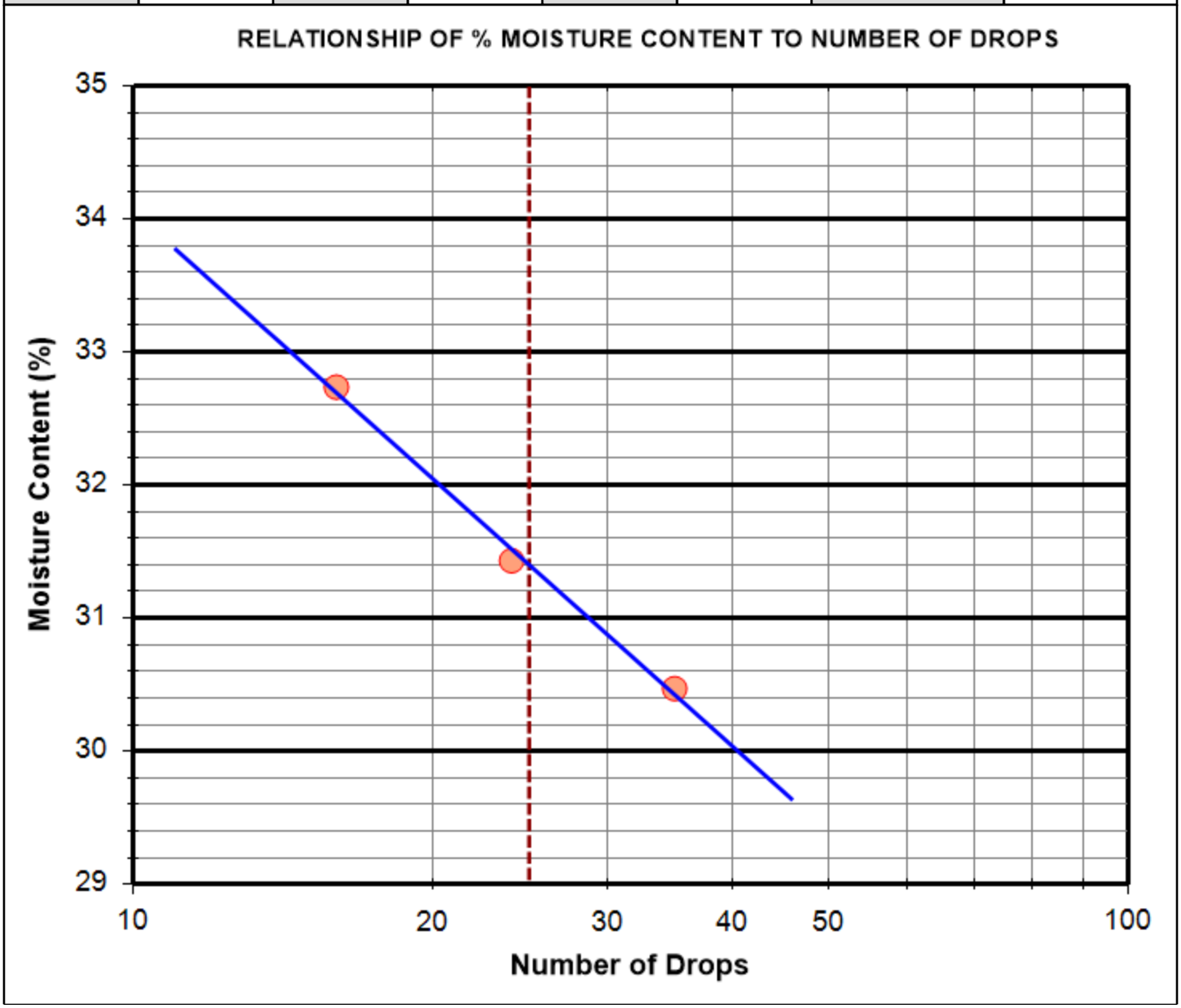
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/6/2019
Boring No.:	W-2	**Sample No.:**	SS-5	**Depth (ft.):**	10.0 - 11.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown wet very soft clayey SILT					**In Situ Moisture Content:**	31.0%
Liquid Limit:	31	**Plastic Limit:**	23	**Plastic Index:**	8	**USCS:**	ML



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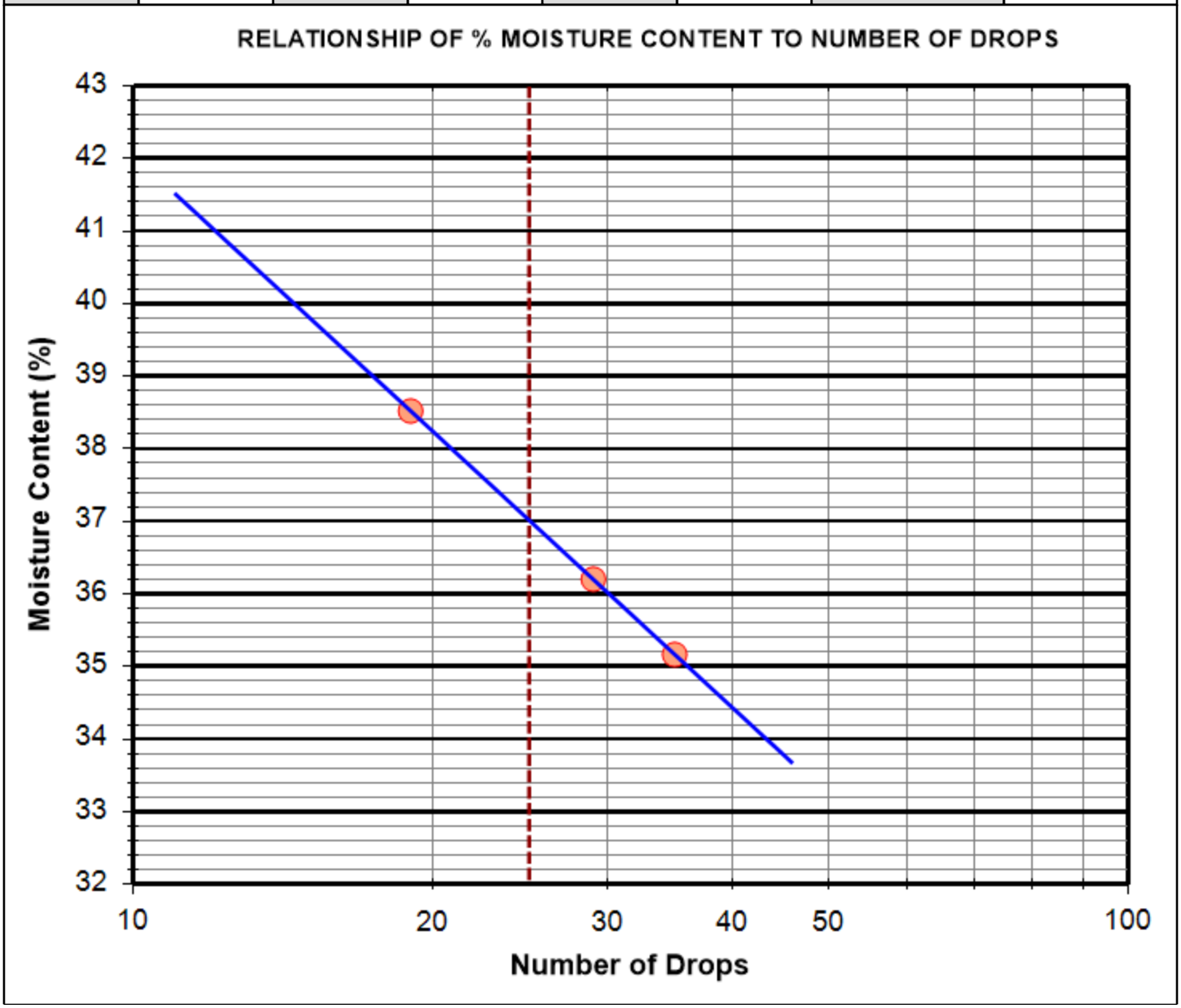
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/6/2019
Boring No.:	W-2	**Sample No.:**	SS-9	**Depth (ft.):**	20.0 - 21.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Gray very moist medium stiff LEAN CLAY with sand					**In Situ Moisture Content:**	27.0%
Liquid Limit:	37	**Plastic Limit:**	16	**Plastic Index:**	21	**USCS:**	CL



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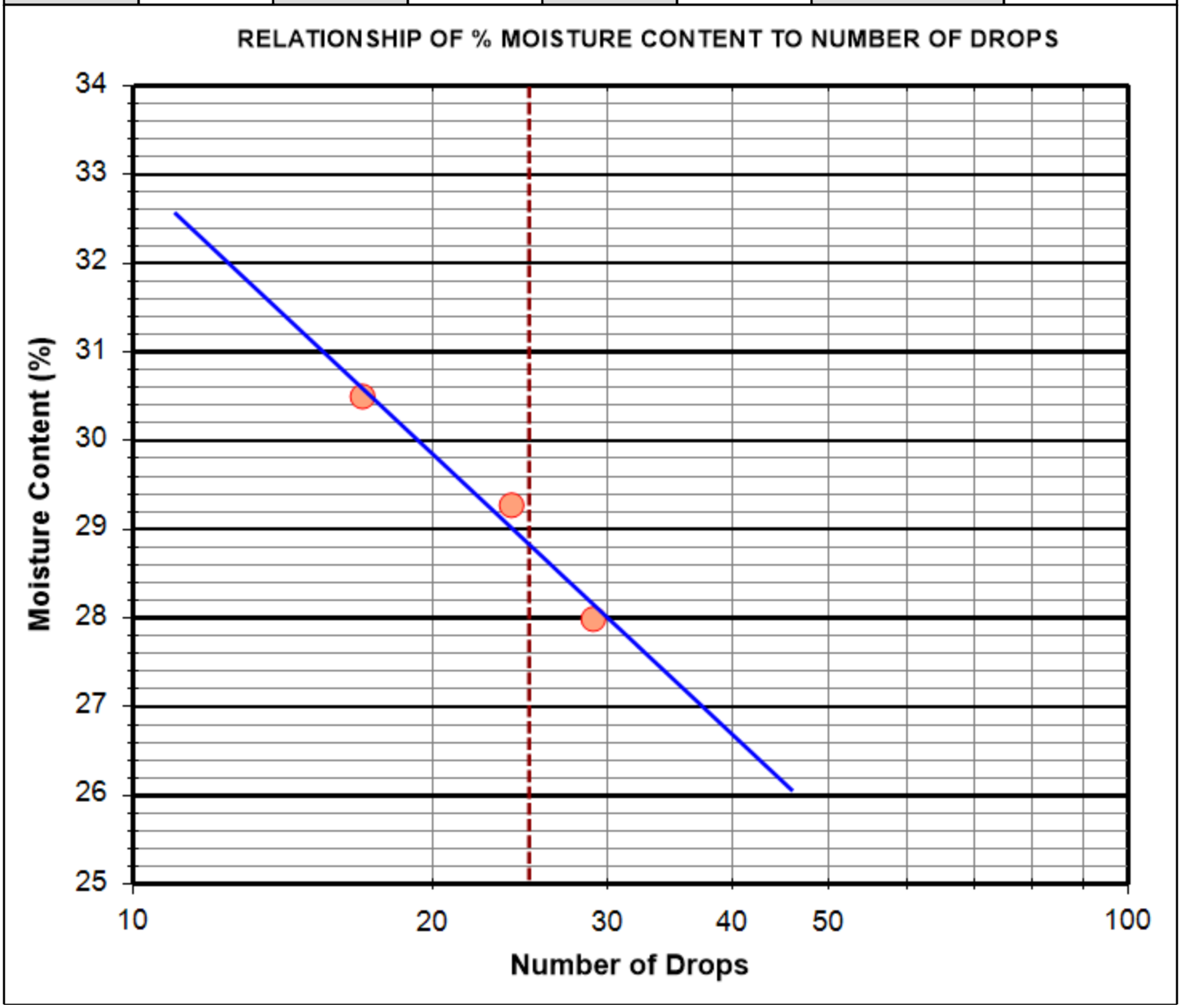
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/5/2019
Boring No.:	W-3	**Sample No.:**	SS-9	**Depth (ft.):**	20.0 - 21.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Gray, trace brown wet medium stiff sandy LEAN CLAY					**In Situ Moisture Content:**	27.5%
Liquid Limit:	29	**Plastic Limit:**	18	**Plastic Index:**	11	**USCS:**	CL



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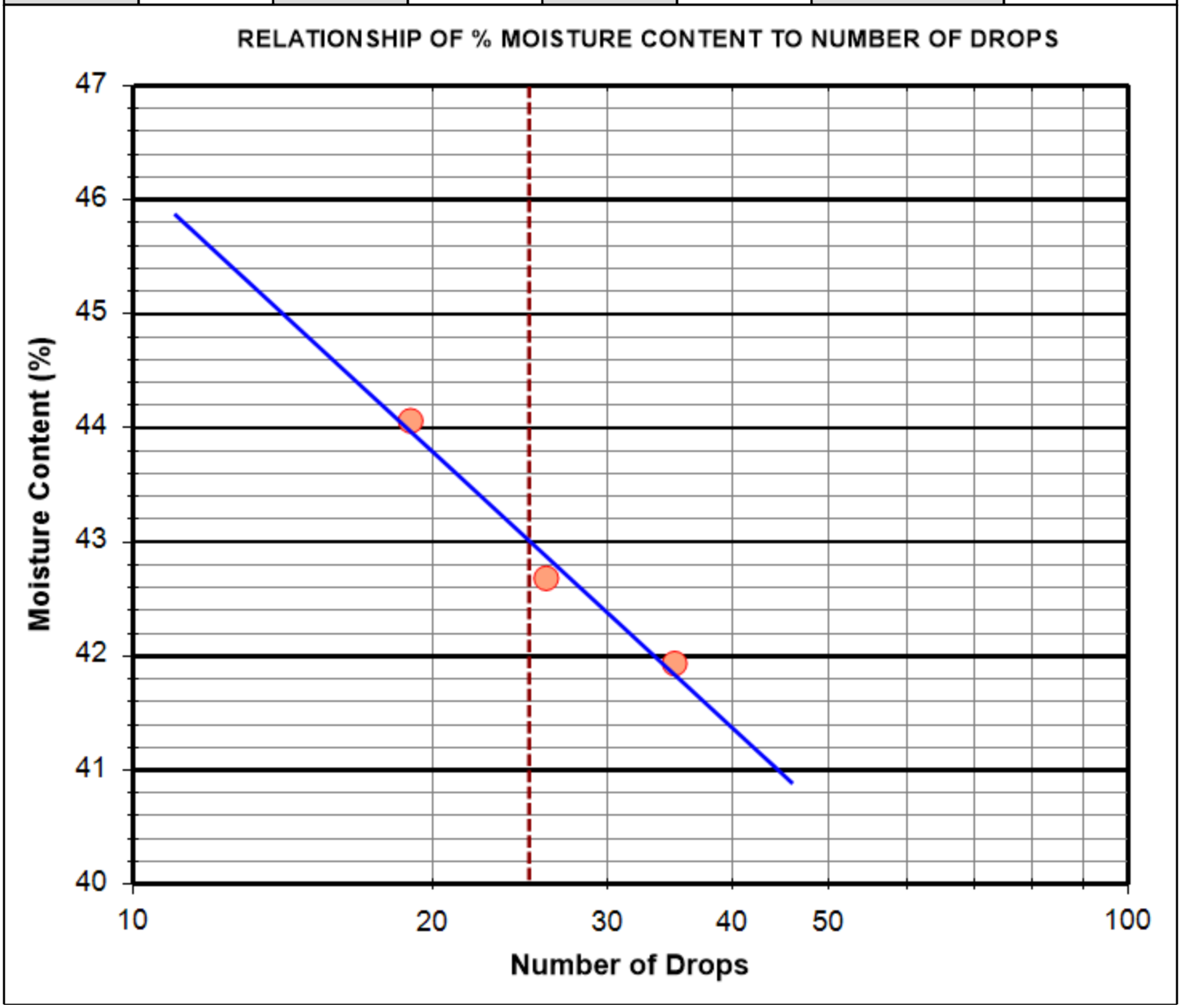
Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/4/2019
Boring No.:	W-4	**Sample No.:**	Bucket	**Depth (ft.):**	0.0 - 5.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff to very stiff LEAN CLAY with carbonate deposits, trace roots					**In Situ Moisture Content:**	18.2%
Liquid Limit:	43	**Plastic Limit:**	18	**Plastic Index:**	25	**USCS:**	CL



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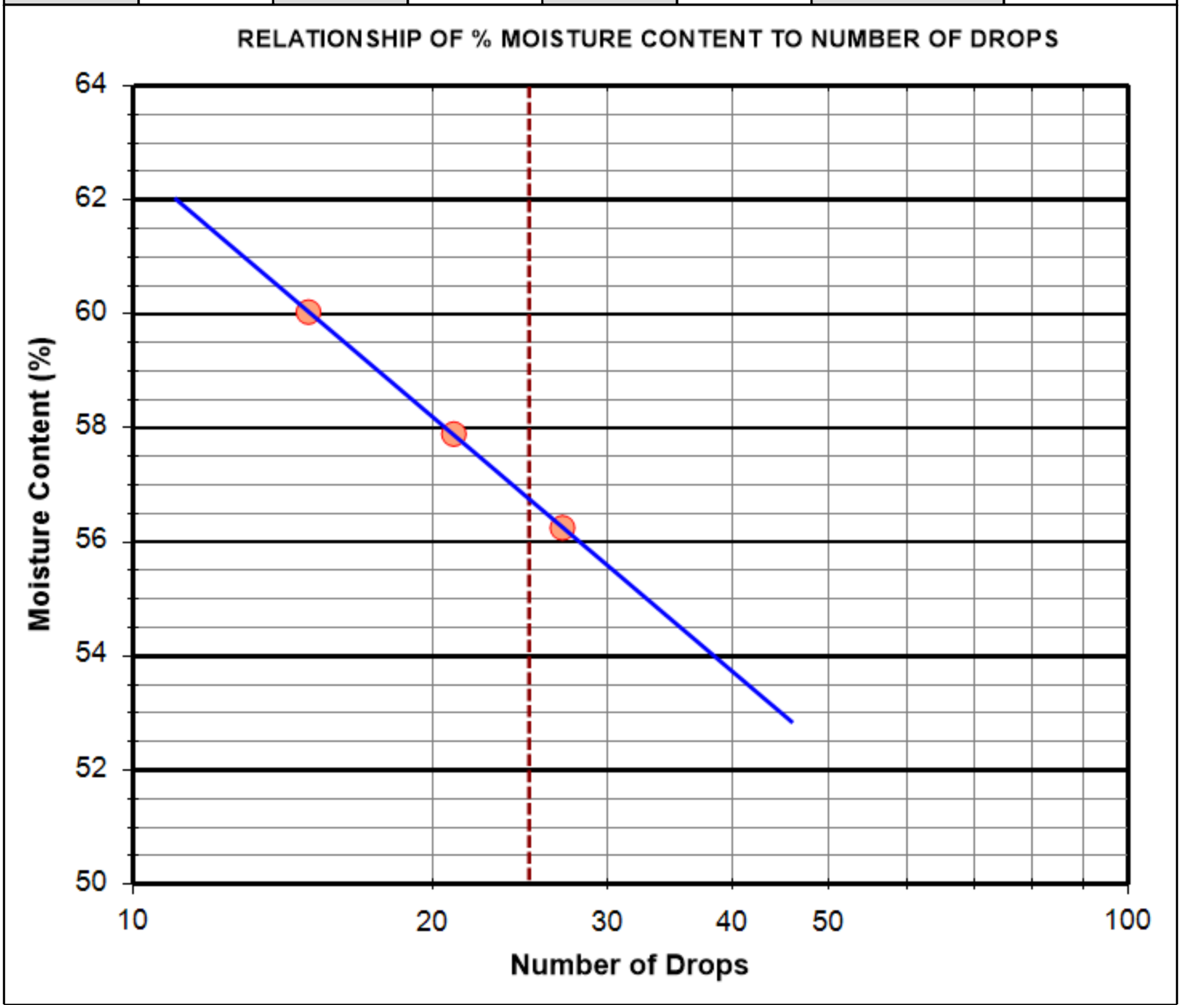
GEOTECHNOLOGY INC
FROM THE GROUND UP

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/6/2019
Boring No.:	W-4	**Sample No.:**	SS-5	**Depth (ft.):**	10.0 - 11.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown moist stiff FAT CLAY, trace sand					**In Situ Moisture Content:**	28.8%
Liquid Limit:	57	**Plastic Limit:**	24	**Plastic Index:**	33	**USCS:**	CH



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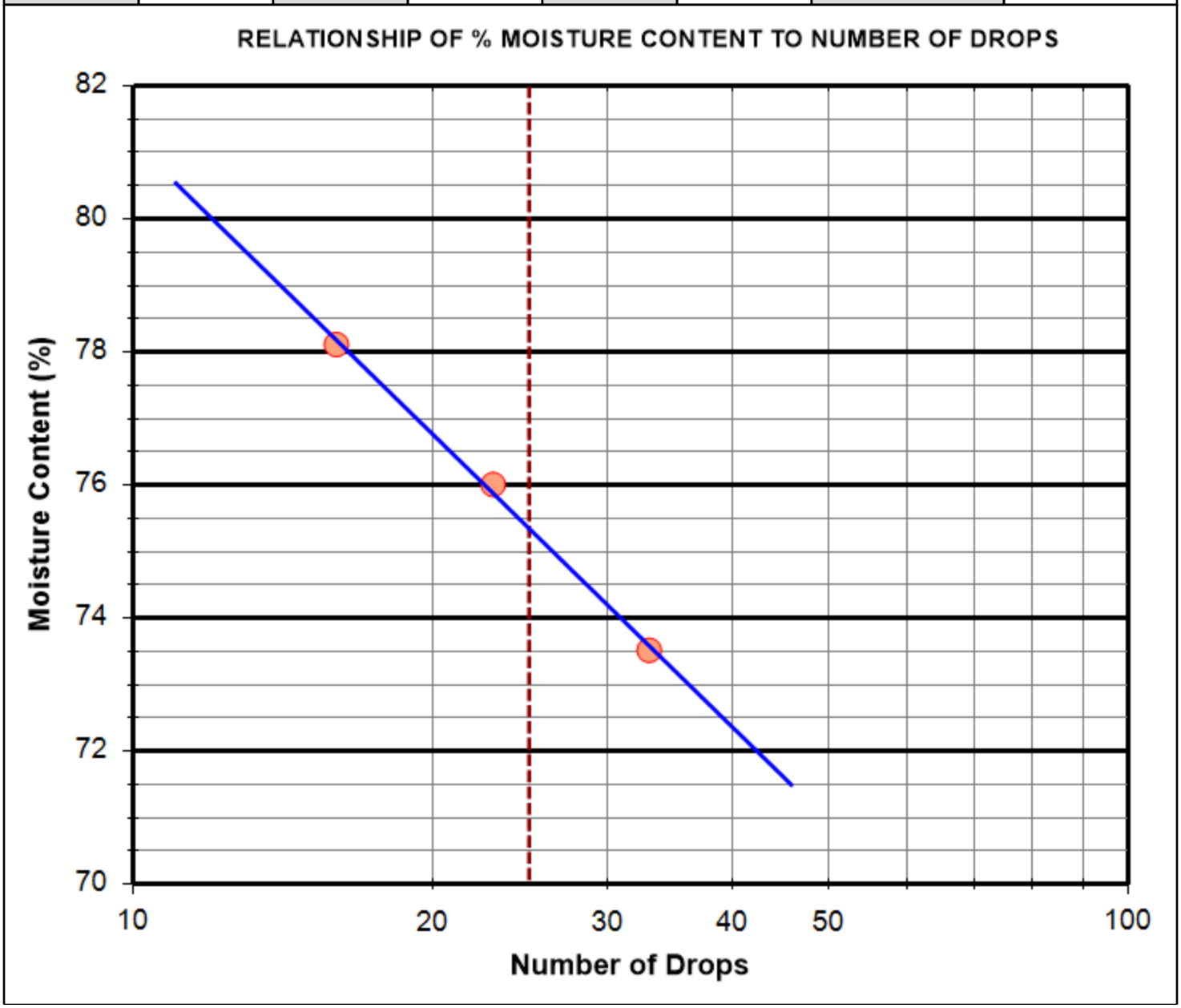
GEOTECHNOLOGY INC
FROM THE GROUND UP

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/6/2019
Boring No.:	W-4	**Sample No.:**	SS-6	**Depth (ft.):**	12.5 - 14.0	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown, trace black moist stiff to very stiff FAT CLAY					**In Situ Moisture Content:**	32.3%
Liquid Limit:	75	**Plastic Limit:**	28	**Plastic Index:**	47	**USCS:**	CH



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GEOTECHNOLOGY INC
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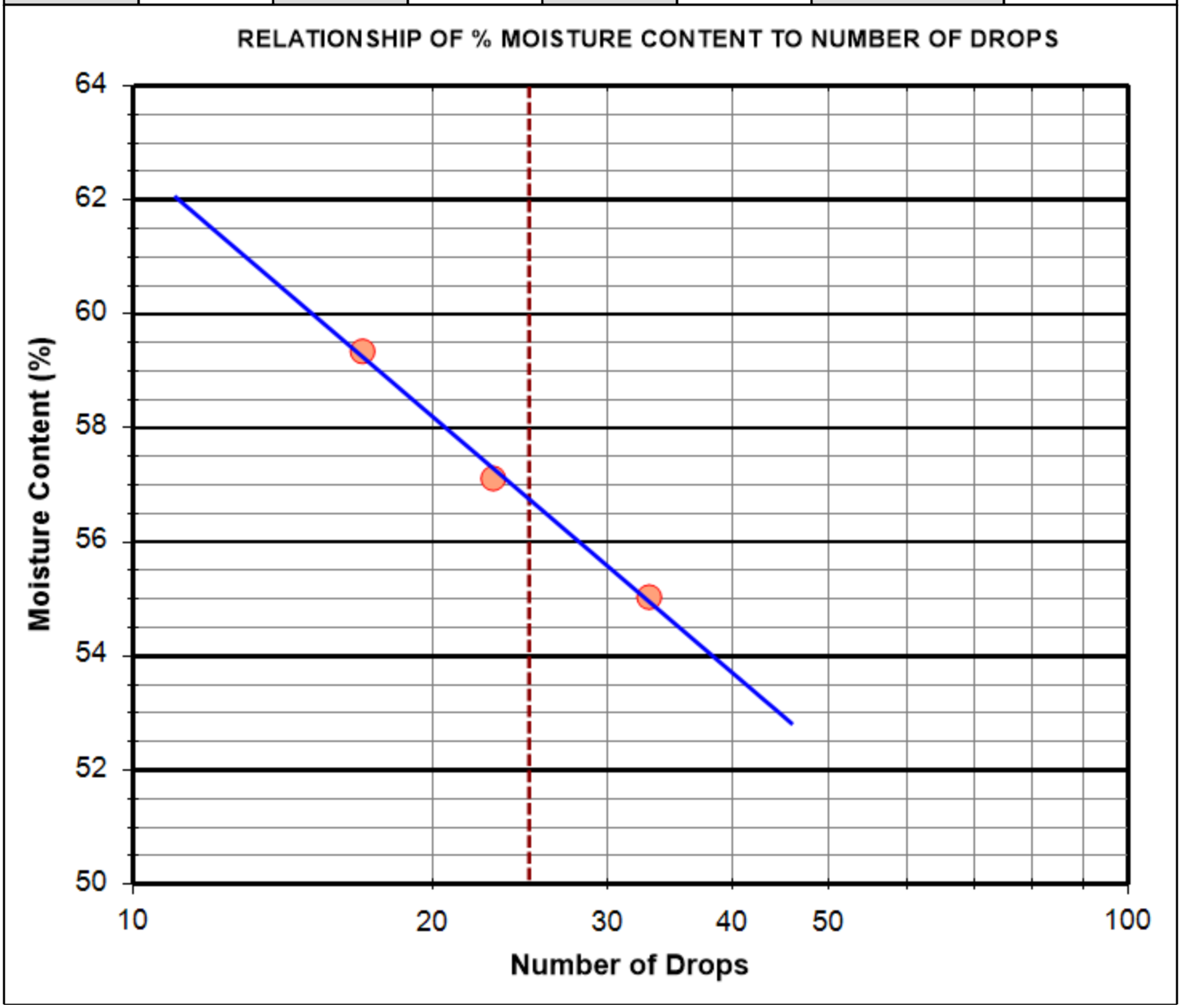
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/6/2019
Boring No.:	W-4	**Sample No.:**	SS-7	**Depth (ft.):**	15.0 - 26.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown, trace gray moist medium stiff FAT CLAY					**In Situ Moisture Content:**	30.5%
Liquid Limit:	57	**Plastic Limit:**	24	**Plastic Index:**	33	**USCS:**	CH



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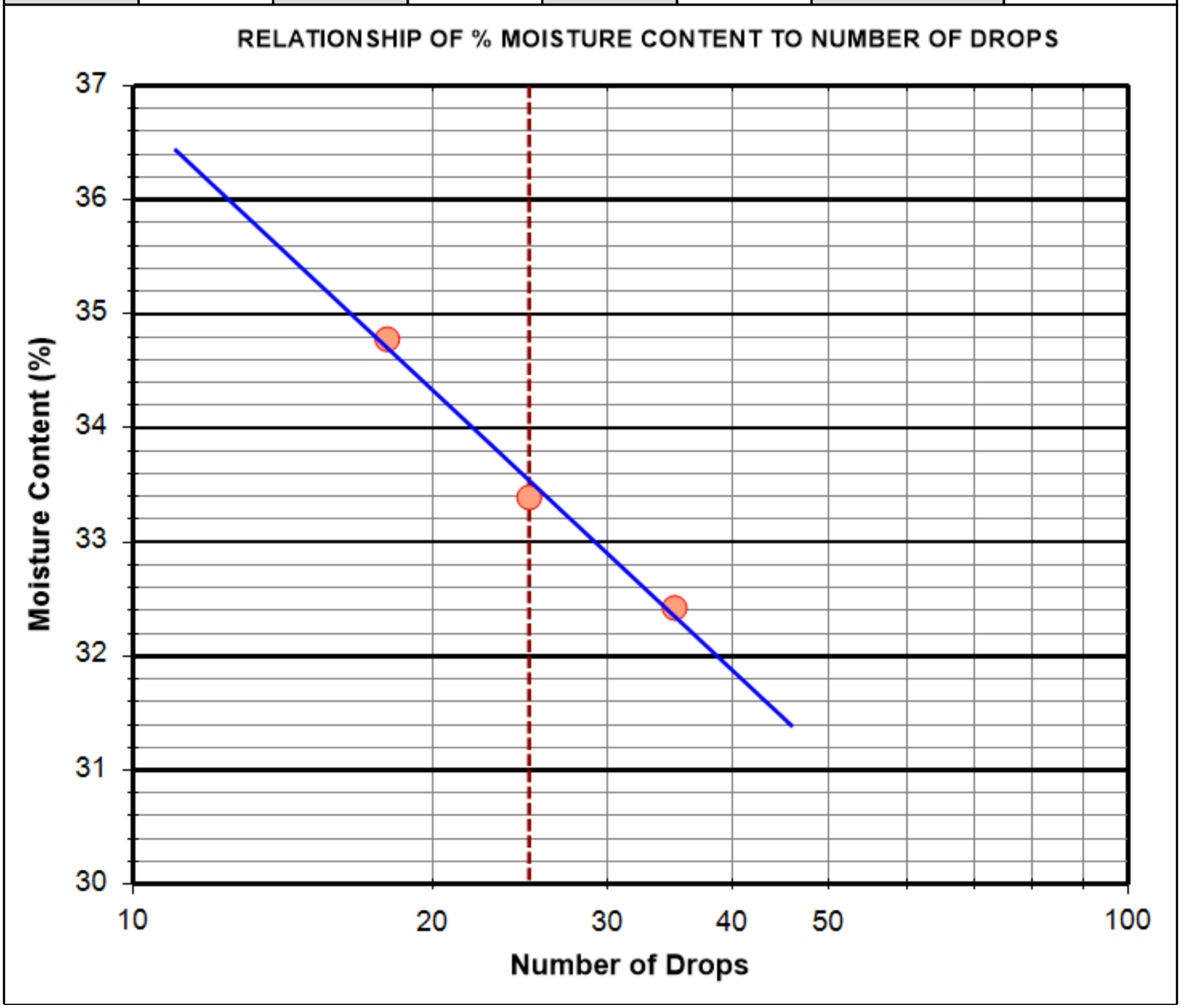
GEOTECHNOLOGY INC
FROM THE GROUND UP

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D-4318

Client:	Etegra, Inc.					**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX					**Date:**	2/5/2019
Boring No.:	W-4	**Sample No.:**	SS-8A	**Depth (ft.):**	17.5 - 18.5	**Sample Preparation Condition:**	Air Dried
Sample Description:	Brown wet very soft clayey SILT					**In Situ Moisture Content:**	35.0%
Liquid Limit:	34	**Plastic Limit:**	25	**Plastic Index:**	9	**USCS:**	ML



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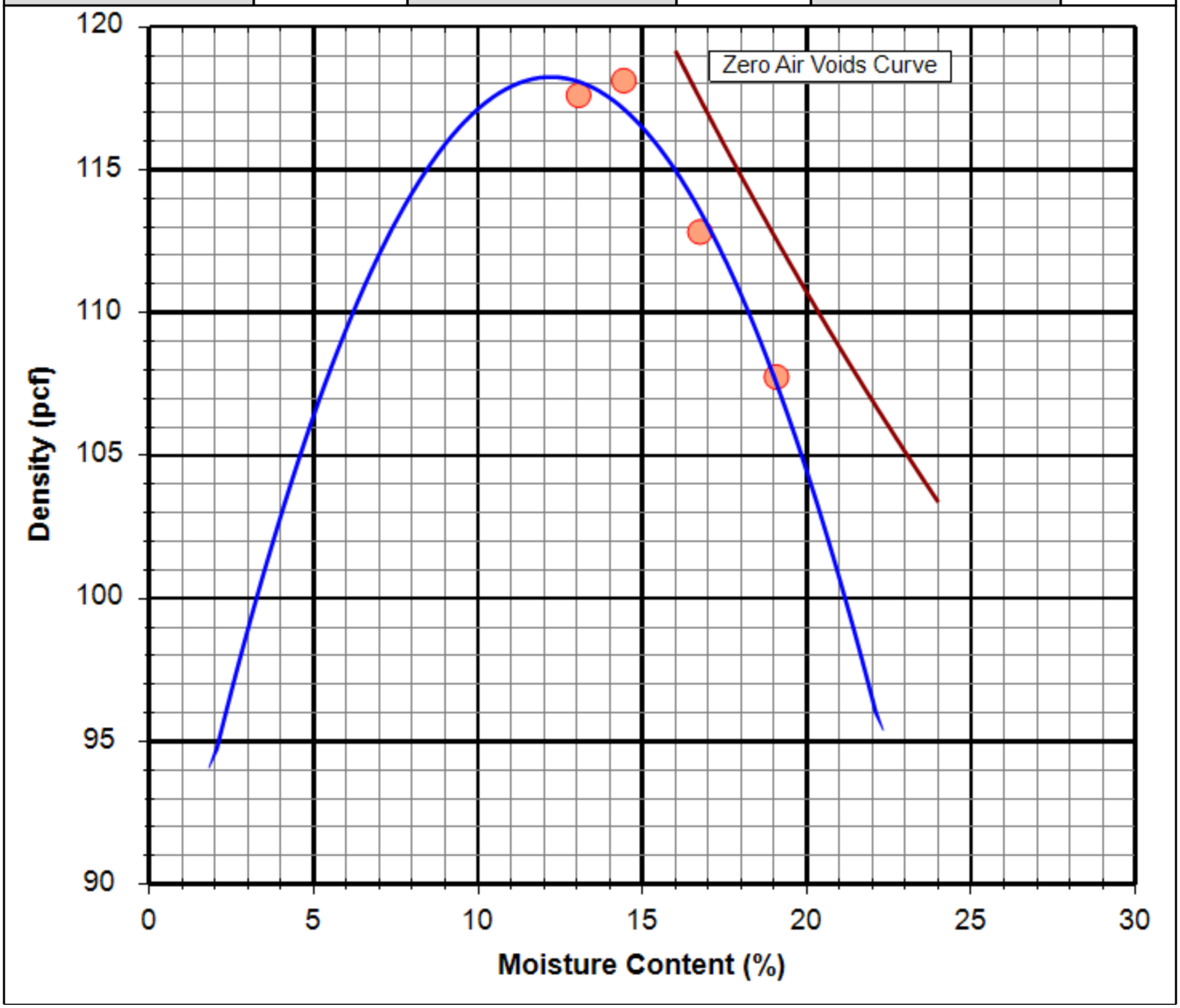
GEOTECHNOLOGY INC
FROM THE GROUND UP

MODIFIED PROCTOR MOISTURE DENSITY TEST, ASTM D1557, METHOD A

Client:	Etegra, Inc.	**Project No.:**	J033011.01
Project:	Port Isabel Texas Kitchen and WWTP Facilities, Port Isabel, TX	**Date:**	2/4/2019
Sample Obtained From:	Boring K-4	**Depth (ft.):**	0.0 - 5.0

Sample Description:	**LL**	**PL**	**PI**	**USCS**
Brown to dark brown slightly moist very stiff lean CLAY	38	18	20	CL

Maximum Dry Density (pcf):	118.2	**Optimum Moisture Content:**	12.3%	**In Situ Moisture Content:**	11.9%



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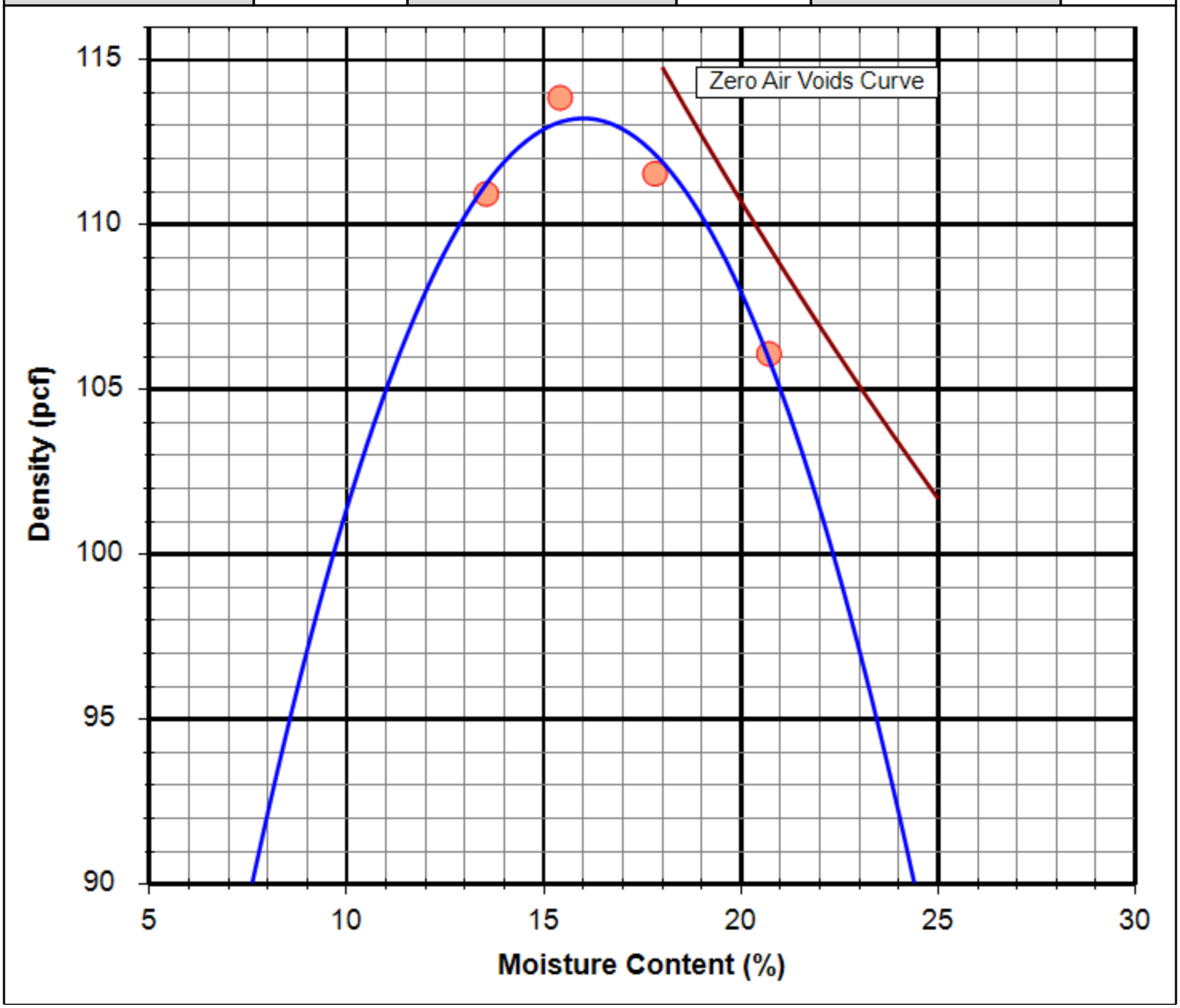
FROM THE GROUND UP

MODIFIED PROCTOR MOISTURE DENSITY TEST, ASTM D1557, METHOD A

Client:	Etegra, Inc.	**Project No.:**	J033011.01
Project:	Port Isabel Texas Kitchen and WWTP Facilities, Port Isabel, TX	**Date:**	2/4/2019
Sample Obtained From:	Boring W-1	**Depth (ft.):**	0.0 - 2.5

Sample Description:	Dark brown moist stiff FAT CLAY, trace roots	**LL**	**PL**	**PI**	**USCS**
		62	20	42	CH

Maximum Dry Density (pcf):	113.2	**Optimum Moisture Content:**	15.9%	**In Situ Moisture Content:**	25.4%



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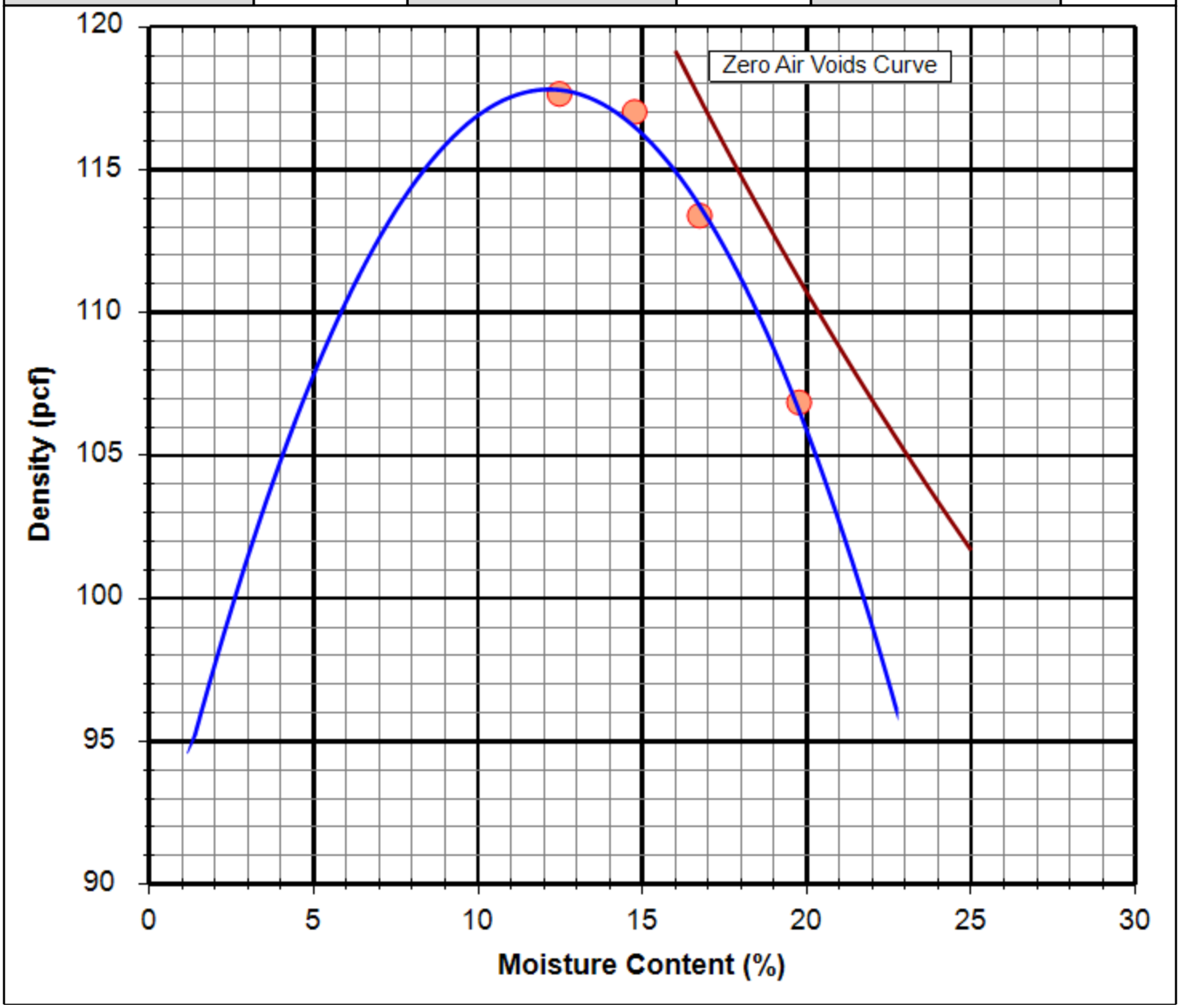
MODIFIED PROCTOR MOISTURE DENSITY TEST, ASTM D1557, METHOD A

Client:	Etegra, Inc.	**Project No.:**	J033011.01
Project:	PIDC Kitchen and WWTP Improvements, Los Fresnos, TX	**Date:**	2/4/2019
Sample Obtained From:	Boring W-4	**Depth (ft.):**	0.0 - 5.0

Sample Description:	Brown moist stiff to very stiff lean CLAY with carbonate deposits, trace roots	**LL**	**PL**	**PI**	**USCS**
		43	18	25	CL

Maximum Dry Density (pcf):	117.8	**Optimum Moisture Content:**	12.3%	**In Situ Moisture Content:**	18.2%



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GEOTECHNOLOGY INC
FROM THE GROUND UP

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
ASTM D2166

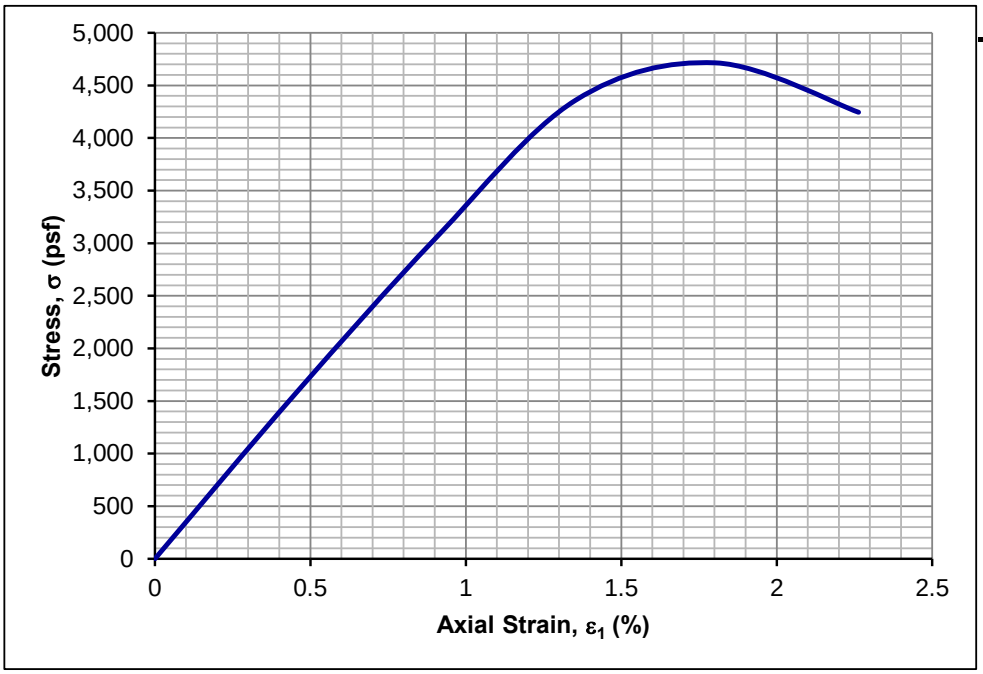
CLIENT : Etegra, Inc. DATE: 1/22/2019
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: K-4 SAMPLE NO.: PT-2A DEPTH (ft.): 2.8-3.3
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown moist very stiff LEAN CLAY with carbonate deposits

LIQUID LIMIT (%): 36 PLASTIC LIMIT (%): 19 PLASTICITY INDEX (%): 17 USCS: CL
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.84
HEIGHT (in.):	5.55
HEIGHT TO DIAMETER RATIO:	1.95
WET UNIT WEIGHT (pcf):	123.8
DRY UNIT WEIGHT (pcf):	107.1
VOID RATIO:	0.60
MOISTURE CONTENT (%)*:	15.6
DEGREE OF SATURATION (%):	71

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**0.8**
AXIAL STRAIN AT FAILURE (%):	**1.8**
TIME TO FAILURE (min.):	**1.4**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**4,710**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**2,355**
SENSITIVITY, S_t:	**-**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from sample cuttings.

GEOTECHNOLOGY INC
FROM THE GROUND UP

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
ASTM D2166

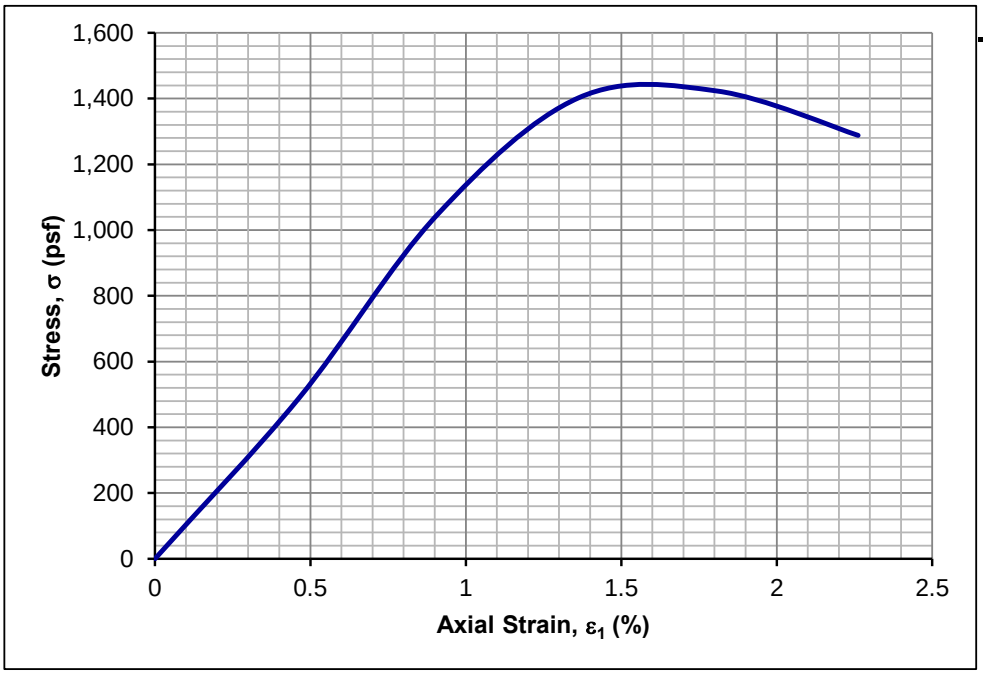
CLIENT : Etegra, Inc. DATE: 1/22/2019
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: K-6 SAMPLE NO.: PT-6A DEPTH (ft.): 12.5-13.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown very moist medium stiff LEAN CLAY

LIQUID LIMIT (%): PLASTIC LIMIT (%): PLASTICITY INDEX (%): USCS:
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.85
HEIGHT (in.):	5.53
HEIGHT TO DIAMETER RATIO:	1.94
WET UNIT WEIGHT (pcf):	120.6
DRY UNIT WEIGHT (pcf):	92.1
VOID RATIO:	0.86
MOISTURE CONTENT (%)*:	31.0
DEGREE OF SATURATION (%):	99

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**0.8**
AXIAL STRAIN AT FAILURE (%):	**1.8**
TIME TO FAILURE (min.):	**1.5**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**1,420**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**710**
SENSITIVITY, S_t:	**-**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.

GEOTECHNOLOGY INC
FROM THE GROUND UP

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
ASTM D2166

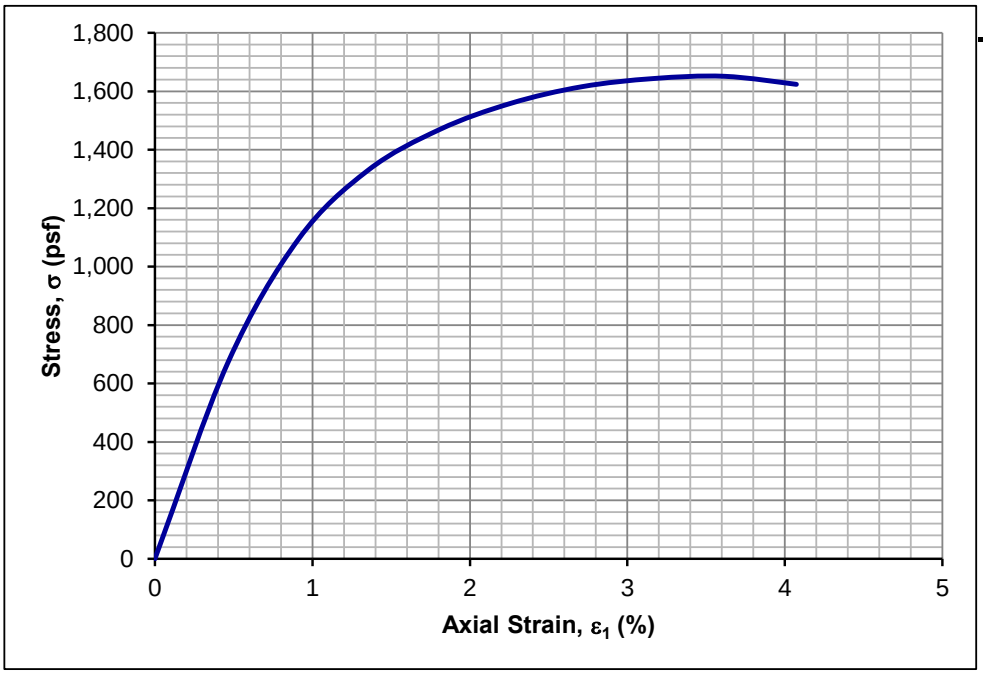
CLIENT : Etegra, Inc. DATE: 1/22/2019
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: W-2 SAMPLE NO.: PT-4A DEPTH (ft.): 7.5-8.0
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown and gray moist medium stiff FAT CLAY, trace gravel

LIQUID LIMIT (%): 75 PLASTIC LIMIT (%): 26 PLASTICITY INDEX (%): 49 USCS: CH
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.84
HEIGHT (in.):	5.54
HEIGHT TO DIAMETER RATIO:	1.95
WET UNIT WEIGHT (pcf):	120.3
DRY UNIT WEIGHT (pcf):	90.9
VOID RATIO:	0.89
MOISTURE CONTENT (%)*:	32.3
DEGREE OF SATURATION (%):	100

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**0.8**
AXIAL STRAIN AT FAILURE (%):	**3.6**
TIME TO FAILURE (min.):	**2.9**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**1,650**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**825**
SENSITIVITY, S_t:	**-**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.

GEOTECHNOLOGY INC
FROM THE GROUND UP

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
ASTM D2166

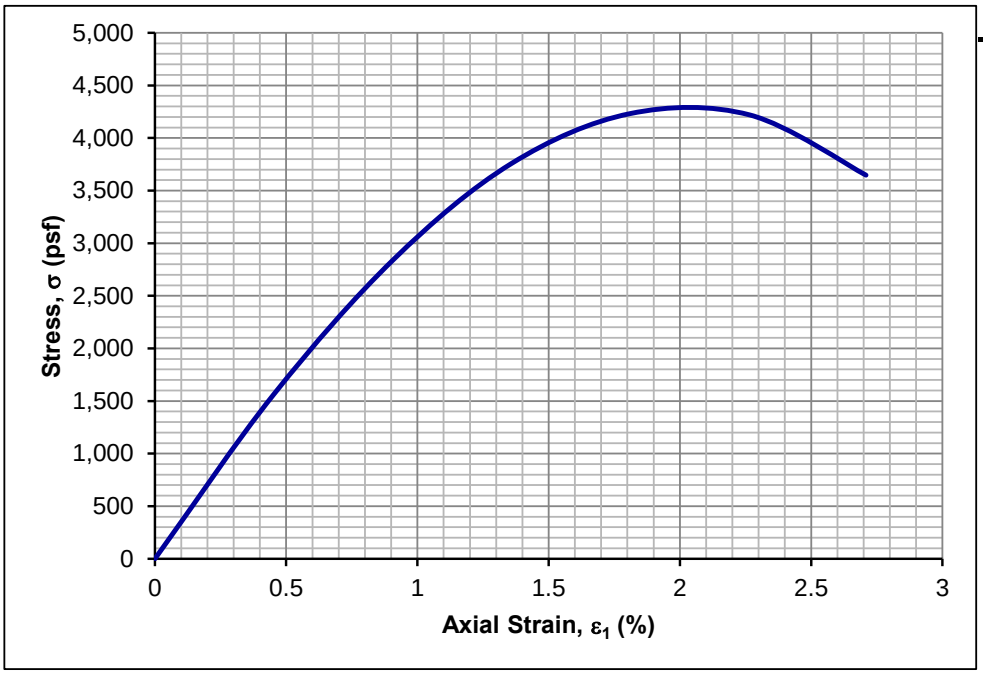
CLIENT : Etegra, Inc. DATE: 1/1/2219
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: W-4 SAMPLE NO.: PT-3A DEPTH (ft.): 5.2-5.7
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Light brown very moist very stiff LEAN CLAY, trace sand

LIQUID LIMIT (%): PLASTIC LIMIT (%): PLASTICITY INDEX (%): USCS:
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

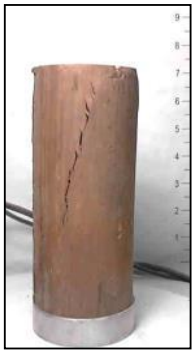
SAMPLE DATA	
DIAMETER (in.):	2.85
HEIGHT (in.):	5.54
HEIGHT TO DIAMETER RATIO:	1.95
WET UNIT WEIGHT (pcf):	119.3
DRY UNIT WEIGHT (pcf):	93.6
VOID RATIO:	0.83
MOISTURE CONTENT (%)*:	27.5
DEGREE OF SATURATION (%):	91

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**0.8**
AXIAL STRAIN AT FAILURE (%):	**1.8**
TIME TO FAILURE (min.):	**1.4**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**4,230**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**2,115**
SENSITIVITY, S_t:	**-**



FAILURE SHAPES

FRONT VIEW



SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.

GEOTECHNOLOGY INC
FROM THE GROUND UP

ONE-DIMENSIONAL SWELL OR COLLAPSE OF SOILS
ASTM D4546

CLIENT : Etegra, Inc. DATE: 2/1/2019
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: K-4 SAMPLE NO.: PT-2B DEPTH (ft.): 3.4-3.5
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown moist very stiff LEAN CLAY with carbonate deposits

LIQUID LIMIT (%): 36 PLASTIC LIMIT (%): 19 PLASTICITY INDEX (%): 17 USCS: CL
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

SAMPLE DATA

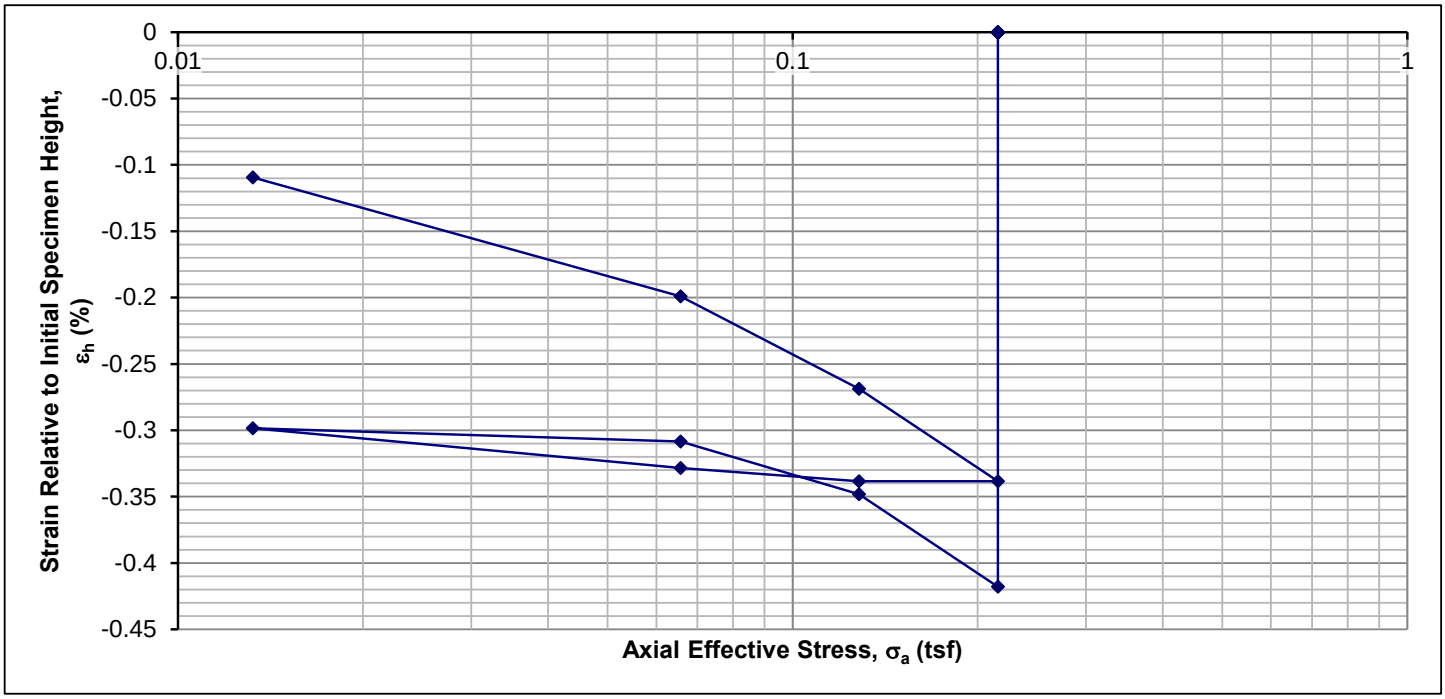
	INITIAL	FINAL
DIAMETER (in.):	2.500	-
HEIGHT (in.):	1.005	1.005
HEIGHT TO DIAMETER RATIO:	2.49	
AVERAGE MOISTURE CONTENT OF TRIMMINGS (%):	14.30	-
MOISTURE CONTENT (%)*:	14.70	17.21
WET UNIT WEIGHT (pcf):	129.0	131.8
DRY UNIT WEIGHT (pcf):	112.5	112.5
VOID RATIO:	0.50	0.50
DEGREE OF SATURATION (%):	79.6	93.2

EQUIPMENT DATA

APPARATUS:	1064 Conbel (SN 726)
TECHNICIAN:	I. Ivanov
TEST METHOD:	B

SWELL/COLLAPSE DATA

INITIAL HEIGHT, h_0 (in.):	1.0050
HEIGHT AFTER DRY LOADING, h_1 (in.):	1.0008
FINAL HEIGHT, h_2 (in.):	1.0050
VERTICAL IN-SITU STRESS, $\sigma_{in\text{-}situ}$ (psf):	450
AXIAL STRESS AT INUNDATION, $\sigma_{inundation}$ (psf):	431
SWELL OR COLLAPSE STRAIN, $\varepsilon_{s,c}$ (%):	0.42
(+ = SWELL, - = COLLAPSE)	



*Moisture contents determined from entire specimen.
REMARKS :

GEOTECHNOLOGY INC
FROM THE GROUND UP

ONE-DIMENSIONAL SWELL OR COLLAPSE OF SOILS
ASTM D4546

CLIENT : Etegra, Inc. DATE: 2/5/2019
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: K-5 SAMPLE NO.: PT-3A DEPTH (ft.): 5.2-5.3
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Light brown to brown moist very stiff FAT CLAY

LIQUID LIMIT (%): 53 PLASTIC LIMIT (%): 21 PLASTICITY INDEX (%): 32 USCS: CH
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

SAMPLE DATA

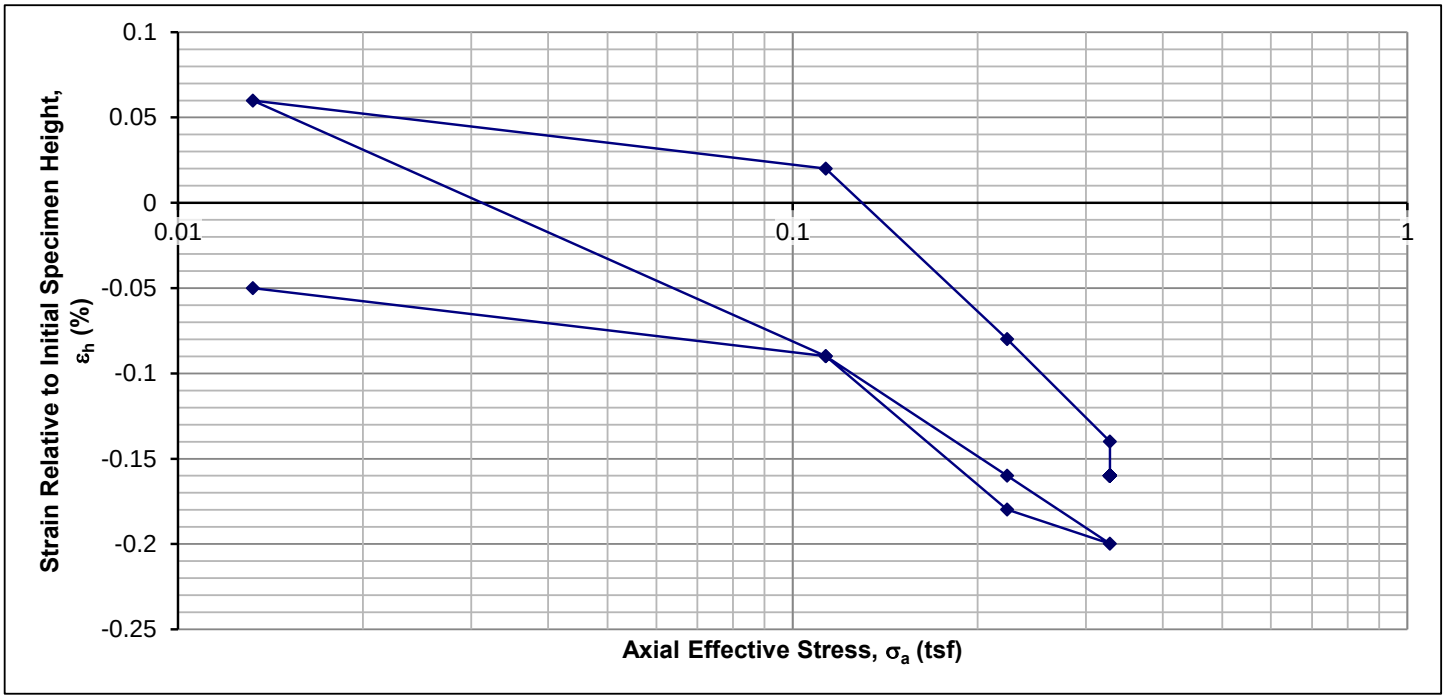
	INITIAL	FINAL
DIAMETER (in.):	2.500	-
HEIGHT (in.):	1.001	1.003
HEIGHT TO DIAMETER RATIO:	2.50	
AVERAGE MOISTURE CONTENT OF TRIMMINGS (%):	11.46	-
MOISTURE CONTENT (%)*:	10.38	24.10
WET UNIT WEIGHT (pcf):	108.0	121.2
DRY UNIT WEIGHT (pcf):	97.8	97.7
VOID RATIO:	0.72	0.73
DEGREE OF SATURATION (%):	38.7	89.6

EQUIPMENT DATA

APPARATUS:	1064 Conbel (SN 725)
TECHNICIAN:	I. Ivanov
TEST METHOD:	B

SWELL/COLLAPSE DATA

INITIAL HEIGHT, h_0 (in.):	1.0010
HEIGHT AFTER DRY LOADING, h_1 (in.):	0.9996
FINAL HEIGHT, h_2 (in.):	1.0026
VERTICAL IN-SITU STRESS, $\sigma_{in\text{-}situ}$ (psf):	560
AXIAL STRESS AT INUNDATION, $\sigma_{inundation}$ (psf):	656
SWELL OR COLLAPSE STRAIN, $\varepsilon_{s,c}$ (%):	-0.02
(+ = SWELL, - = COLLAPSE)	



*Moisture contents determined from entire specimen.
REMARKS :

GEOTECHNOLOGY INC
FROM THE GROUND UP

ONE-DIMENSIONAL SWELL OR COLLAPSE OF SOILS
ASTM D4546

CLIENT : Etegra, Inc. DATE: 2/5/2019
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: K-6 SAMPLE NO.: PT-5A DEPTH (ft.): 10.0-10.2
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Brown very moist medium stiff LEAN CLAY

LIQUID LIMIT (%): 42 PLASTIC LIMIT (%): 24 PLASTICITY INDEX (%): 18 USCS: CL
GRAVEL (%): 0.3 SAND (%): 1.2 SILT (%): 55.4 CLAY (%): 43.1
SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

SAMPLE DATA

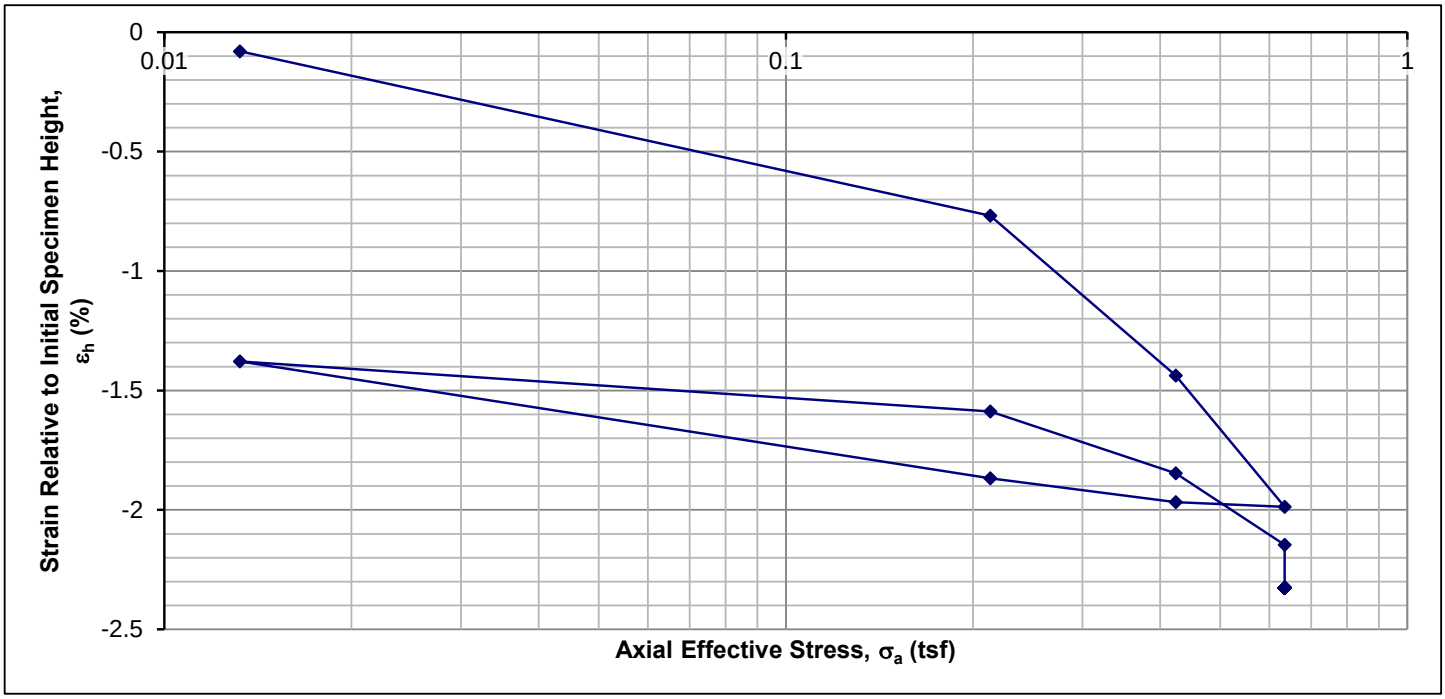
	INITIAL	FINAL
DIAMETER (in.):	2.500	-
HEIGHT (in.):	1.002	1.025
HEIGHT TO DIAMETER RATIO:	2.50	
AVERAGE MOISTURE CONTENT OF TRIMMINGS (%):	30.38	-
MOISTURE CONTENT (%)*:	30.93	30.34
WET UNIT WEIGHT (pcf):	119.9	116.6
DRY UNIT WEIGHT (pcf):	91.6	89.5
VOID RATIO:	0.84	0.88
DEGREE OF SATURATION (%):	99.3	92.7

EQUIPMENT DATA

APPARATUS:	1064 Conbel (SN 726)
TECHNICIAN:	I. Ivanov
TEST METHOD:	B

SWELL/COLLAPSE DATA

INITIAL HEIGHT, h_0 (in.):	1.0015
HEIGHT AFTER DRY LOADING, h_1 (in.):	0.9800
FINAL HEIGHT, h_2 (in.):	1.0248
VERTICAL IN-SITU STRESS, $\sigma_{in\text{-}situ}$ (psf):	1,200
AXIAL STRESS AT INUNDATION, $\sigma_{inundation}$ (psf):	1,269
SWELL OR COLLAPSE STRAIN, $\varepsilon_{s,c}$ (%):	-0.18
(+ = SWELL, - = COLLAPSE)	



*Moisture contents determined from entire specimen.
REMARKS :

GEOTECHNOLOGY INC
FROM THE GROUND UP

ONE-DIMENSIONAL SWELL OR COLLAPSE OF SOILS
ASTM D4546

CLIENT : Etegra, Inc. DATE: 2/1/2019
PROJECT NO.: J033011.01
PROJECT: PIDC Kitchen and WWTP Improvements
LOCATION: Los Fresnos, TX

BORING NO.: W-4 SAMPLE NO.: PT-3B DEPTH (ft.): 5.7-5.9
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Light brown very moist very stiff LEAN CLAY, trace sand

LIQUID LIMIT (%): PLASTIC LIMIT (%): PLASTICITY INDEX (%): USCS:
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.70 (Assumed)

SAMPLE DATA

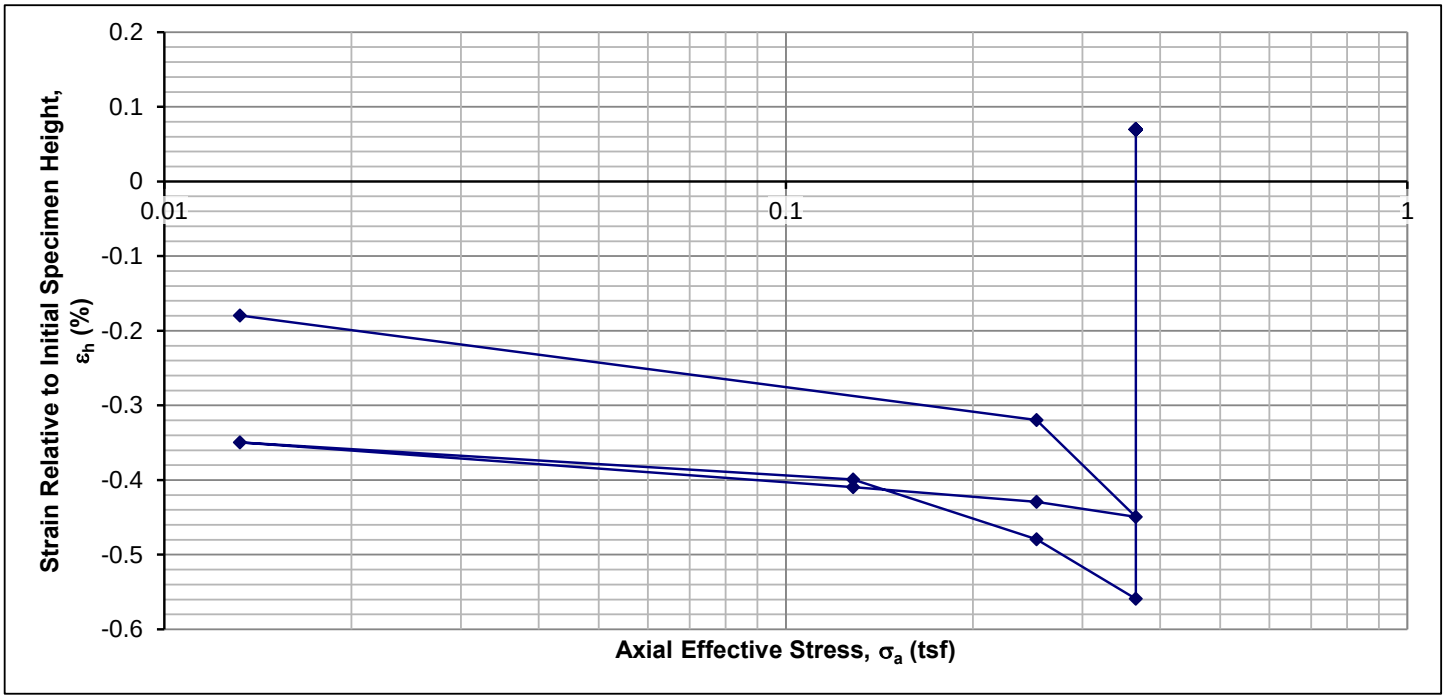
	INITIAL	FINAL
DIAMETER (in.):	2.500	-
HEIGHT (in.):	1.002	1.001
HEIGHT TO DIAMETER RATIO:	2.50	
AVERAGE MOISTURE CONTENT OF TRIMMINGS (%):	25.70	-
MOISTURE CONTENT (%)*:	26.12	28.92
WET UNIT WEIGHT (pcf):	118.7	121.4
DRY UNIT WEIGHT (pcf):	94.1	94.2
VOID RATIO:	0.79	0.79
DEGREE OF SATURATION (%):	89.2	98.9

EQUIPMENT DATA

APPARATUS:	1064 Conbel (SN 725)
TECHNICIAN:	I. Ivanov
TEST METHOD:	B

SWELL/COLLAPSE DATA

INITIAL HEIGHT, h_0 (in.):	1.0015
HEIGHT AFTER DRY LOADING, h_1 (in.):	0.9959
FINAL HEIGHT, h_2 (in.):	1.0008
VERTICAL IN-SITU STRESS, $\sigma_{in\text{-}situ}$ (psf):	680
AXIAL STRESS AT INUNDATION, $\sigma_{inundation}$ (psf):	731
SWELL OR COLLAPSE STRAIN, $\varepsilon_{s,c}$ (%):	0.63
(+ = SWELL, - = COLLAPSE)	



*Moisture contents determined from entire specimen.
REMARKS :

APPENDIX III:
DESIGN OF RIBBED MAT FOUNDATIONS

DESIGN OF RIBBED MAT FOUNDATIONS

BY

JOSEPH P. HARTMAN

AND

B. H. JAMES

U.S. ARMY CORPS OF ENGINEERS

SOUTHWESTERN DIVISION

DALLAS, TEXAS

REVISED

JANUARY 1988

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PART I - GENERAL REQUIREMENTS FOR RIBBED MATS

1. REFERENCES.

1.1 Engineering Instruction Manual, Corps of Engineers, Southwestern Division, (latest edition).

1.2 "Criteria for Selection and Design of Residential Slabs-on-Ground," Building Research Advisory Board (BRAB), Report No. 33 to the Federal Housing Administration, 1968.

1.3 "Design and Construction of Post-Tensioned Slabs-on-Ground," Post Tensioning Institute (PTI), 1980.

1.4 TM 5-818-7, Foundations in Expansive Soils, Corps of Engineers, 1983.

1.5 Letter, SWDED-G, 16 April 1987, Criteria for Developing Geotechnical Design Parameters for Ribbed Mat Design Methodology (Criteria Letter XV 7-12).

2. BACKGROUND. Ribbed mat foundations consist of a thin slab on grade which acts monolithically with a grid of stiffening beams beneath the slab. The beams (ribs) are cast in trenches dug in the foundation soil. Ribbed mats combine the economic advantages of shallow foundations with the performance advantages of monolithic floors. Ribbed mats are especially useful for minimizing differential foundation movements in areas with expansive soils.

3. DESIGN METHODS.

3.1 EXPANSIVE SOILS.

3.1.1 Behavior.

3.1.1.1 Center Lift. In the center lift condition the soil near the edge of the slab drops in relation to the soil near the center. This is due to moisture retention by the interior soils and the drying and shrinking of perimeter soils. As this occurs, the perimeter soil provides less support for the edge of the slab which then acts as a cantilever. This is illustrated in Figure A1 of Appendix A.

3.1.1.2 Edge Lift. In the edge lift condition the soil near the edge of the slab rises in relation to the soil near the center. This is due to the increasing moisture content and subsequent swelling of soil near the edge. The swelling soil raises the edge of the slab, causing some of the slab to lift off the soil. Interior loads cause the slab to sag and recontact the soil at some interior location. The slab thus tends to act as

a beam, simply supported by the soil at the edge, and by soil towards the center of the slab. The amount of support at the center depends on numerous parameters such as interior loads, rib bending stiffness, soil swell pressures, and the magnitude of soil swelling. Typical edge lift behavior is illustrated in Figure A3 of Appendix A.

3.1.2 SWD Method. All ribbed mats on expansive soils, except for family housing, shall be designed in accordance with the provisions of Part II of this report. Ribbed mats for family housing may be designed in accordance with Part II or paragraphs 3.1.3 or 3.1.4.

3.1.3 PTI Method. The PTI method (reference 1.3) may only be used for design of family housing foundations on expansive soils. Specifically, slab width (short dimension) should not exceed 40 feet, rib depths should not exceed 30 inches, loading should consist only of perimeter loads and light interior distributed loads (DL+LL$\leq$100 psf), soils should be fairly weak in-situ materials with no extensive substitution of non-expansive fill. When using the PTI method, the following provisions shall apply: Rib spacing shall not exceed 15 feet; concrete tensile stress shall not exceed $4\sqrt{f'c}$; the minimum effective prestress shall be 100 psi.

3.1.4 BRAB Method. The BRAB report (reference 1.2) may only be used for design of foundations for family housing. However, the PTI method is preferred, since the BRAB method may produce unreasonable results for large foundations.

3.1.5 Computer Method. In lieu of paragraph 3.1.2, ribbed mats may be designed using appropriate computer programs. Such programs must be capable of modeling the variable soil swell due to moisture changes, and the non-linear soil-structure interaction near the perimeter of the foundation. One such computer program is CBEAMC, program X0050 in the Corps of Engineers Civil Engineering Library.

3.1.6 Load Factors. When using the above methods to design ribbed mats for center lift and edge lift conditions, load factors may be multiplied by .75 (strength method) or allowable stresses may be increased by one-third (working stress method). This provision does not apply to the allowables given for the PTI method, since those allowables have already been increased from the usual provisions of ACI.

3.2 NON-EXPANSIVE SOILS. Ribbed mat slabs on non-expansive soils need not be designed for bending due to center lift or edge lift conditions. Beam on elastic foundation analyses may be used to determine the effects of concentrated loads on ribs, or ribs may be designed as conventional strip or spot footings.

3.3 SOIL PROPERTIES. Soil properties for design of ribbed mats will be provided in the Foundation Design Analysis by the Corps of Engineers. Criteria for developing these properties is included in reference 1.5. The properties necessary for design in accordance with paragraph 3.1.2 consist of the following, which are defined in Appendix A:

qa - allowable bearing pressure
k - subgrade modulus
Ym - soil heave
Lm - edge moisture variation distance
Psw - pressure of swelling soil acting on perimeter rib

4. MINIMUM REQUIREMENTS.

4.1 SUBGRADE PREPARATION. A vapor barrier, capillary water barrier, and a minimum of 18 inches of non-expansive fill will normally be used beneath ribbed mats. Additional non-expansive fill will often be used to lessen the effects of highly expansive soils. These requirements will be detailed in the Foundation Design Analysis.

4.2 SLAB. For family housing and other small lightly loaded buildings a 4 inch slab may be used. For other buildings the minimum slab thickness will be 5 inches. Minimum slab reinforcing shall be 0.2 percent. Where slabs are subjected to vehicular loading they must be designed for the maximum wheel load, similar to paving. Use 650 psi flexural strength concrete for slabs subject to wheel loads.

4.3 GRID GEOMETRY. Ribs should be located to form a continuous grid. Rib spacing should not exceed 20 feet in expansive soils, or 25 feet in non-expansive soils. Locations of ribs should conform to significant wall and column loads, and may be used to resist thrusts from rigid frame reactions. Ribs should be provided around large openings in the slab. In expansive soils diagonal ribs are required at exterior corners.

Expansion joints should be provided at 250 foot intervals, and should also be used to break irregularly shaped buildings into rectangular segments. Foundations for family housing do not require expansion joints due to irregular shapes.

4.4 RIB SIZE. Minimum rib depth is 20 inches. Rib depths should usually not exceed 3 feet to minimize construction difficulties related to placing reinforcement and maintaining trench walls. If deeper ribs are used, rib width should also be increased. Minimum rib width is 12 inches except for family housing foundations, where 10 inch ribs may be used. Sufficient rib width must also be provided to transfer wall and column loads to the soil as strip footings. The allowable soil bearing capacity may not be exceeded when considering the width of the rib plus an effective slab width on each side of the rib. The

effective slab width for bearing is limited to the thickness of the slab. At column locations an alternate is to provide fillets at rib intersections, sufficient to act as spot footings for column loads.

4.5 RIB CAPACITY. Concrete should have a minimum compressive strength of f'c=3000 psi at 28 days. Reinforcing shall be grade 60, except ties may be grade 40. Minimum reinforcing ratio (As/Ag) shall be .0033 top and .0033 bottom, this may be reduced to .005 total in non-expansive soils. Use #3 ties at 24 inches, minimum. These minimums should be sufficient for shrinkage stresses and for unpredictable soil behavior.

4.6 PRESTRESSED MATS. For prestressed ribbed mats, not designed per PTI, all the above minimum requirements apply except that slab and rib top reinforcement may be deleted and replaced by appropriate post-tensioning strands. Mild steel shall still be provided in the bottom of ribs. Minimum effective prestress shall be 100 psi on the gross area of the slab, including effects of subgrade friction as calculated by the PTI method, reference 1.3. Concrete tensile stress shall be limited to $3\sqrt{f'c}$ and shear stress limited to $1.1\sqrt{f'c}$. A one-third overstress may be allowed per paragraph 3.1.6.

4.7 CONSTRUCTION DETAILS.

4.7.1 Conventionally Reinforced. Construction joint spacing should not exceed 50 feet in either direction. A horizontal construction joint may be provided in the ribs at the base of the capillary water barrier when unstable trench walls may cause construction difficulties. However, this joint is discouraged because of increased potential for shrinkage cracks in the slab.

4.7.2 Prestressed. Construction joint spacing shall not exceed 75 feet in either direction. Tendons within each placement shall be stressed to 15 percent of the final prestress not more than 24 hours after the concrete has attained sufficient strength to withstand the partial prestress. Other construction procedures for prestressed ribbed mats shall conform to reference 1.3.

4.7.3 Contractor Designs. Ribbed mat foundations may be designed as prestressed or conventionally reinforced as selected by the engineer. The plans and specifications shall not include the option of changing the ribbed mat from one type to another. The reason for this prohibition is that design parameters (e.g., moments of inertia) may be dependent on the type of ribbed mat being designed and may affect calculated shears and moments. This does not prohibit revisions of the slab type as a result of contractor value engineering proposals. However, such revisions must include a complete design of the ribbed mat foundation using appropriate design parameters in accordance with this report.

PART II - ANALYSIS OF RIBBED MAT FOUNDATIONS ON EXPANSIVE SOILS

1. SCOPE. This part of the report contains the basic rules for design of ribbed mats in expansive soils. This method may be used to predict shears, moments and deflections in ribs subject to soil movement due to changing moisture content. For a commentary on the design method refer to Appendix A; for example design calculations refer to Appendix B. The design method from Part II should be used in conjunction with the "minimum requirements" for ribbed mats, as presented in Part I.

2. GENERAL

2.1 NOTATION.

	C	= Correction factor for equivalent cantilever length
	D	= Beam deflection (IN)
	I	= Moment of inertia per foot, I=Ir/S (IN^4/FT)
	Ir	= Moment of inertia of rib (IN^4)
*	k	= Modulus of subgrade reaction (PCI)
	Lo	= Basic length of cantilever (FT)
	Lc	= Equivalent length of cantilever, center lift (FT)
	Le	= Equivalent length of simple beam, edge lift (FT)
	Li	= Distance from perimeter to location of interior load (FT)
*	Lm	= Edge moisture variation distance (FT)
	Lb	= Width of soil bearing at perimeter, edge lift (FT)
	M	= Bending moment per foot (FT-LB/FT)
	Mr	= Bending moment per rib, Mr=MxS (FT-LB)
	Pi	= Interior load (PLF)
	Pp	= Perimeter load (PLF)
*	Psw	= Pressure of swelling soil on perimeter rib (PSF)
	R	= End reaction at perimeter for equivalent simple beam (PLF)
	S	= Rib spacing (FT)
	w	= Uniform load (PSF)
	V	= Shear per foot (LB/FT)
	Vr	= Shear per rib, Vr=VxS (LB)
*	Ym	= Soil heave (IN)
	θ	= Rotation of support of equivalent cantilever (RAD)

* q_a = ALLOWABLE BEARING PRESSURE (PSF)

2.2 UNITS. The equations presented in section 3 are written for units as defined in the above notation. If other units are used the equations must be modified appropriately.

2.3 RIB DEFINITIONS. Ribs are defined as perimeter, transverse or diagonal as shown in Figure 1. Note that transverse refers to ribs parallel to either axis of the building.

* VALUES NEEDED FROM GEOTECH

FIGURE 1 - RIB DEFINITIONS

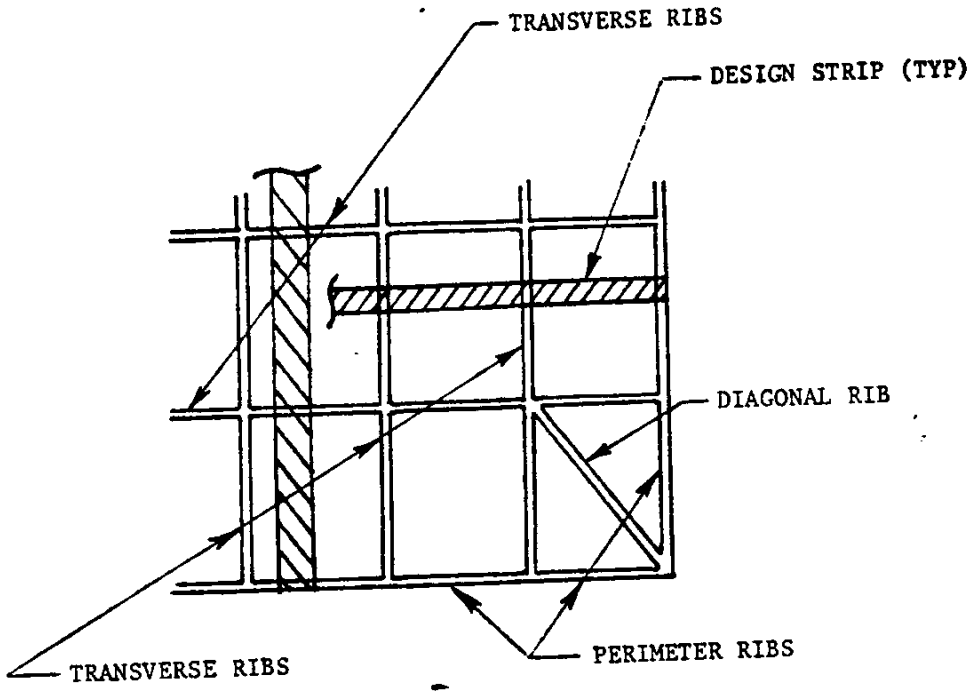
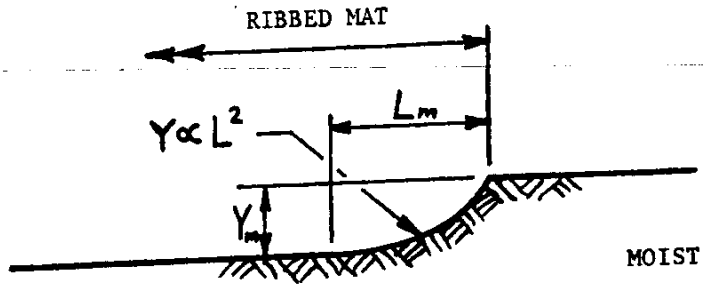


FIGURE 2 - SOIL EDGE PROFILE



2.4 STRIP ANALYSIS. The analysis is based on a strip assumption, ignoring the effects of the grid configuration of the ribs. The formulas and examples presented below are for an equivalent 1-foot strip, using "per foot" values for loads and stiffness.

2.5 SOIL EDGE PROFILE. For edge lift the maximum swell occurs at the perimeter and decreases rapidly toward the interior. The soil profile is assumed to be parabolic (in the unloaded condition) and is illustrated in Figure 2.

3. ANALYSIS METHOD.

3.1 TRANSVERSE RIB - CENTER LIFT.

3.1.1 General. Center lift analysis is based on an equivalent cantilever beam to determine moments, shears and deflections.

3.1.2 Moment. The length of the equivalent cantilever can be calculated as:

$$Lc = C \times Lo$$

where: $$Lo = 2.3 + .4\ Lm$$

$$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$$

The maximum moment may then be calculated from statics using conventional cantilever formulas such as:

$$M = Pp\ Lc + 1/2\ w\ Lc^2$$

The moment can then be assumed to be constant for a distance Lc/2 and then to decrease linearly to zero at a distance 5Lc from the perimeter. To obtain the design moment for a given rib, multiply the calculated per-foot moment by the appropriate rib spacing ($Mr = M \times S$).

3.1.3 Shear. The maximum shear may be calculated from statics using the same equivalent cantilever as for moment.

$$V = Pp + w\ Lc$$

The shear may then be assumed to decrease linearly from V at the cantilever support, to zero at a distance 5Lc from the perimeter. To obtain the design shear for a given rib, multiply the calculated per-foot shears by the appropriate rib spacing ($Vr = V \times S$).

3.1.4 Deflection. Deflection at the perimeter is the sum of three components: bending deflection of the equivalent cantilever, vertical translation of the cantilever support, and rotation

of the cantilever support. Rotation of the support may be calculated as:

$$\theta = M^{1.4} / 9800\ I\ k^{.5}$$

The perimeter deflection is then:

$$D = .11 + 12\ Lc\ \theta$$

where .11 inches is an approximation for the support translation plus the cantilever bending, and (12 Lc) is the length in inches.

Use the deflection calculated above to compare with allowable deflection. The allowable deflection may be determined by using 4Lc as the length between points of zero and maximum deflection.

3.2 TRANSVERSE RIB - EDGE LIFT.

3.2.1 General. Edge lift analysis is based on an equivalent simple beam, supported at the perimeter and at some interior location.

3.2.2 Deflection. The first step in calculating deflection is to determine the length of the equivalent simple beam. The appropriate length depends on many parameters, including the deflection. Therefore, deflection must first be estimated to determine equivalent length, then a deflection is calculated based on that length. The process is repeated until calculated deflection matches the assumed deflection. The equivalent simple beam length may be calculated as:

$$Le = 7.5\ I^{.17}\ Li^{.37}\ D^{.12} / w^{.07}\ Pi^{.11}$$

The perimeter end reaction for this beam may be calculated from statics. For a given case the reaction may be:

$$R = Pp + 1/2\ w\ Le + Pi(Le-Li)/Le$$

The width of soil bearing at the perimeter can be approximated as:

$$Lb = 1.1\ (R/Psw)$$

where Psw is selected from a curve of heave versus bearing pressure, corresponding to the estimated deflection used during this iteration (see reference 1.5).

The edge deflection is found by determining the soil swell at a distance Lb from the perimeter, based on the parabolic swell profile:

$$D = Ym(Lm-Lb)^2/Lm^2$$

When satisfying deflection criteria, use the calculated deflection and equivalent simple beam length.

3.2.3 Moment. Once the simple beam equivalent length has been determined, the bending moments may be calculated based on statics. To obtain rib design moments, multiply per-foot moments by the rib spacing.

3.2.4 Shear. Once the simple beam equivalent length has been determined, the shears may be calculated based on statics. To obtain rib design shears, multiply per-foot shears by the rib spacing. Near the interior support the design shear need not exceed:

$$V = Pi + w(Le-Li)$$

This is due to the effects of the actual distributed soil support, rather than the point support assumed in the simple beam analysis.

3.2.5 Special Cases. If Pi=0 or if Li>Le make the following substitution in the equation for Le:

$$1.4 = Li^{.37} / Pi^{.11}$$

The equation for the simple beam length then becomes:

$$Le = 10.5\ I^{.17}\ D^{.12} / w^{.07}$$

3.3 PERIMETER RIB.

3.3.1 Center Lift. For center lift the perimeter rib will have no support from the soil and must be designed to span between transverse ribs for the perimeter wall loads.

3.3.2 Edge Lift. For edge lift the soil pressure on the perimeter rib will exceed the applied perimeter loads. The perimeter rib must designed to span between transverse ribs for this net upward force.

3.4 DIAGONAL RIB. Diagonal ribs are used to support exterior corners for center lift conditions, if loss of support occurs under both perimeter ribs. Diagonal ribs must be designed to provide the same moment and shear capacity as the larger of the two adjacent transverse ribs.

3.5 INTERIOR RIB. Interior ribs and rib intersections should be located at significant wall and column loads. The ribs can be designed for these loads as strip or spot footings, using beam-on-elastic-foundation methods. Differential soil movement due to moisture change is assumed not to occur except at the perimeter. However, to account for unpredictable interior soil movements, interior ribs must have the minimum size and capacity as required in Part I.

APPENDIX A - COMMENTARY ON PART II

1. SCOPE. Actual behavior of ribbed mats in expansive soils involves complex, non-linear, soil-structure interaction. The best solution for such behavior is provided by computer programs. The hand design method has been developed to approximate such computer results. Hand solutions have been checked by computer analyses; results have been within acceptable limits of error. However, such checks have been made only for a limited range for each design parameter, as shown in Table A1, corresponding to the usual values for military construction within Southwestern Division. If a wider range of parameters is applied to the hand design formulas, the results may be less accurate.

TABLE A1

Parameter	Units	Minimum	Maximum
k	pci	50	200
Ym	in	0.5	3.0
Lm	ft	2	8
I	in^4/ft	750	6000
Pp	lb/ft	1000	5000
Pi	lb/ft	0	5000
Li	ft	6	20
w	psf	100	250
Psw	psf	2000	8000

2. GENERAL.

2.1 NOTATION.

Ir = moment of inertia of rib. For non-prestressed rib mats Ir should be the effective moment of inertia, calculated per ACI 318, Section 9.5.2.3.

k = Modulus of subgrade reaction. This parameter is the ratio of the soil pressure at the base of the concrete and the corresponding settlement. Since modulus values are typically determined by plate-load test at the ground surface, they should be corrected for depth and for footing size (expected high pressure area between concrete and soil). Analyses have indicated that the high bearing pressure area for center lift conditions will occur in an area several feet long parallel to the transverse rib and several feet on each side of the rib. A crude approximation for this area would be 5 feet square. This approximation should be adequate for design, since calcualations are not sensitive to small changes in the modulus of subgrade reaction.

qa = Allowable bearing pressure. This is the safe bearing capacity of the soil at the base of the ribs. A factor of safety of 3.0 is recommended for computing this value.

Lm = Edge moisture variation distance. This represents the distance, inward from the edge of the slab, over which the moisture content of the soil changes. Much judgement is required in determining this value.

Psw = Pressure of swelling soil on perimeter rib. This is the interface pressure between the soil and the base of the exterior rib, due to an increase in soil moisture content. The pressure which can be exerted by the swelling soil is dependent on the amount the surface of the soil is allowed to rise. Therefore Psw is usually presented as a curve of pressure versus heave, as described in reference 1.5 of Part I. The actual upward deflection of the edge of the slab is a complex interaction between swell potential, structural loads, and mat stiffness, all of which combine to determine the interface pressure near the perimeter.

Ym = Soil heave. This is the differential vertical movement of the soil representing either soil heave (edge lift) or soil shrinkage (center lift). The magnitude of Ym is the computed vertical movement of a particle of soil at the ground surface due to a change in moisture content. This value should be based on the accumulation of potential volume changes for the full thickness of the active zone (Za), with no significant loads applied to the foundation. The value of Ym may differ for edge lift and center lift conditions.

Pi, Pp, w = Applied loads. Loads should consist of full dead plus live loads, including dead load of the slab and ribs.

2.2 UNITS.

2.3 RIB DEFINITIONS.

2.4 STRIP ANALYSIS. The hand solution formulas have been developed for analysis of an equivalent 1 foot strip. This is convenient for uniform loads and for soil properties, but requires some calculations for appropriate concentrated loads and bending stiffness. Rib stiffness must be divided by rib spacing to get the per-foot stiffness. If column loads exist they must also be divided by the rib or column spacing to provide an equivalent load per foot. If interior wall loads are parallel to the transverse rib, they must be divided by the rib spacing. These calculations are illustrated in Appendix B.

2.5 SOIL EDGE PROFILE. The edge lift condition occurs when increased moisture content swells exterior soils, and this effect extends under the edge of the slab. The center lift condition occurs when soils under the slab are generally moist and seasonal drying occurs on the exterior, again extending under the slab. This causes the soil to shrink away from the edge of the slab.

The analysis method is based on an assumed parabolic swell profile which occurs uniformly along the perimeter. This is a convenient idealization of actual soil behavior, which is certainly more erratic. However, the parabolic profile has better correlation with measured swells than do other possible edge profile assumptions. Note that the soil profile is not used in the hand design formulas for center lift. However, a parabolic profile was used in the computer analyses for center lift, which formed the basis for the hand design forumlas.

3. ANALYSIS METHOD. Many of the formulas for shears, moments and reactions are idealized, assuming Pp and R are exactly at the perimeter and that w extends to the perimeter. These approximations should usually be acceptable, but the formulas may be modified to account for actual load patterns.

3.1 TRANSVERSE RIB - CENTER LIFT

3.1.1 General. Typical behavior of a transverse rib for center lift conditions is shown in Figure A1. This illustrates the soil bearing pressure and the shear, moment and deflection. Note that the effects of the soil movement extend much farther than the moisture variation distance. The moment and shear distribution close to the edge resemble cantilever behavior.

3.1.2 Moment. The extent of significant moments is illustrated in Figure A1. The length of the equivalent cantilever can be taken as a basic length (Lo) which is dependent on the moisture variation distance, times a correction factor (C) which accounts for secondary effects of several parameters. The value of the correction factor will usually be slightly greater or less than unity. The correction factor was developed to permit accurate approximations of computer results. It was developed from the ratios of actual values to usual values for significant parameters. For example, the "usual" values are: Ym = 1 in, $I = 1500$ in^4/ft, Pp = 3000 lb/ft. Thus:

$$C = (Ym/1.0)^{.12} (I/1500)^{.16} (3000/Pp)^{.12}$$

$$C = .8\ Ym^{.12}\ I^{.16}/\ Pp^{.12}$$

A similar approach was used to develop all the formulas in Part II which have an exponential format.

3.1.3 Shear. Maximum shear occurs near the support of the equivalent cantilever. The extent of significant shears is illustrated in Figure A1.

3.1.4 Deflection. Formulas for deflection include an assumed concrete modulus of elasticity Ec = 3,320,000 psi, for both center lift and edge lift.

Vertical movement at the perimeter is much greater than the bending deflection of the equivalent cantilever. To predict the deflection it is necessary to consider translation and rotation at the support of the equivalent beam. The most significant component is due to rotation at the support. These

components of deflection are shown in Figure A2. The sum of the cantilever bending and the support translation are approximated by the value 0.11 inch. The percent error due to this approximation is negligible when total deflections are large. The percent error is greater when total deflections are small, but then the deflections are not significant anyway.

Allowable deflections (see Part I, reference 1.1) are expressed as a ratio of the difference in vertical movement at any two points, compared to the distance between those points. For example: $D \leq L/600$, where D is the differential displacement. In such formulas it is appropriate to use the point of maximum deflection and a point of near-zero deflection as the two measuring points. For center lift behavior the maximum deflection occurs at the perimeter, and deflections tend to die out at approximately 4Lc (four times the equivalent cantilever length) from the perimeter. Therefore, the ratio D/4Lc is appropriate for comparison with allowable deflections.

3.2 TRANSVERSE RIB - EDGE LIFT.

3.2.1 General. Typical behavior of a transverse rib for edge lift conditions is shown in Figure A3. This illustrates the soil bearing pressure and the shear, moment and deflection. Soil swell lifts the edge of the ribbed mat, which actually rises off the soil for some distance from the perimeter. For shear and moment, this portion of the rib acts as a simply supported beam spanning between soil support at the perimeter and at an interior location.

3.2.2 Deflection. Vertical movement at the perimeter is driven by the tendency of the soil to swell, and is resisted by the downward loads applied on the soil. As the soil swells at the perimeter the slab is lifted off the interior soil. This concentrates soil reactions near the edge, causing very high pressures. The pressures rise so high that they limit the capacity of the soil to swell. Thus, the soil cannot swell as much as it would if not loaded. Deflections can be predicted by balancing the upward force of the soil (the swell pressure times the bearing width) with the downward force of applied loads. This downward force can be determined from statics once an equivalent simple beam length is determined. The method for determining the deflection is shown in Figure A4.

Allowable deflections are expressed as ratios, as discussed in the commentary on paragraph 3.1.4. From Figure A3 it can be seen that the appropriate values for this ratio are the edge deflection and the equivalent simple beam length (D/Lc).

Edge lift deflections are mainly a function of soil properties and applied loads, bending stiffness of the ribs has only a secondary effect. Therefore, it may not be possible to control deflections by increasing the rib stiffness. It may be necessary to accommodate calculated deflections by using a less brittle superstructure or by detailing the superstructure to make it less sensitive to deflections. Or it may be necessary to modify soil properties to minimize the edge heave.

3.2.3 Moment. The moments can be calculated by statics, using the equivalent simple beam. The maximum moment will occur at the point of zero shear. Note that the maximum moment is quite sensitive to the beam length, therefore the iterative solution for deflection and appropriate swell pressure must converge accurately before calculating moments.

3.2.4 Shear. Shears can also be calculated by statics from the equivalent simple beam. Note that shears will reduce gradually to near-zero around the interior end of the beam because of the distributed soil support.

3.2.5 Special Cases. If no concentrated interior load exists, or if it is very far from the perimeter, the formula for the simple beam length must be adjusted as shown. This adjusted formula was also developed to duplicate results from computer solutions.

3.3 PERIMETER RIB.

3.4 DIAGONAL RIB.

3.5 INTERIOR RIB. Potential soil heaves in the interior are unpredicatable and are generally due to localized moisture conditions, for example, due to a leaking pipe. Such conditions cannot be accounted for by design forumlas. Adequate strength and stiffness for such unpredictable heaves should be supplied by the minimum requirements listed in Part I of the report. For interior wall or column loads the interior ribs should be designed in accordance with Part I, section 3.2.

FIGURE A1 - CENTER LIFT BEHAVIOR

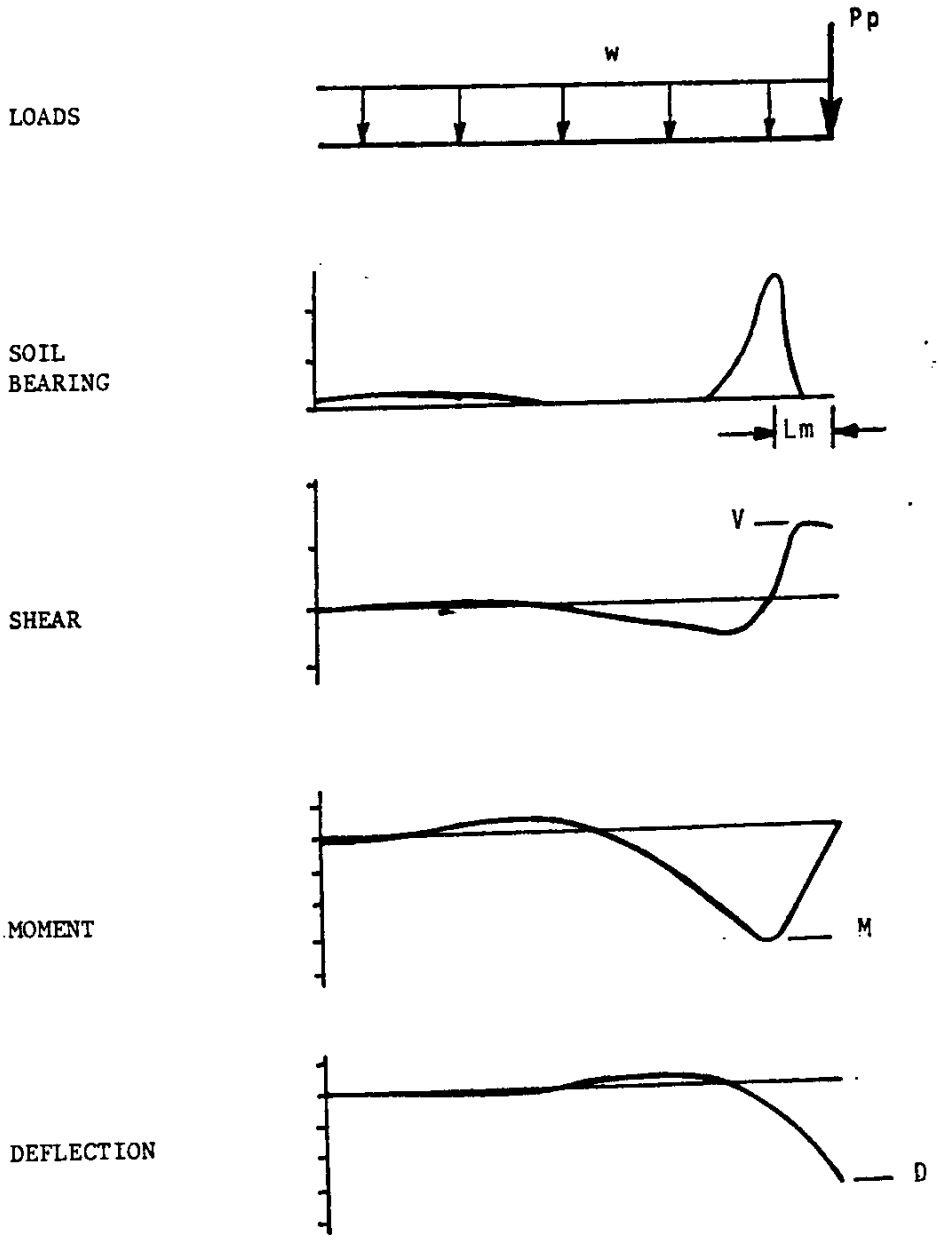
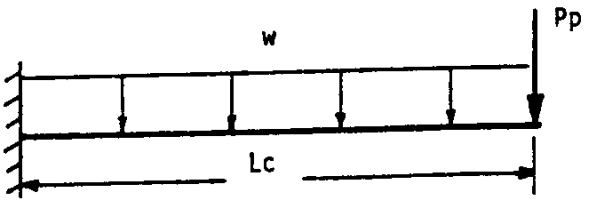
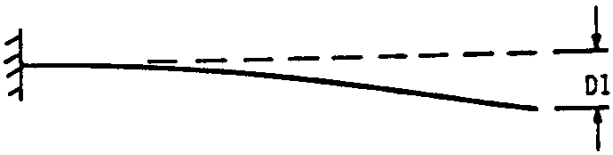


FIGURE A2 - CENTER LIFT DEFLECTION

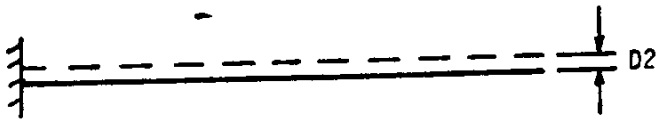
EQUIVALENT CANTILEVER



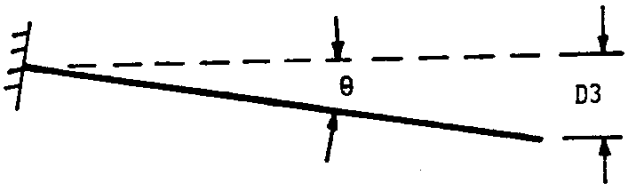
CANTILEVER BENDING



SUPPORT TRANSLATION



SUPPORT ROTATION



$$D = D1 + D2 + D3$$

$$D1 + D2 = .11$$

$$D3 = 12\ Lc\ \theta$$

FIGURE A3 - EDGE LIFT BEHAVIOR

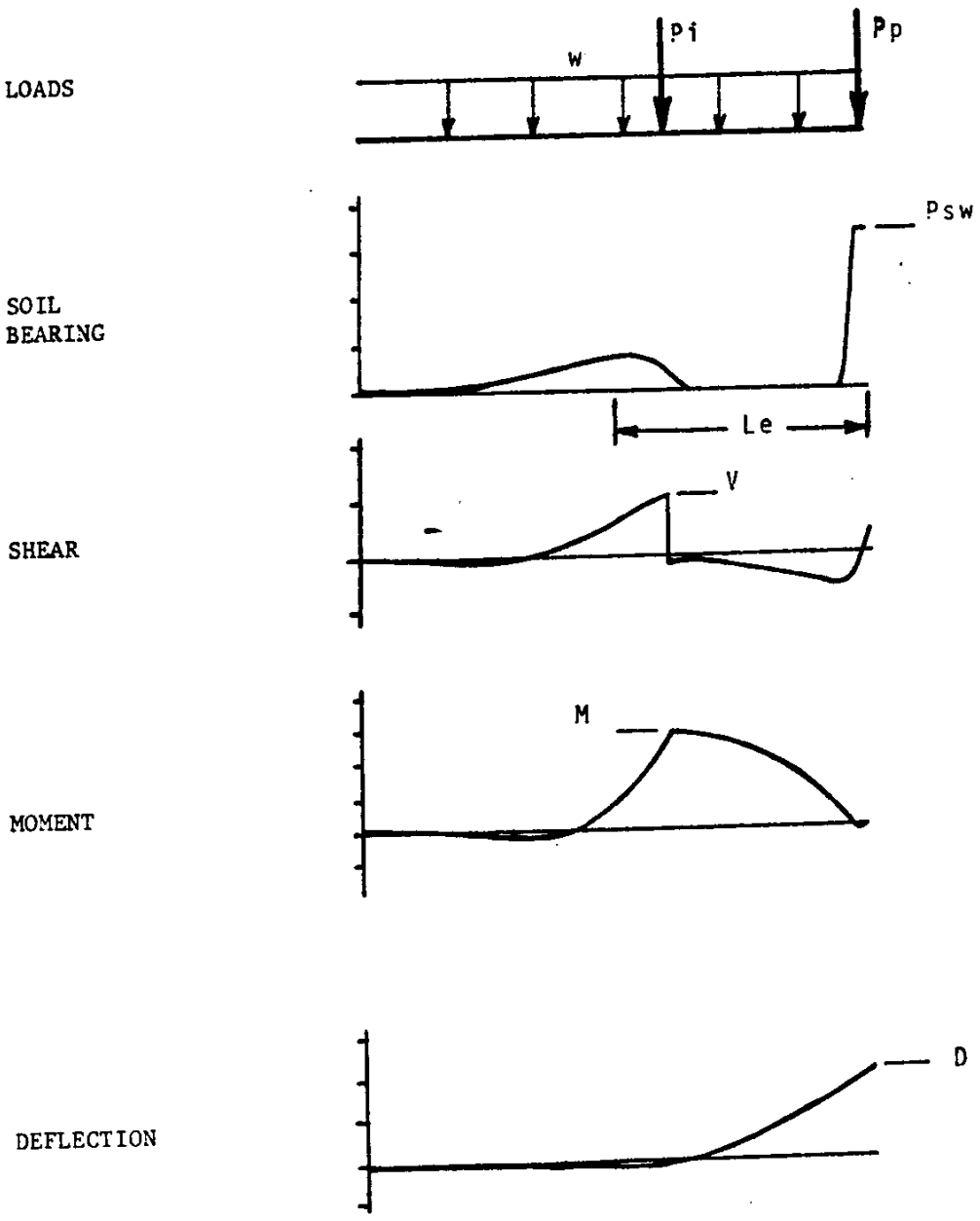
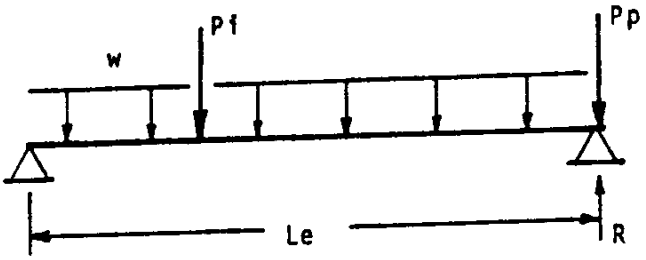


FIGURE A4 - EDGE LIFT DEFLECTION

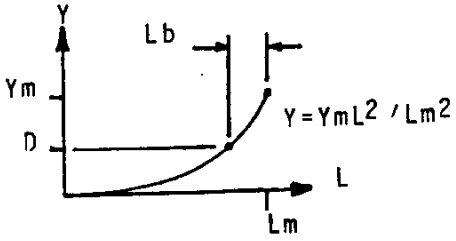
EQUIVALENT SIMPLE BEAM



DEFLECTED SHAPE

BEARING PRESSURE

SOIL EDGE PROFILE



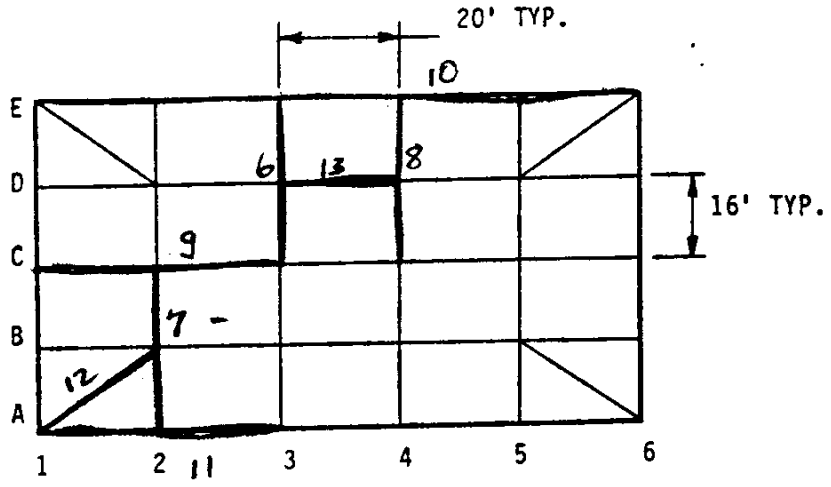
APPENDIX B - DESIGN EXAMPLE

(RIBBED MAT DESIGN IN EXPANSIVE SOIL)

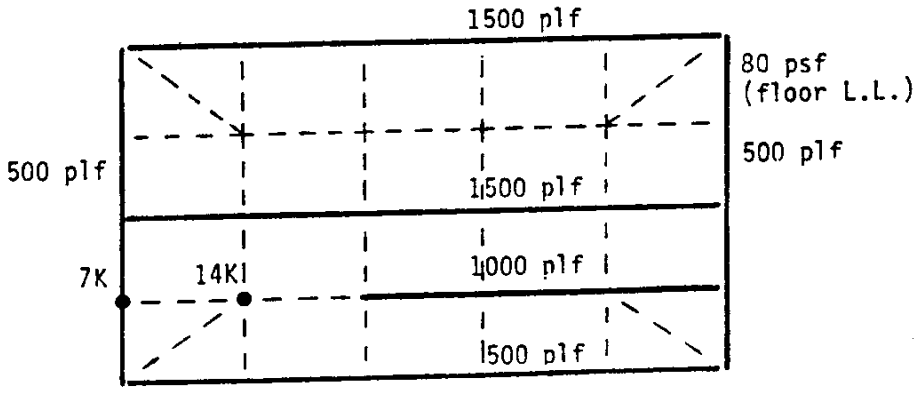
1. SOIL DATA (ref. Part I - 3.3)

qa = 2000 psf
Psw = (see page B9)
k = 100 pci
Lm = 6 ft
Ym = 1.5 in for center lift
Ym = 1.0 in for edge lift

2. FOUNDATION PLAN (ref. Part I - 4.3)



3. LOADS



4. BEARING DESIGN FOR RIBS (ref. Part I - 4.4)

Maximum wall load (P) = 1500 plf

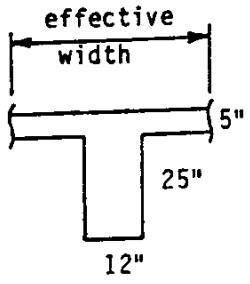
Width $\geq$ P/qa = 1500/2000 = .75 ft

Use 12 inch wide ribs (minimum)

5. INTERIOR RIB PROPERTIES (ref. Appendix A - 2.1)

Ec = 3,320,000 psi

(effective flange width per ACI 318, section 8.10.2 For "span length" use 4Lc for center lift or Le for edge lift)



Let Ir = 36,000 in^4 for center lift
Ir = 24,000 in^4 for edge lift
(ref. ACI 318, section 9.5.2.3, verify Ir after calculating M)

I = Ir/S (in^4/ft):

Rib spacing	16 ft	20 ft
Center lift	2250	1800
Edge lift	1500	1200

6. CENTER LIFT DESIGN - RIB E3/C3

6.1 Loads (ref. Appendix A - 2.1)

slab weight = 150 pcf x 5/12 ft = 62 psf

w = DL + LL = 62 + 80 = 142 psf

rib weight = 150 pcf x 2.5 ft x 1.0 ft = 375 plf

Pp = rib + wall = 375 + 1500 = 1875 plf

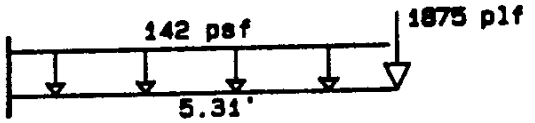
6.2 Equivalent cantilever (ref. Part II - 3.1)

$Lo = 2.3 + .4\ Lm = 2.3 + (.4 \times 6) = 4.7$ ft

$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$

$C = .8 \times 1.5^{.12} \times 1800^{.16} / 1875^{.12} = 1.13$

$Lc = Lo\ C = 4.7 \times 1.13 = 5.31$ ft



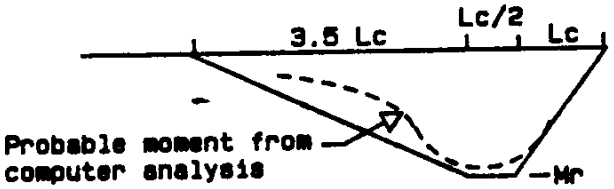
6.3 Moment (ref. Part II - 3.1.2)

$M = Pp\ Lc + 1/2\ w\ Lc^2$

$M = 1875 \times 5.31 + 1/2 \times 142 \times 5.31^2 = 12{,}000$ ft-lb/ft

$Mr = M \times S = 12000 \times 20 = 240{,}000$ ft-lb/rib

Design moments:

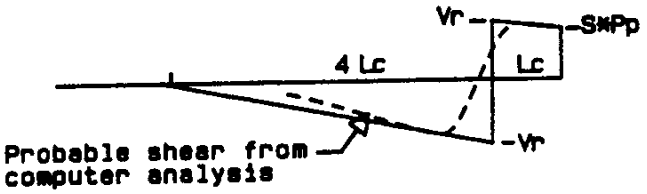


6.4 Shear (ref. Part II - 3.1.3)

$V = Pp + w\ Lc = 1875 + 142 \times 5.31 = 2630$ lb/ft

$Vr = V \times S = 2630 \times 20 = 52{,}600$ lb/rib

Design shears:



6.5 Reinforcing in rib (ref. Part I - 3.1.6 and 4.5)

As = (Mr/ad)/1.33

As = 240 / (1.76 x 28 x 1.33) = 3.66 in^2 (top)

use 3 #10 bars

v = Vr/bd = 52600 / (12 x 28) = 157 psi

vc = $(1.1\sqrt{f'c})1.33$ = 80 psi

Av = (v-vc)b s/(fs 1.33)

Av = (157-80) 12 x 12 / (24000 x 1.33) = .35 in^2/ft

use #4 stirrups @ 12 in

6.6 Deflection (ref. Part II - 3.1.4)

$\theta = M^{1.4} / 9800\ I\ k^{.5}$

$\theta = 12000^{1.4} / (9800 \times 1800 \times 100^{.5})$ = .0021 .0029 radians

D = .11 + 12 Lc θ = .11 + 12 x 5.31 x .0029 = .29 in

D/4Lc = .29 / (4 x 5.31 x 12) = 1/879 O.K.

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7. EDGE LIFT DESIGN - RIB A2/C2

7.1 Loads

w = 142 psf (same as above)

Pp = rib + wall = 375 + 500 = 875 plf

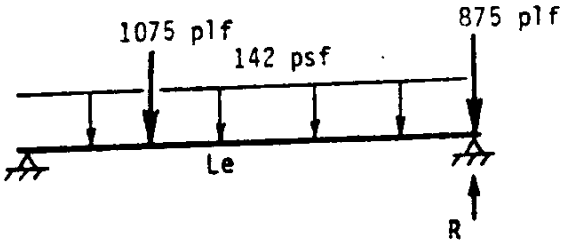
Pi = rib + wall* = 375 + 700 = 1075 plf

* equivalent wall load = column load / rib spacing

14000/20 = 700 plf (ref. Appendix A - 2.4)

Li = 16 ft

7.2 Equivalent simple beam (ref. Appendix A - 3.2.1)



7.3 Deflection (ref. Part II - 3.2.2)

$Le = 7.5\ I^{.17}\ Li^{.37}\ D^{.12}/\ w^{.07}\ Pi^{.11}$

$Le = 7.5 \times 1200^{.17} \times 16^{.37} \times D^{.12}/\ 142^{.07} \times 1075^{.11}$

$Le = 22.9\ D^{.12}$

assume D = .50 in (somewhat less than Ym = 1.0 in)

$Le = 22.9 \times .50^{.12} = 21.1$ ft

$R = Pp + 1/2\ w\ Le + Pi(Le-Li)/Le$

$R = 875 + (142 \times 21.1)/2 + 1075(21.1-16.0)/21.1 = 2633$ plf

from heave/pressure curve (p B9), for D=.50 find Psw=2000

$Lb = 1.1(R/Psw) = 1.1(2633/2000) = 1.45$ ft

$D = Ym(Lm-Lb)^2/Lm^2$

$D = 1.0(6.0-1.45)^2/6.0^2 = .575$ in $\neq$ .50 inch assumed!

assume D = .54 in

$Le = 22.9 \times .54^{.12} = 21.3$ ft

$R = Pp + 1/2\ w\ Le + Pi(Le-Li)/Le$

$R = 875 + (142 \times 21.3)/2 + 1075(21.3-16.0)/21.3 = 2655$ plf

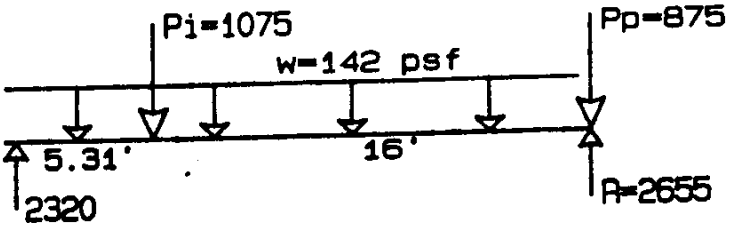
from heave/pressure curve, for D=.54 find Psw=1800 psf

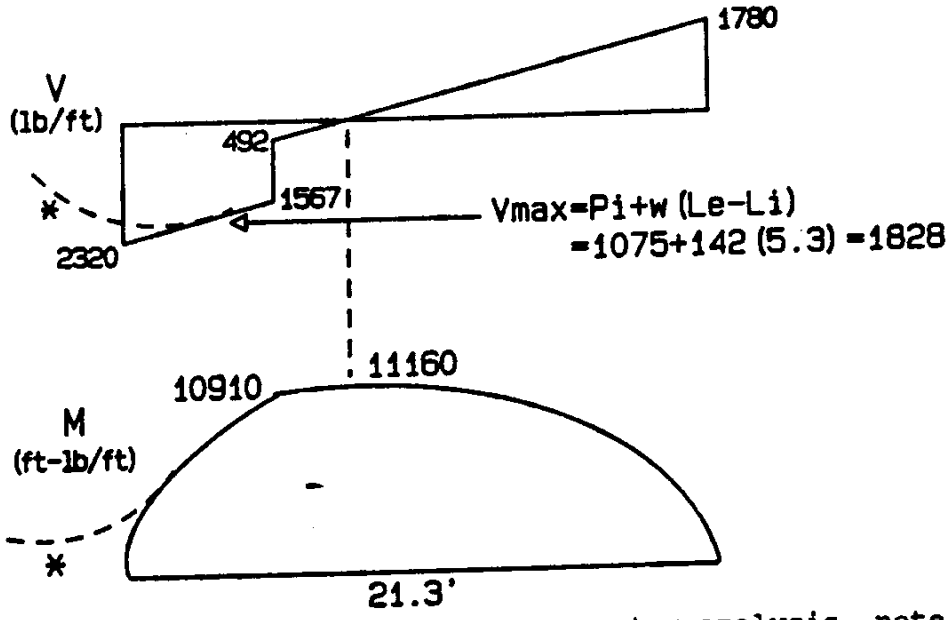
$Lb = 1.1(R/Psw) = 1.1(2655/1800) = 1.62$ ft

$D = 1.0(6.0-1.62)^2/6.0^2 = .533$ in CONVERGED!

D/Le = .54/(21.3x12) = 1/473 O.K. for non-brittle walls

7.4 Moment and shear (ref. Part II - 3.2.3 and 3.2.4)





* probable shear and moment from computer analysis, note that calculated V=2320 lb will not occur, due to the effects of distributed support from the soil

8. EDGE LIFT DESIGN - RIB E4/C4

8.1 Loads

w = 142 psf (same as above)

Pp = 1875 plf (same as rib E3/C3)

Li = 32 ft (wall along rib C1/C6)

8.2 Deflection

since Li>Le use:

$Le = 10.5\ I^{.17}\ D^{.12} / w^{.07}$ (ref. Part II - 3.2.5)

$Le = 10.5 \times 1200^{.17} \times D^{.12} / 142^{.07} = 24.77\ D^{.12}$

assume D = .48 in

then $Le = 24.77 \times .48^{.12} = 22.7$ ft

$R = Pp + 1/2\ w\ Le = 1875 + (142 \times 22.7)/2 = 3485$ plf

from heave/pressure curve, for D=.48 find Psw=2100 psf

$Lb = 1.1(R/Psw) = 1.1(3485/2100) = 1.825$ ft

$D = Ym(Lm-Lb)^2/Lm^2$

$D = 1.0(6.0-1.825)^2/6.0^2 = .484$ inch CONVERGED!

8.3 Find shears and moments by statics, similar to rib A2/C2.

9. CENTER LIFT DESIGN - RIB C1/C3

9.1 Loads

$w = slab + LL + wall^* = 62 + 80 + 94 = 236$ psf

* wall = wall load / rib spacing = 1500/16 = 94 psf
(ref. Appendix A - 2.4)

$Pp = rib + wall = 375 + 500 = 875$ plf

9.2 Equivalent cantilever

$Lo = 2.3 + .4\ Lm = 2.3 + (.4 \times 6) = 4.7$ ft

$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$

$C = .8 \times 1.5^{.12} \times 2250^{.16} / 875^{.12} = 1.28$

$Lc = Lo\ C = 4.7 \times 1.28 = 6.02$ ft

9.3 Moment

$M = Pp\ Lc + 1/2\ w\ Lc^2$

$M = 875 \times 6.02 + (236 \times 6.02^2)/2 = 9544$ ft-lb/ft

$Mr = M \times S = 9544 \times 16 = 153{,}000$ ft-lb/rib

9.4 Shear

$V = Pp + w\ Lc = 875 + (236 \times 6.02) = 2296$ plf

$Vr = V \times S = 2296 \times 16 = 36{,}700$ lb/rib

9.5 Deflection

$\theta = M^{1.4} / 9800\ I\ k^{.5}$

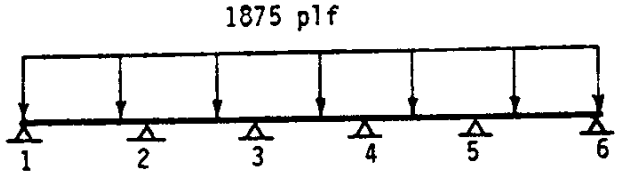
$\theta = 9544^{1.4} / 9800 \times 2250 \times 100^{.5} = .0017$ radian

$D = .11 + 12\ Lc\ \theta = .11 + (12 \times 6.02 \times .0017) = .23$ in

10. CENTER LIFT DESIGN - PERIMETER RIB E1/E6 (ref. Part II-3.3.1)

10.1 Span between transverse ribs

Pp = 1875 plf (from calculations for rib E3/C3)

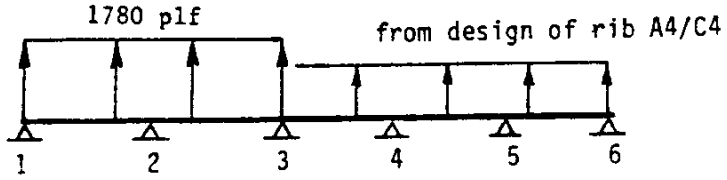


10.2 Analyze by conventional methods

11. EDGE LIFT DESIGN - PERIMETER RIB A1/A3 (ref. Part II - 3.3.2)

11.1 Span between transverse ribs for net upward force (from calculations on rib A2/C2)

R - Pp = 2655 - 875 = 1780 plf (upward)



11.2 Analyze by conventional methods

12. CENTER LIFT DESIGN - DIAGONAL RIB A1/B2 (ref. Part II - 3.4)

12.1 Provide the larger shear and moment capacity of rib B1/B2 or rib A2/B2.

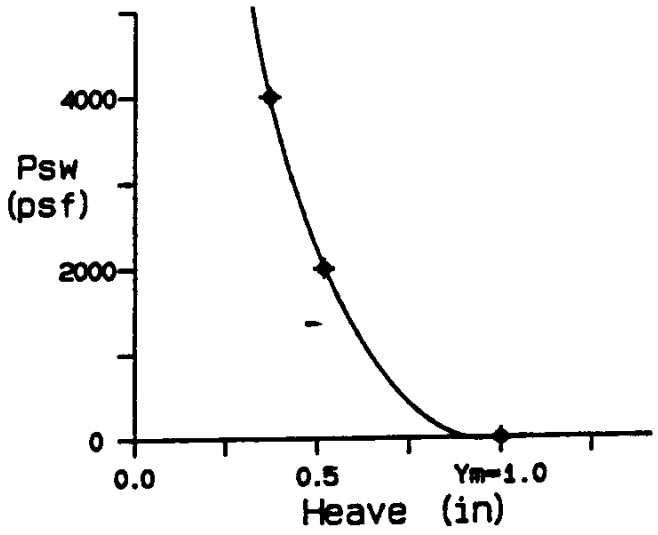
13. RIB D3/D4 (ref. Part I - 4.5)

13.1 Interior rib with no wall or column loads

$A_s \geq .0033\ A_g = .0033 \times 12 \times 30 = 1.20\ in^2$ (top and bot.)

This is the typical minimum reinforcement for the full length of all ribs.

14. HEAVE VERSUS SWELL PRESSURE CURVE (ref. Appendix A - 2.1)



DEPARTMENT OF THE ARMY
SOUTHWESTERN DIVISION, CORPS OF ENGINEERS
1114 COMMERCE STREET
DALLAS, TEXAS 75242-0216

REPLY TO
ATTENTION OF

16 APR 1987

SWDED-G

SUBJECT: Criteria for Developing Geotechnical Design Parameter for SWD Ribbed Mat Design Methodology

Commander, Albuquerque District, ATTN: SWAED-TA
Commander, Fort Worth District, ATTN: SWFED-F
Commander, Galveston District, ATTN: SWGED-G
Commander, Little Rock District, ATTN: SWLED-G
Commander, Tulsa District, ATTN: SWTED-G

1. Reference is made to criteria letter SWDED-TS/G dated 23 December 1986, subject "Design Criteria for Ribbed Mat Foundation".

2. The above reference criteria letter require certain geotechnical parameters be furnished in the Foundation Design Analysis when a ribbed mat slab foundation is recommended in expansive soil areas. Enclosure 1, for addressees only, provides guidance for development of these parameters. These procedures were developed by the Ft. Worth District with review in the Southwestern Division. Questions and/or comments should be directed to either Mr. A.L. Branch, FTS 334-2117 or Mr. Jack Fletcher, FTS 729-6365.

FOR THE COMMANDER:

William [signature]
for ARTHUR D. DENYS, P.E.
Chief, Engineering Division

Encl

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5. APPENDIX A

5.1 EXAMPLE PROBLEM

DEVELOPMENT OF GEOTECHNICAL DESIGN PARAMETERS FOR RIBBED MAT FOUNDATIONS

1. REFERENCE.

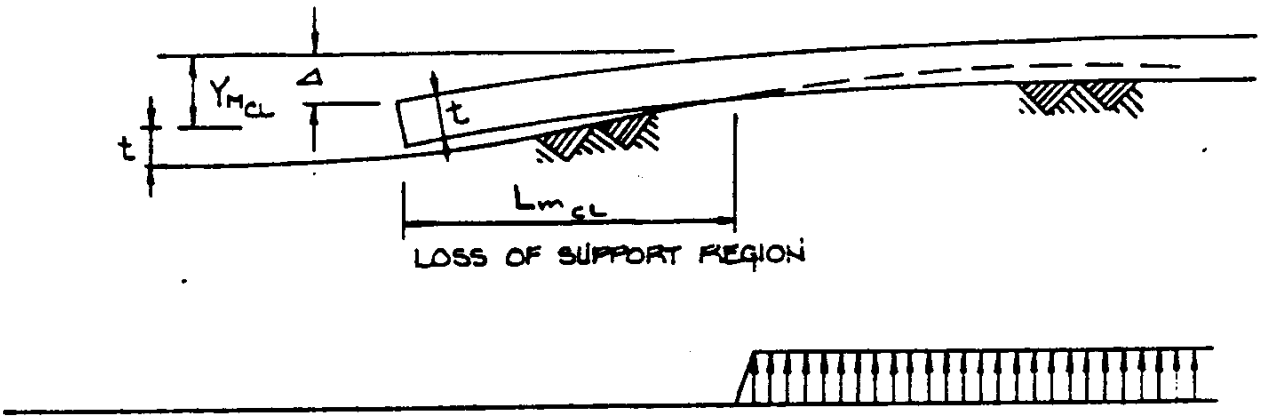
1.1 SWDED-TS/G, Design Criteria for Ribbed Mat Foundations, dated 23 Dec 86.

1.2 TM 5-818-7, Foundations in Expansive Soils, Corps of Engineers, 1983.

2. BACKGROUND. The recently developed structural design methodology (reference 1) models the interaction of a ribbed mat slab on an expansive subgrade for purposes of structural design. This method appears equally suited to stiffened mat systems such as flat mats, modified flat mats and inverted ribbed mats. Utilization of the methodology requires the expansion and refinement of the geotechnical design parameters furnished in the foundation design analysis. The purpose of this report is to (1) identify and (2) provide a rational method of determining these parameters.

3. SOIL-STRUCTURE INTERATION MODES. Two heave induced deformation conditions appropriate for ribbed mat slab structural analysis are (a) center lift and (b) edge lift.

3.1 CENTER LIFT. Center lift considers doming of the foundation in the interior region of a slab on grade differentially to the perimeter region as depicted on figure 1. This may be caused either by drying of the expansive subgrade around the perimeter beam or by wetting of the dry expansive subgrade in the interior region. Perimeter drying results from (1) below average precipitation and/or (2) reduced or no landscape watering and/or



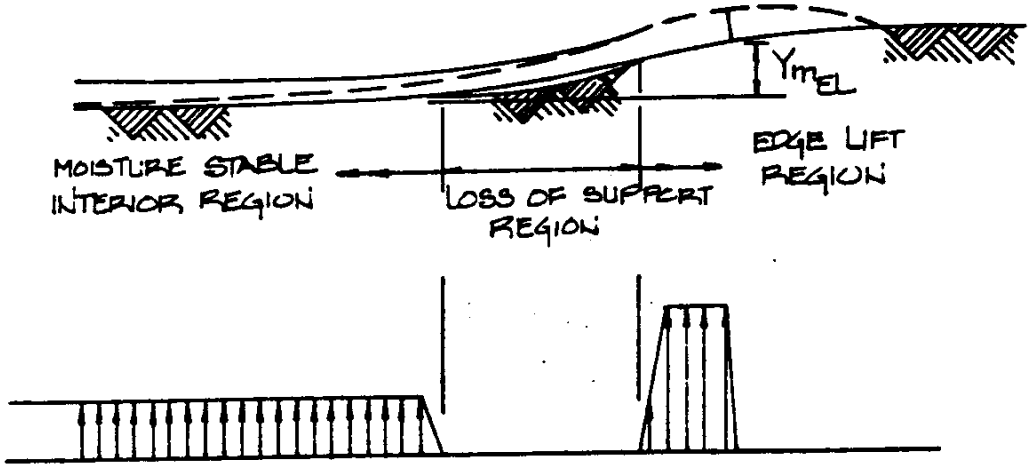
CENTER LIFT

FIGURE 1

(3) removal of old paving or hard stand. Interior wetting results from (1) disruption of the site moisture equilibrium by "capping" the site with the relatively impervious slab or by removal of thick brush or trees from the site (thus eliminating evapo-transportation) and/or (2) leaky inservice or abandoned utilities. Loss of support along perimeter and first interior transverse stiffener beam results if (1) the magnitude of center lift heave is large enough and (2) the beams are sufficiently rigid to cantilever from the supported interior region.

3.2 EDGE LIFT. Edge lift involves more complex soil-structure interactions than does center lift. In edge lift, the structure is supported by heaving subgrade in the perimeter region and in the relatively moisture stable interior region. Loss of support develops when (1) the edge lift heave deformation

is large enough and (2) the spanning beam is sufficiently rigid. Edge lift mode is depicted on figure 2.



EDGE LIFT
FIGURE 2

Soil-structure interaction within the interior supported region is reasonably represented as a beam on non-linear subgrade. Soil-structure interaction in the perimeter region is somewhat more complex because the soil deflects under the structural load as a beam on non-linear subgrade, but also the swelling soil either loads and/or deflects the beam upward. To further complicate matters, the amount of edge lift heave and the soil-beam interface pressure are interrelated and unique for each specific site. Background parameter studies for reference 1 indicate that the structural analyses are particularly sensitive

to edge lift parameters (edge lift heave magnitude and limiting beam-soil interface pressure). For example, large values for these may cause the solution to either fail to converge or indicate that the beam must be very deep and/or very heavily reinforced. While site conditions may sometimes dictate massive, very rigid stiffener beams, this is not generally the case. Generally, edge lift heave of less than 1.0 to 1.5 inches used in the design method given in reference 1 produce reasonable, constructable beams.

4. DETERMINATION OF CENTER LIFT AND EDGE LIFT PARAMETERS FOR STRUCTURAL DESIGN.

4.1 CENTER LIFT - Center lift parameters to be provided in the foundation design analysis includes (1) modulus of subgrade reaction (K_1), (2) design allowable bearing for beams (q_{all}), (3) magnitude of center lift (Y_{mcL}) and (4) loss of support distance around the perimeter (L_{mcL}).

4.1.1 MODULUS OF SUBGRADE REACTION - The modulus of subgrade reaction should be taken as K_1 = 200 pci for beams up to 12 inches wide bearing on compacted, nonexpansive fill. Higher values may be justified for granular nonexpansive fills consisting of gravel, crushed rock or limestone screenings or for cement stabilized materials if these materials extend significantly ($D \geq 3B$) below the stiffener beam of width B. The foundation design analysis should direct that K_1 values be factored to account for width effects such that $K_{design} = K_1/B$, where B is the effective beam width in feet for soil structure interaction. Note that the resultant effective beam width may include a significant width of the slab and is therefore

significantly greater than actual beam width. Studies indicate that significant load distribution occurs over an "effective" width of approximately five. It should be noted that structural design calculations are not sensitive to K value.

4.1.2 DESIGN ALLOWABLE BEARING. A design allowable bearing value (q_{all}) has historically been assigned for sizing of stiffener beams, perimeter beams and enlarged beam intersections beneath columns. Values are typically given considering the beam to be a continuous strip footing or the beam intersection to be a spot footing (carrying either line or concentrated loads, respectively). The allowable bearing value is typically developed based on the average strength of engineered fill at shallow depth with a factor of safety of not less than 3.0. Design loads typically include full dead load plus half live load. The purpose in sizing the beams and beam intersections for this design allowable is to provide uniform contact pressures at the beam-soil interface therefore limiting inservice differential settlement. The assumptions of minimal load sharing between the slab and beams, ample safety factor on the fill strength, and minimum beam widths specified in the SWD EIM combine to limit the mobilized soil strains to low levels. This leads to very small structurally induced deflections given uniform, nominal fill depths. Actual values assigned for design bearing allowables have seldom exceeded q_{all} = 2.0 KSF although values as high as 3.0 KSF have been assigned in limited cases where required and justifiable. Seldom are there structural requirements for larger allowables bearing values since specified minimum beam widths generally govern.

4.1.3 MAGNITUDE OF CENTER LIFT HEAVE POTENTIAL. - The magnitude of center lift heave potential (Y_{mcL}) given in the foundation design analysis should be the residual heave potential at the site. The value of Y_{mcL} should include effects due to subgrade removal and replacement criteria, any surcharge effects due to fill above original subgrade and the weight of the proposed structure. Maximum design value for center lift potential should not exceed 1.5 inches. Where attainable with reasonable removal/replacement depths ($\leq$36 inches), it is desirable to limit Y_{mcL} to not more than 1.0 inch, which is well within the "tolerable" inservice deformation range of most structures. Minimum remove/replace depth should be taken to the bottom elevation of the ribbed mat slab beams.

The heave potential is determined by three soil parameters: the coefficient of swell (C_s), depth of active zone (X_a) and expansion pressure (P_{exp}).

Caution should be used in selecting coefficient of swell (C_s) values for heave analyses since swell pressure test results significantly underestimate C_s values compared to controlled expansion-consolidation-rebound tests. Additionally, both test methods tend to give low C_s values since most rebound time curves are terminated well before primary swell is completed.

An appropriate design value of the depth of the active zone (X_a) typically lies between the present depth to the stable relative moisture content (estimated by observing the relationship of moisture content to the plastic limit) and the maximum depth observed, such as the maximum depth of weathering. Typical X_a values for the central and north Texas regions and

central Oklahoma region appear to vary from about 10 to 15 feet. These values have been estimated for (1) regression heave analyses for distressed structures and (2) depth of moisture variation versus approximate return/duration interval studies. Values smaller than 14 feet may be applicable in specific cases such as where the active zone is the distance between the structural foundation element or slab on grade and a perched water table; a condition common in these regions.

Center lift heave analyses should consider "saturated" conditions to a depth of X_a. If a nominal remove/replace depth and saturated subgrade assumptions indicate unreasonable residual heave potential, consider increasing the depth of remove/replace and/or recommending a more defensive design to prevent saturation of the subgrade.

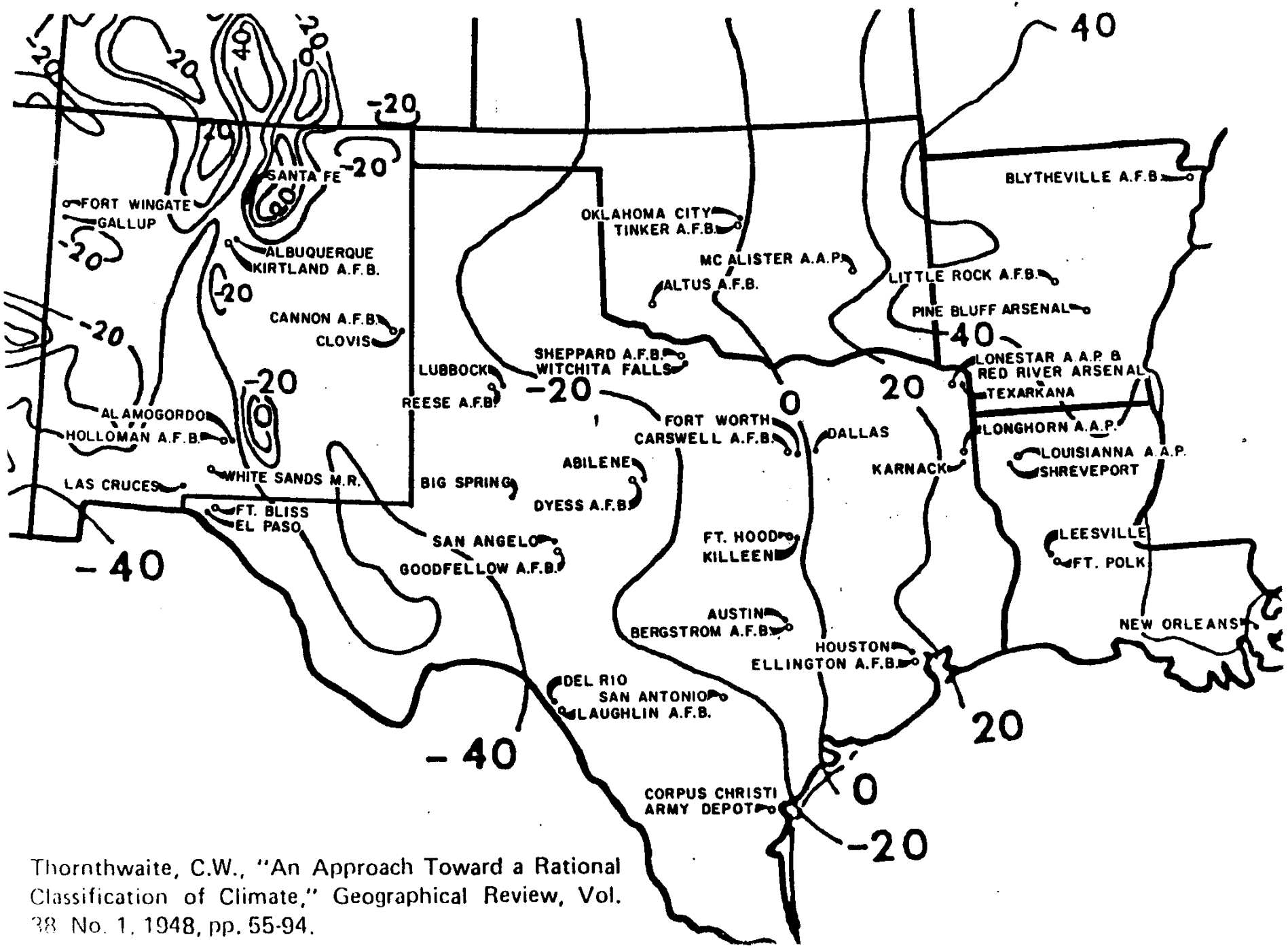
Expansion pressures should be developed versus depth using small depth intervals. These should be developed from laboratory data for the site. Additionally, these may be supplemented using proper correlations with nearby, preferably adjacent sites.

4.1.4 **EDGE MOISTURE VARIATION DISTANCE**. The edge moisture variation distance (L_{mcL}) may control the design of interior stiffener beams which are adjacent to the perimeter. The maximum moments and shear are induced in the transverse beams when these elements cantilever free of foundation support from the interior supported region to the outside of the perimeter beam. The length of cantilver is largely controlled by the value of L_{mcL}. SWD adopted this concept from Post-Tensioning Institute (PTI) guidelines, originally developed for lightly loaded flexible mats in the late 1970's and early 1980's. Standard practice in the

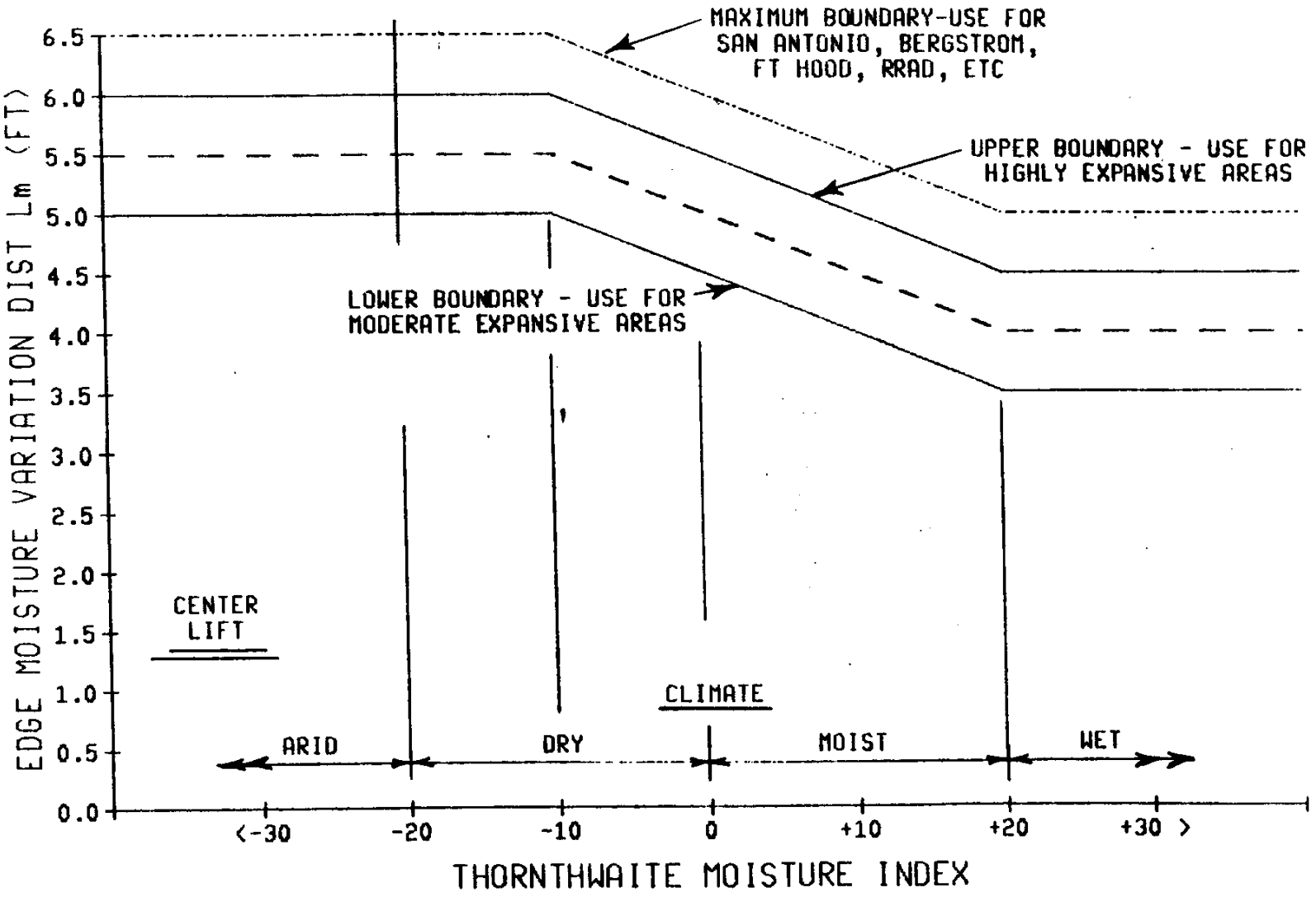
San Antonio area has been to assign upper or near upper bound values from TMI for design L_{mCL} values. At least two aspects of designs probably tend to moderate the actual edge moisture variation distance experienced; these being (1) relatively deep perimeter beams which act as a physical barrier and (2) the non-expansive fill blanket which tends to make changes in moisture content (and therefore any resultant heave or shrinkage) more uniform and provide a surcharge effect as well. Other factors, however, tend to offset these moderating effects. These include very short return interval of edge moisture variation events presented in TMI (reported by some sources to range from 1 to 2 years). Typical project design life of projects exceeds 20 or 30 years and, since we're still using many World War II facilities, it may well exceed 50 years. Estimated edge moisture variation values considering a 100 percent probability of experiencing a 20 to 30-year return interval event may well be twice typical TMI values.

Based on a subjective combination of all factors, it is suggested that L_{mCL} be taken as the edge moisture variation distance determined using figures 3 and 4. These values should be modified, either up or down, based on site secific soils investigations and engineering judgement.

4.2 EDGE LIFT - Edge lift parameters to be provided in the foundation design analysis include (1) modulus of subgrade reaction (K_1), (2) magnitude of edge lift heave (Y_{mEL}), (3) limiting soil-beam interface pressure (P_{sw}) for that portion of the beam being acted on by the heaving subgrade and (4) a value for edge moisture variation distance (L_{mEL}).



Thornthwaite, C.W., "An Approach Toward a Rational Classification of Climate," Geographical Review, Vol. 38 No. 1, 1948, pp. 55-94.



APPROXIMATE RELATIONSHIP BETWEEN THORNWAITE INDEX AND MOISTURE VARIATION DISTANCE

FIGURE 4

4.2.1 MODULUS OF SUBGRADE REACTION. - Values given for center lift are considered appropriate for edge lift also. $K_1 = 200$ pci

4.2.2 SOIL-BEAM INTERFACE PRESSURE. Discussion of both limiting soil-beam interface pressure and magnitude of edge lift heave parameters (Psw and Y_{mEL}) are best handled concurrently since both are intimately related and the analysis necessary for solution determines both simultaneously.

The area of soil-beam contact in the swelling perimeter region involves a somewhat complex soil-structure interaction situation. As edge lift develops and loss of support occurs between the perimeter and interior regions, the heaving soil may well exert a pressure on the stiffener beams well in excess of typical design interface pressures (q_{all}). As the soil column swells and lifts the overlying beam, the soil-beam contact area increases toward the interior region to accommodate the greater structural reaction.

The soil-structure interaction in the edge lift region can be visualized as a three-component system: (1) a structural element (a beam or mat strip), (2) an element of nonexpansive fill beneath the structural element plus that piece of the expansive subgrade restrained against heave by the weight of the overlying fill and the stresses induced beneath the structural element, and (3) the heaving column of soil to a depth of X_a beneath the bottom of the nonexpansive fill blanket (figure 5).

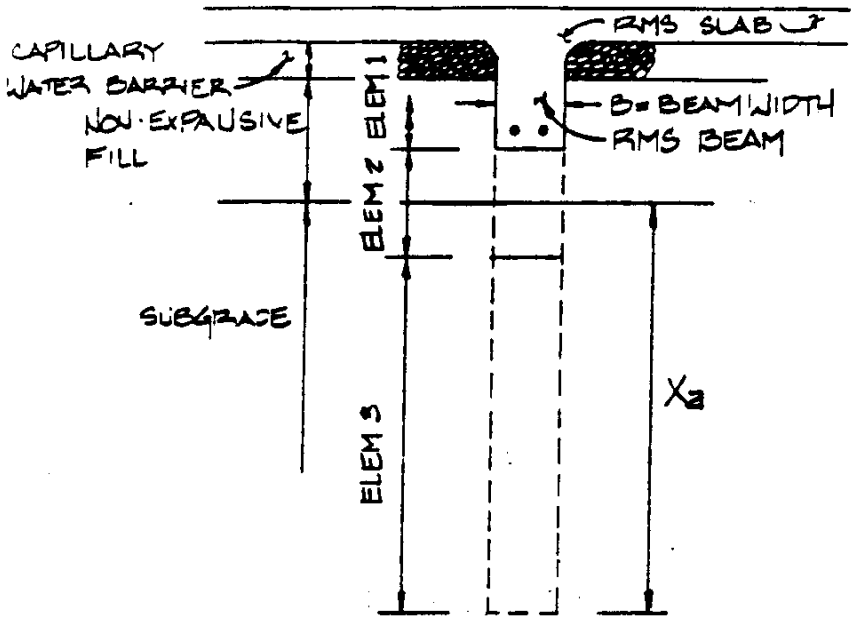


FIGURE 5

The load-deformation relationship of element 1 interacting with element 2 can be represented by a P-Y curve shown in figure 6.

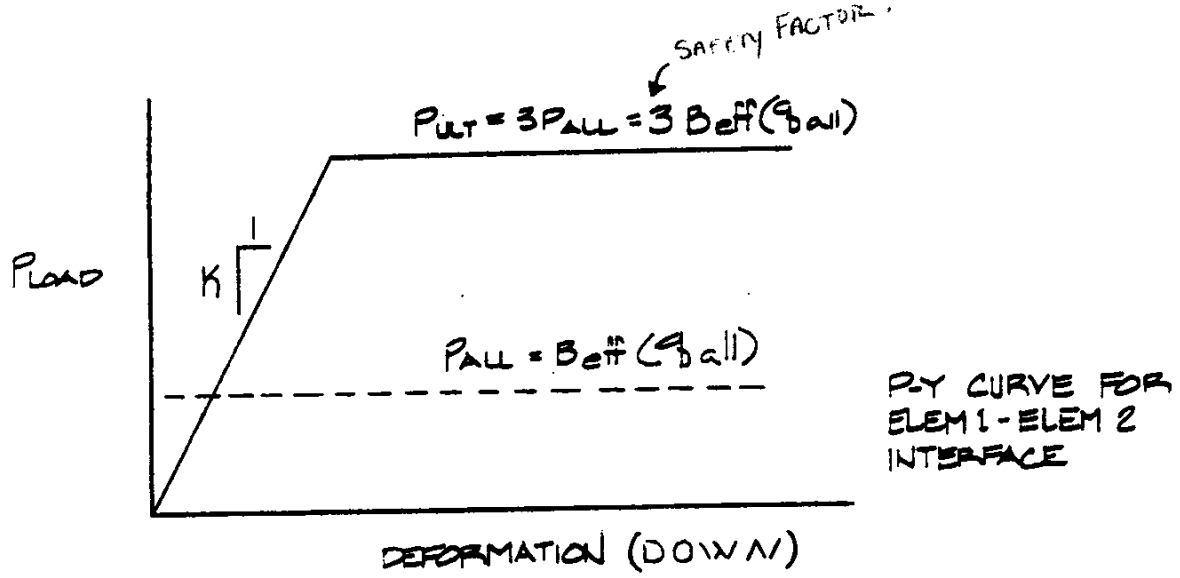


FIGURE 6

The load-deformation relationship of element 3 interacting with elements 1 and 2 in the column immediately below the beam as shown on figure 7. The plot consists of the net heave potential of the swelling soil column versus those forces resisting the tendency to swell, taken at the base of the structural beam.

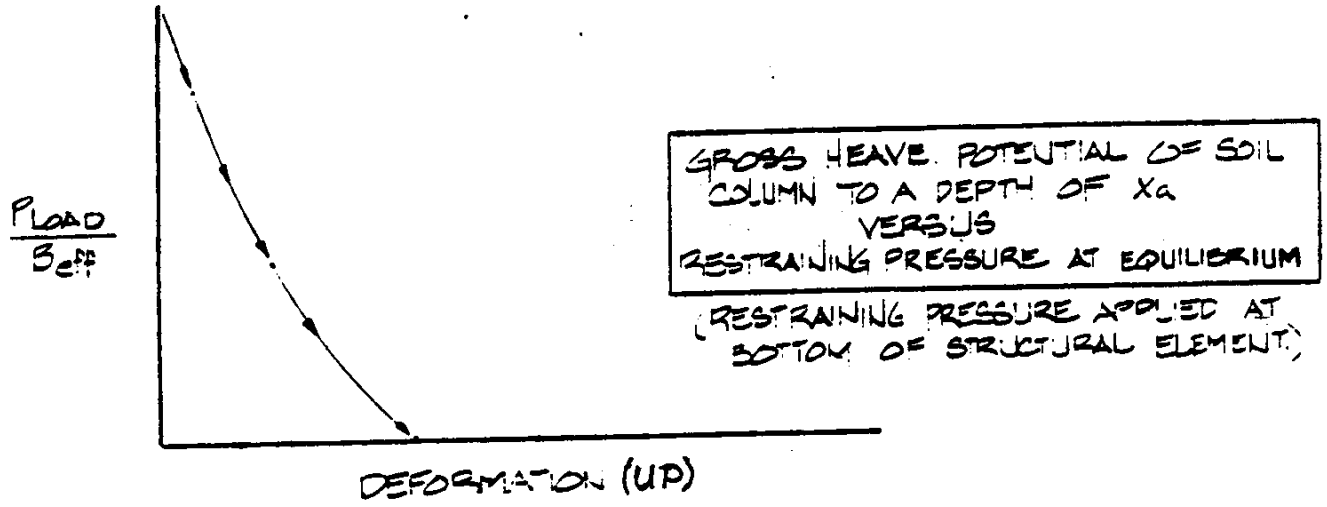
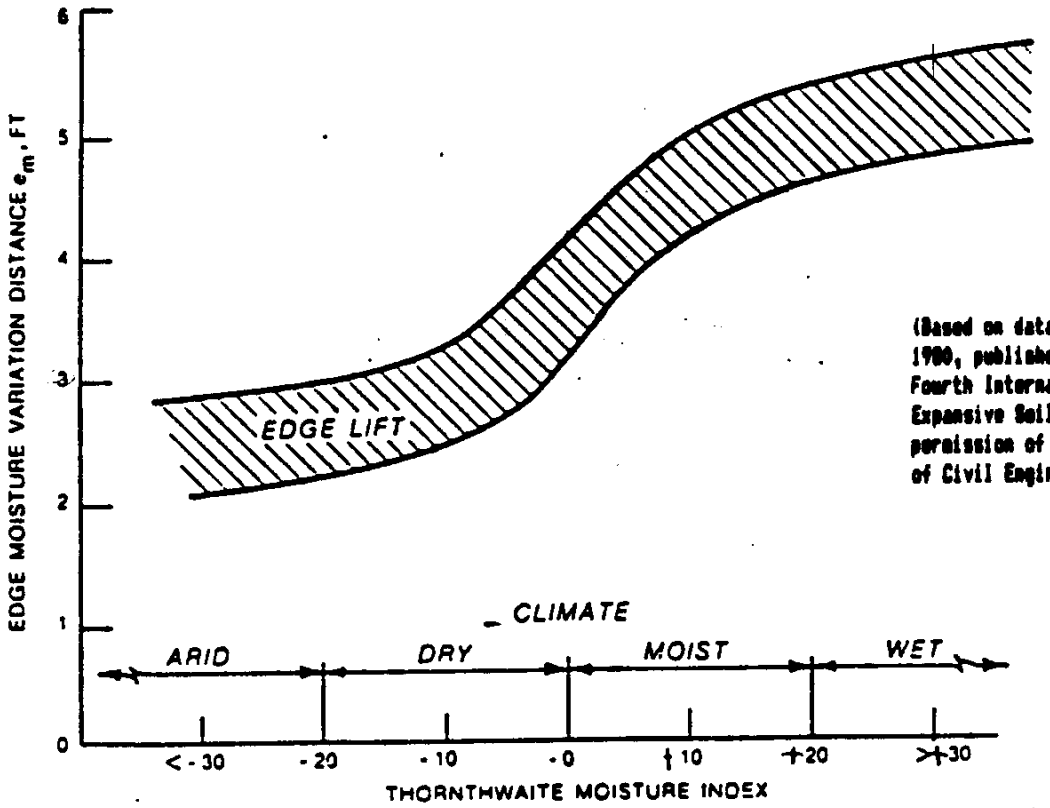


FIGURE 7

These relationships can be added algebracially to produce a composite p-y curve which can be easily utilized by available soil-structure interaction programs for structural analysis. Since such analysis is within the purview of the structural engineer, the geotechnical engineer need only furnish the pressure heave relationship in useable form in the Foundation Design Analysis. It is suggested that this information be provided in a tabulated format giving coordinates for at least three points. These minimum three points should be the Psw and Yme coordinates for (1) pressure equal to Pult, (2) pressure equal to Pall and (3) pressure equal to zero.

Pult = F.S. x Pallow

4.2.3 EDGE MOISTURE VARIATION DISTANCE. Edge moisture variation distance (L_{mL}) appropriate for edge lift analysis may be taken from the TMI chart given in figure 8.



(Based on data from W. K. Wray, 1980, published in Proceedings, Fourth International Conference on Expansive Soils, Vol. 1, with permission of the American Society of Civil Engineers)

Approximate relationship between the Thornthwaite Moisture Index and the edge lift-off distance.

FIGURE 8

The TMI values represent approximate environmentally induced events. As a result, upper bound values should be selected for design. It is recommended, however, that average values be used for all SWD projects. Additionally, recommendations should be made in the foundation design analysis to limit the potential for developing "hot spots" due to long term sources of free water around the building perimeter.

function of the type of bldg, ie, Brick stucco or not

4.3 Certain structure-site situations may well warrant deleting edge lift analyses as follows:

4.3.1 Where the proposed structure is a pre-engineered metal building without interior masonry walls or heavy interior dead or permanent live loads.

4.3.2 Where defensive design efforts have been incorporated and reasonable confidence exists that these will be constructed and maintained as intended.

4.3.3 Where minor architectural distress (such as cracking of masonry walls, plaster walls, tiled surfaces) is not likely to cause undue user concern or raise inservice maintenance requirements significantly.

5. APPENDIX A

5.1 EXAMPLE PROBLEM. An example problem is provided in Appendix A.

APPENDIX A

EXAMPLE PROBLEM

1. Required. - Develop geotechnical parameters for the structural design of a ribbed mat slab given the following:

a. Proposed Structure. - Office/Administration type structure located in San Antonio, Texas, 60X150 feet in plan. The structure is to consist of double wythe masonry (face brick over CMU) load bearing exterior walls and isolated interior colums at 20 ft. centers.

b. Proposed Site. - One acre, minimal topographic relief, site covered with mesquite trees.

c. Subsurface Conditions. - Drilling program (5 borings) indicates the foundation materials consist of (1) a surface stratum of high plasticity clay grading into medium plasticity clay with depth to a total thickness of 14 feet, (2) a water bearing sand and gravel stratum from 1 to 7 feet thick overlying, (3) an expansive clay shale formation.

d. Summary Laboratory Test Data. -

Stratum	Depth (ft)	USCS	$\bar{W}_o$ (%)	γ_d (pcf)	LL	PI	P_{exp} (tsf) (net)	C_s	C_c	Cu (ts
1	0-4	CH	25	105	65	45	0.8 -1.0	0.06	0.02	0.
2	4-14	CL	14	108	44	30	0.6	0.06	0.18	0.
3	14-20	GC	6	-	25	12	0	-	-	5C B/
4	20 plus	Wea. Clay Shale	22	110	70	52	2.0	0.09	0.22	1.

2. Determine Parameters Required for Center Lift Analysis:

a. Modulus of Subgrade Reaction (K_1). - Mat slab will be founded on nonexpansive fill, therefore it is reasonable to assign a valve of K_1 = 200 PCI. The structural engineer should factor this value based on effective beam width such that $K_{design} = K_1$ (1ft/Beff, ft).

b. Design Bearing Allowable (q_{all}). - Since beams will be supported on nonexpansive fill and the building loads will range from light to moderate, it appears that a design bearing allowable of q_{all} = 2.0 KSF is appropriate.

c. Magnitude of Center Lift Heave Potential (Y_{mcl}). -

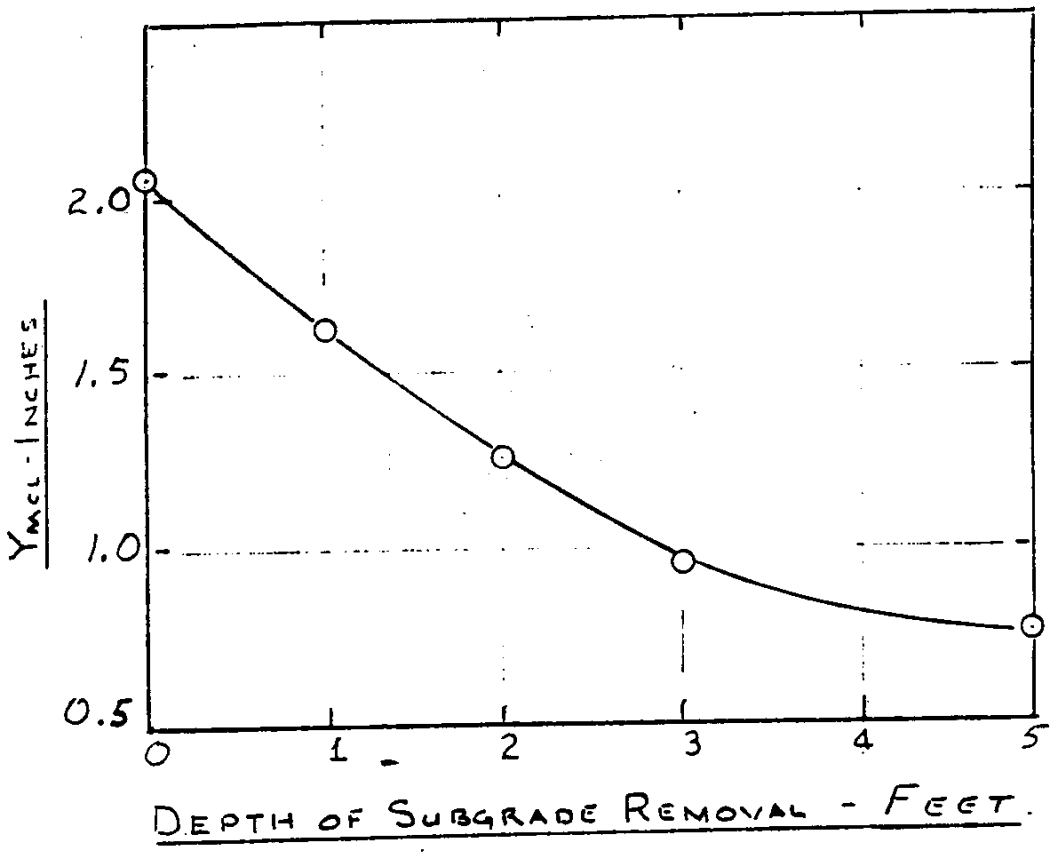
(a) Calculate site heave potential

Given: C_s = 0.06, e_o = 0.60, P_o = effective overburden pressure, P_{srp} = gross swell pressure, P_f = effective pressure resisting heave beneath mat including P_o, (surcharge due to fill and structural dead load, h_i = heave for soil layer h inches thick, and an X_a = 10 feet.

$C_s = .06$
$e_o = .60$

z (ft)	z (ft)	Po (tsf)	Pexp (tsf)	Pf (tsf)	h (in)	$h_i = \frac{C_{sh}}{1+e_o} \log_{10} \frac{P_{exp}}{P_f}$ (inches)	h_i (bottom to top) (inches)
0-1	0.5	0.03	1.0	0.1 (.07+.03)	12	0.45	2.07
1-2	1.5	0.1	1.0	0.17 (.07+.1)	12	0.35	1.62
2-3	2.5	0.17	1.1	0.24 (.07+.17)	12	0.30	1.27
3-4	3.5	0.23	0.8	0.3	12	0.19	0.97
4-6	5	0.33	0.9	0.4	24	0.31	0.78
6-8	7	0.46	1.0	0.53	24	0.25	0.47
8-10	9	0.6	1.2	0.67	24	0.22	0.22
10-12	11	0.73	1.35	0.8	24	0.2	N/A
12-14	13	0.86	0.9	0.93	24	N/A	N/A

Determine required depth of subgrade replacement and residual heave potential after replacement with nonexpansive fill. A plot of replacement depth versus residual heave taken from the above table follows:



Removal and replacement to 3.0 feet will reduce the heave potential to approximately 1.0 inch, thus Y_{mc1} = 1.0 inch. Note that significant additional removal would be required to reduce the residual heave potential any sigificant additional amount.

d. Edge Moisture Variation Distance (L_{mc}) - taken from figures 3 and 4 as L_{mc} = 6.5 feet.

3. Determine parameters required for Edge Lift analyses:

a. Modulus of Subgrade Reaction (K_1). - Same as for Center Lift.

b. Design Allowable Bearing (q_{all}). - Same as for Center Lift.

c. Soil - Beam Interface Pressure (P_{sw}) and Magnitude of Edge Lift Heave Potential (Y_{me}). -

Determine the residual heave potential for the soil column beneath a typical beam for a range of assumed interface pressures.

A summary of calulations and results is presented in tabulated form on page 6. A plot of soil-beam interface pressure versus heave potential is shown on page 7. A reasonable bilinear representation of the results, for use by the structural engineer, can be developed assuming a linear relationship between the following points:

P_{sw}, TSF	Y_{me}, Inches
0.0	Y_{me} = 1.25
q_{all} = 1.00	Y_{me} = 1.0
q_{ult} = 3(q_{all}) = 3.00	Y_{me} = 0.6

d. Edge Moisture Variation Distance, (L_{me}). The edge moisture variation distance is taken from figure 8 as L_{me} = 3.0 feet.

Given: D_f = 2.0ft B = 1.0ft

C_s = 0.06 e_o = 0.60

γ_m = 125pcf

Z ft	$\bar{Z}$ ft	h in	$\frac{\bar{Z}-D_f}{B}$	I_s	P_o tsf	P_o+P_s tsf	P_{exp} tsf	q_{app} = 0.5tsf: ΔP tsf	P_f tsf	Δh_i in	q_{app} = 1.0tsf: ΔP tsf	P_f tsf	Δh_i in	q_{app} = 2.0tsf: ΔP tsf	P_f tsf	Δh_i in	q_{app} = 4.0tsf: ΔP tsf	P_f tsf	Δh_i in
0-3	<----------COMPACTED NONEXPANSIVE FILL TO A DEPTH OF 3.0 FT.---------->																		
3-4	3.5	12	1.5B	0.28	0.23	0.30	0.8	0.14	0.37	0.15	0.28	0.51	0.08	0.56	0.79	0	1.12	1.35	0
4-6	5.0	24	3.0B	0.15	0.33	0.40	0.9	0.07	0.40	0.32	0.15	0.48	0.25	0.30	0.63	0.14	0.60	0.93	0
6-8	7.0	24	5.0B	0.09	0.46	0.53	1.0	0.05	0.53	0.25	0.09	0.55	0.23	0.18	0.64	0.17	0.36	0.82	0.09
8-10	9.0	24	7.0B	0.07	0.60	0.67	1.2	0.04	0.67	0.23	0.07	0.67	0.23	0.14	0.74	0.19	0.28	0.88	0.12
10-12	11.0	24	9.0B	0.05	0.73	0.80	1.35	0.03	0.80	0.20	0.05	0.80	0.20	0.1	0.83	0.2	0.20	0.93	0.16
12-14	13.0	24	11.0B	0.04	0.83	0.90	0.9	0.02	0.90	0.0	0.04	0.90	0.0	0.08	0.91	0	0.16	0.99	0
								$\Sigma\Delta h_i$ = 1.15			$\Sigma\Delta h_i$ = 0.99			$\Sigma\Delta h_i$ = 0.7			$\Sigma\Delta h_i$ = 0.38	

Where:

Z = depth interval

$\bar{Z}$ = mean depth

B = beam width

D_f = beam depth

γ_z = stress with depth

P_o = overburden pressure

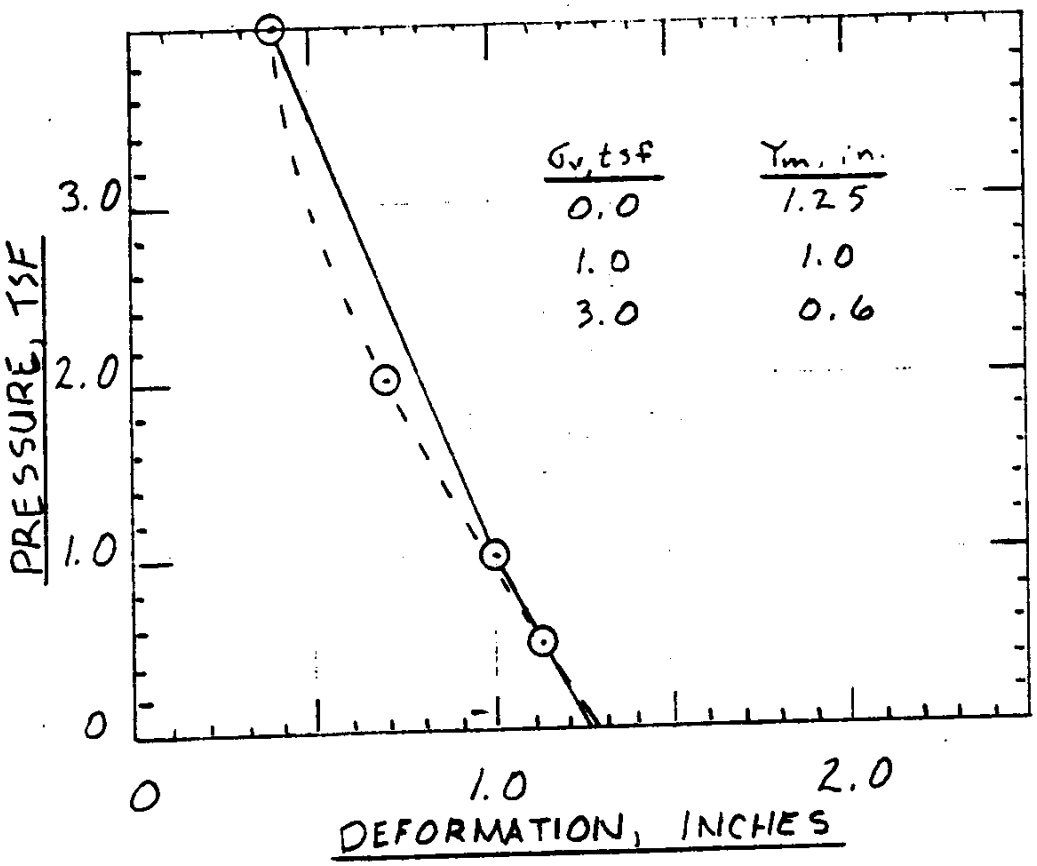
P_s = surcharge pressure next to beam

P_{exp} = expansion pressure

P_f = vertical pressure resisting heave below beam

P = stress @ depth due to to q_{app} $\Delta P = (I_s)(q_{app})$

P_f = { $\Delta P + P_o$ or $P_o + P_s$ | whichever is greater }



EDGE LIFT PARAMETER

EXAMPLE PROBLEM