Questions:

1. Drawings sheet AD101-9, Keynote Legend DM01 says “Remove and dispose of modular structure…”. The Excel file says “Loto Building Removal and Relocation on site”. Please clarify
   A: Please bid as if it was going to be relocated on site, if it is deemed unsafe we will need to revise the cost for haul off instead of relocating.

2. Does the slab we are removing have any interior/exterior beams? How deep?
   A: We have not received as-builts for the building to be demolished. The existing slab to be demolished is assumed to be a slab-on-grade with no beams. The depth of the existing slab-on-grade is unknown.

3. Is the contractor to hire a testing lab to conduct the testing requirements shown on S-004-0?
   A: The International Building Code (IBC) requires that the owner or owner’s authorized agent, other than the contractor, shall employ one or more approved agencies to provide special inspections and tests in accordance with the requirements of the code.

4. Do you have a Geotechnical Report?
   A: We do not have a geotechnical report unique to this project. The geotechnical report used for the foundation design is from a past project on this site: "Geotechnical Engineering Study for Rio Nogales GSU Spare Foundation" dated March 9, 2020 (attached).

5. Who is the Fire Protection subcontractor to the system we are tying into?
   A: Western State is the current Fire protection System it will be tying into.

6. There is a 2” gap in between the two buildings. How we fill this gap at the new door 101B? and at the ends of the building?
   A: Refer to details 3 & 4 on sheet A-351.

7. Is there an expansion joint in between the existing slab and the new slab?
   A: Yes, there is an expansion joint. We are not doweling into the existing slab or foundations.

8. Is there and expansion joint between the new slab and the existing site concrete?
   A: Yes, there is an expansion joint. We are not doweling into the existing site concrete slab.

9. What is MISC (Utilities) on the excel file?
   A: Please disregard and use the attached bid sheet.

10. Per Structural notes says to excavate 3’ beyond slab. How do we finish this 3’? Provide detail.

    A: Per ASI/Revision 1, the excavation 3'-0" beyond slab extents was removed from the scope.

GEOTECHNICAL ENGINEERING STUDY

FOR

RIO NOGALES GSU SPARE FOUNDATION
SEGUIN, TEXAS

RABA KISTNER

12821 W Golden Lane
San Antonio, TX 78249

PO Box 690287
San Antonio, TX 78269

P 210.699.9090
F 210.699.6426
TBPE Firm F-3257

WWW.RKCI.COM

Project No. ASA19-128-00
March 9, 2020

Mr. Sid Mielke, P.E.
Structural Engineering Associates, Inc.
3838 NW Loop 410
San Antonio, Texas 78229

RE: Geotechnical Engineering Study
Rio Nogales GSU Spare Foundation
Seguin, Texas

Dear Mr. Mielke:

RABA KISTNER Consultants Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKCI Proposal No. PSA19-120-00, dated December 10, 2019. The purpose of this study was to drill borings within the proposed transformer pad location, to perform laboratory testing to evaluate and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the proposed transformer pad, as well as to provide paving guidelines for heavy hauling equipment to access the site during construction.

The following report contains our design recommendations and considerations based on our current understanding of the information provided to us. There may be alternatives for value engineering of the foundation system, and RKCI recommends that a meeting be held with the Owner and design team to evaluate these alternatives.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance with value engineering or on the materials testing-quality control program during construction, please call.

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

Isaac Molina, P.E.
Project Engineer

R. Blake Wright, P.E.
Project Manager

STATE OF TEXAS
R. BLAKE WRIGHT
117753
LICENSED
PROFESSIONAL ENGINEER

RBW/IM/kv

Attachments

Copies Submitted: Above (Electronic)

GEOTECHNICAL ENGINEERING STUDY

For

RIO NOGALES GSU SPARE FOUNDATION
SEGUIN, TEXAS

Prepared for

STRUCTURAL ENGINEERING ASSOCIATES, INC.
San Antonio, Texas

Prepared by

RABA KISTNER CONSULTANTS, INC.
San Antonio, Texas

PROJECT NO. ASA19-128-00

March 9, 2020

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The following figures are attached and complete this report:

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Important Information About Your Geotechnical Engineering Report

PROJECT DESCRIPTION

RABA KISTNER Consultants Inc. (RKCI) has completed the authorized subsurface exploration for the proposed transformer pad to be located at the Rio Nogales Power Station complex in Seguin, Texas, as illustrated on Figure 1. The proposed structure is anticipated to be a reinforced concrete mat similar to existing foundations at the facility. The mat foundation will have two pedestals, each capable of supporting a separate transformer. The project will also provide paving guidelines for heavy hauling equipment to access the site during construction. This report briefly describes the procedures utilized during this study and presents our findings along with our geotechnical recommendations for the proposed improvements. Site grading plans have not been provided at this time.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of south/central Texas and for the use of Structural Engineering Associates, Inc. (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods. The attachments and report text should not be used separately.

The recommendations submitted in this report are based on the data obtained from two borings drilled at this site and our understanding of the project information provided to us. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

If final grade elevations are significantly different than those discussed herein (more than plus or minus 1 ft), our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by 2 borings and 3 dynamic cone penetrometer (DCP) tests shown on the Boring Location Map, Figure 1. These locations are approximate and distances were measured using a hand-held, recreational-grade GPS locator. Boring elevations, as annotated on our boring logs, were estimated using the referenced drawing. The 2 borings were drilled to depths ranging from 30 to 40 ft below the existing ground surface using a truck-mounted drilling rig. During drilling operations split-spoon (with standard penetration test) and relatively undisturbed Shelby tube samples were collected at the depths annotated on our boring logs.

Each sample was visually evaluated in the laboratory by a member of our geotechnical engineering staff. The geotechnical engineering properties of the strata were evaluated by natural moisture content, Atterberg limits, and consolidation tests; as well as grain size analyses.

The laboratory test results are presented in graphical or numerical form on the boring logs illustrated on Figures 2 and 3. A key to classification terms and symbols used on the logs is presented on Figure 4. The results of the laboratory and field testing are also tabulated on Figure 5 for ease of reference. DCP test results are presented on Figure 6 and consolidation test results are presented on Figure 7.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the Client.

GENERAL SITE CONDITIONS

GEOLOGY

A review of the Geologic Atlas of Texas, San Antonio Sheet, indicates that this site is naturally underlain with soils of the Leona Formation. The Leona Formation is associated with terrace deposits of the Nueces and Leona Rivers and typically consists of clays/silts grading down into coarse gravel and cobbles. The Leona Formation can be highly variable and can therefore result in highly variable conditions over relatively short distances. Key geotechnical engineering concerns for development supported on the Leona Formation are the expansive nature of the clays, the consistency and/or relative density of the deposits, and the absence/presence as well as thickness of potentially water-bearing gravels.

SEISMIC CONSIDERATIONS

The following information has been summarized for seismic considerations associated with this site per ASCE 7-16 edition.

- Site Class Definition: **Class C**. Based on the soil borings conducted for this investigation and our experience in the area, the upper 100 ft of soil may be characterized as very dense soil and soft rock.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% Of Critical Damping): $\mathbf{S_s = 0.052g}$.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% Of Critical Damping): $\mathbf{S_1 = 0.027g}$.
- Values of Site Coefficient: $\mathbf{F_a = 1.3}$
- Values of Site Coefficient: $\mathbf{F_v = 1.5}$
- Where g is the acceleration due to gravity.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec, adjusted: $\mathbf{S_{ms} = 0.068g}$
- 1 sec, adjusted: $\mathbf{S_{m1} = 0.041g}$

The Design Spectral Response Acceleration Parameters (SA) are as follows:

- 0.2 sec SA: $\mathbf{S_{DS} = 0.045g}$
- 1 sec SA: $\mathbf{S_{D1} = 0.027g}$

STRATIGRAPHY

On the basis of our recent borings, the subsurface stratigraphy at this site can generally be described as dark brown clay soils overlying reddish-brown clay with gravel to depth of 23 to 25 ft, which in turn is underlain by tan marl, then tan gravel with varying amounts of clay. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by RKCI in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time may result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

GROUNDWATER

Groundwater was not observed in the borings either during or immediately upon completion of the drilling operations. All borings remained dry during the field exploration phase. However, it is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly following periods of precipitation. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

FOUNDATION RECOMMENDATIONS

Please note that the foundation capacities presented herein are based on the Allowable Stress Design methodology. In general, the allowable values given herein for foundations can be increased by 33 percent for wind or other transitory loads (2018 IBC, Section 1806.1).

SITE GRADING

Based on our understanding of the project, site grading requirements will be minimal for this project. However, site grading plans can result in changes in almost all aspects of foundation recommendations.

We have prepared foundation recommendations based on the existing ground elevations and the stratigraphic conditions encountered at the time of our study. If site grading is performed and differs from the existing grades by more than plus or minus 1 ft, RKCI must be retained to review the site grading plans prior to bidding the project for construction. This will enable RKCI to provide input for any changes in our original recommendations that may be required as a result of site grading operations or other considerations.

EXPANSIVE SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at the site were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). PVR values of 3-3/4 in. were estimated for the stratigraphic conditions encountered in our borings. A surcharge load of 1 psi (concrete slab and sand cushion), an active zone of 15 ft, and dry moisture conditions were assumed in estimating the PVR value.

PVR values of less than 3/4 in. were estimated using a surcharge load of 24 psi (3,500 psf to account for the structural loads of the transformer on the pedestal). PVR values of 1-1/3 in. were calculated around the pedestal in the containment area when expansive soils are removed by overexcavating and backfilling with suitable select material to an elevation of 552 ft msl. If PVR values of 1 in. are required outside the pedestal area, overexcavation and backfilling with suitable select fill material should extend to an elevation of 551 ft msl.

Spare GSU Storage Foundation (FFE 556 ft msl)	
Depth of Overexcavation and Select Fill Replacement (ft) (1)	**Estimated PVR with Limestone Base Select Fill(2) (in.)**
556	3-3/4
553	1-3/4
552	1-1/3
551	1
550	Less than 1

(1)Below the ground surface elevation existing at the time of our study
(2)Material requirements are provided in the *Select Fill* section of this report.

The TxDOT method of estimating expansive soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the calculated PVR values due to isolated changes in moisture content (such as due to leaks, landscape watering, etc.) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

Overexcavation and Select Fill Replacement

We recommend that the overexcavation extend a minimum of 3 ft beyond the proposed structure footprints and sloped up towards the adjacent grade. Where the potential for differential movement is objectionable (particularly at the entry ways), it may be beneficial to consider methods of reducing anticipated movements (i.e. sloping the pad overbuild excavation at a slope at 1 vertical to 1 horizontal, or flatter). To maintain the estimated PVR values, subsequent fill placed in the improvement area should consist of select fill material in accordance with the *Select Fill* section of this report.

The overexcavated plastic soils may be reused on site as general fill but must be placed outside of the proposed pad. Plastic soils may be used as general fill provided that the potential vertical movements in excess of those discussed previously will not adversely impact either the structural or operational tolerances for the proposed improvements for which this material is being considered.

Drainage Considerations

When overexcavation and select fill replacement is selected as a method to reduce the potential for expansive soil-related movements at any site, considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structure. Filling an excavation in relatively impervious plastic clays with relatively pervious select fill material creates a “bathtub” beneath the structure, which can result in ponding or trapped water within the fill unless good surface and subsurface drainage is provided.

Water entering the fill surface during construction or entering the fill exposed beyond the structure lines after construction may create problems with fill moisture control during compaction and increased access for moisture to the underlying expansive clays both during and after construction.

Several surface and subsurface drainage design features and construction precautions can be used to limit problems associated with fill moisture. These features and precautions may include but are not limited to the following:

- Installing berms or swales on the uphill side of the construction area to divert surface runoff away from the excavation/fill area during construction;
- Sloping of the top of the subgrade with a minimum downward slope of 1.5 percent out to the base of a dewatering trench located beyond the structure perimeter;
- Sloping the surface of the fill during construction to promote runoff of rain water to drainage features until the final lift is placed;
- Sloping of a final, well maintained, impervious clay or pavement surface (downward away from the structure) over the select fill material and any perimeter drain extending beyond the structure, with a minimum gradient of 6 in. in 5 ft;
- Constructing final surface drainage patterns to prevent ponding and limit surface water infiltration at and around the structure perimeter;
- Locating the water-bearing utilities, roof drainage outlets and irrigation spray heads outside of the select fill and perimeter drain boundaries; and
- Raising the elevation of the ground level floor slab.

Details relative to the extent and implementation of these considerations must be evaluated on a project-specific basis by all members of the project design team. Many variables that influence fill drainage considerations may depend on factors that are not fully developed in the early stages of design. For this reason, drainage of the fill should be given consideration at the earliest possible stages of the project.

ENGINEERED SLAB ON SELECT FILL PAD

An engineered slab on a select fill pad is expected to provide the most economical and practical foundation system for this project. The proposed structure may be founded on a mat foundation or an engineered beam and slab foundation, provided the selected foundation type can be designed to withstand the anticipated soil-related movements (see *Expansive Soil-Related Movements*) without impairing either the structural or the operational performance of this structure.

Allowable Bearing Capacity

If overexcavation and select fill replacement (as recommended in the *Expansive Soil-Related Movements* section) is performed to an elevation of 552 ft msl or lower, the parameters in the following table may be utilized.

Net Allowable Bearing Capacity Parameters	
Minimum depth below final grade	18 in.
Minimum beam / strip footing width	12 in.
Minimum widened beam / spread footing width	18 in.
Maximum allowable bearing pressure for grade beams or strip footings bearing on select fill	3,500 psf

The above presented maximum allowable bearing pressures will provide a factor of safety of about 3, provided that fill is placed as discussed herein and the subgrade is prepared in accordance with the recommendations outlined in the *Site Preparation* section of this report.

Modulus of Subgrade Reaction

A Modulus of Subgrade Reaction (k) of 37 pci may be used for design of the transformer pad founded on a 2 foot thick, minimum, select fill foundation pad provided that the foundation pad is constructed in strict accordance with the recommendation presented in this report.

Uplift Resistance

Resistance to vertical force (uplift) is provided by the weight of the concrete footing plus the weight of the soil directly above the footing. For this site, it is recommended that the ultimate uplift resistance be based on total unit weights for soil and concrete of 125 pcf and 150 pcf, respectively. The calculated ultimate uplift resistance should be reduced by a factor of safety of 1.2 to calculate the allowable uplift resistance.

Lateral Resistance

Horizontal loads acting on spread footings will be resisted by passive earth pressure acting on one side of the footing and by base adhesion for footings in cohesive soils. Resistance to sliding for foundations bearing on natural/compacted soil or select fill can be calculated utilizing an ultimate coefficient of friction of 0.30. The ultimate resistance for these foundations should be limited to 750 psf. An equivalent fluid pressure of 250 pcf can be utilized to determine the ultimate passive resistance, if required.

FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

Drainage is an important key to the successful performance of any foundation. Good surface drainage should be established prior to and maintained after construction to help prevent water from ponding within or adjacent to the structure foundation and to facilitate rapid drainage away from the structure. Failure to provide positive drainage away from the structure can result in localized differential vertical movements in soil supported foundations.

Current ordinances, in compliance with the Americans with Disabilities Act (ADA), may dictate maximum slopes for walks and drives around and into new structures. These slope requirements can result in drainage problems for structures supported on expansive soils. We recommend that, on all sides of the structure, the maximum permissible slope be provided away from the structure.

Where a select fill overbuild is provided outside of the foundation footprint, the surface should be sealed with an impermeable layer (pavement, geomembrane, or clay cap) to reduce infiltration of surface waters. Careful consideration should also be given to the location of water bearing utilities, as well as to provisions for drainage in the event of leaks in water bearing utilities. All leaks should be immediately repaired.

Other drainage and subsurface drainage issues are discussed in the *Expansive Soil-Related Movements* section of this report and under *Pavement Construction Considerations*.

SITE PREPARATION

Foundation areas and all areas to support select fill should be stripped of all existing base material, vegetation, and organic topsoil. Furthermore, as discussed in a previous section of this report, overexcavation and select fill replacement be utilized to reduce expansive soil-related movements.

Exposed subgrades should be thoroughly proofrolled in order to locate and densify any weak, compressible zones. A minimum of 5 passes of a fully-loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or his representative to document subgrade condition and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with suitable, compacted on-site clays, free of organics, oversized materials, and degradable or deleterious materials.

Upon completion of the proofrolling operations and just prior to fill placement or slab construction, the exposed subgrade should be moisture conditioned by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density determined by ASTM D698, Compaction Test. The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum moisture content until permanently covered.

SELECT FILL

Materials used as select fill for site grading preferably should be crushed stone or gravel aggregate.

> **Imported Crushed Limestone Base** – Imported crushed limestone base materials should be should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Type A or B, Grades 1-2 or 3.

If the above-listed material is being considered for bidding purposes, the material should be submitted to the Geotechnical Engineer for evaluation at a minimum of 10 working days or more prior to the bid date. Failure to do so will be the responsibility of the contractor.

Soils classified as CH, MH, ML, SM, GM, OH, OL and Pt under the USCS are not considered suitable for use as select fill materials at this site.

Select Fill Placement and Compaction Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by ASTM D1557. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction for imported crushed limestone base.

SHALLOW FOUNDATION EXCAVATIONS

Shallow foundation excavations should be observed by the Geotechnical Engineer or their representative prior to placement of reinforcing steel and concrete. This is necessary to observe that the bearing soil at the bottom of the excavations are similar to those encountered in our borings and that excessive loose materials and water are not present in the excavations. If soft soils are encountered in the foundation excavations, they should be removed and replaced with a compacted non-expansive fill material or lean concrete up to the design foundation bearing elevations.

EXCAVATION SLOPING AND BENCHING

If utility trenches or other excavations extend to or below a depth of 5 ft below construction grade, the contractor or others shall be required to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

EXCAVATION EQUIPMENT

Our boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that earth-work and utility contractors interested in bidding on the work perform their own tests in the form of test pits to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

UTILITIES

Utilities which project through slab-on-grade, slab-on-fill, or any other rigid unit should be designed with either some degree of flexibility or with sleeves. Such design features will help reduce the risk of damage to the utility lines as vertical movements occur. These types of slabs will generally be constructed as monolithic, grid type beam and slab foundations.

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches. It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

PAVEMENT CONSTRUCTION CONSIDERATIONS

Based on the project information provided to us, we understand an 8 in. flexible base section is planned for a permanent access road and parking. This section is sufficient for the intended used of this access road during construction and also for occasional use for maintenance and other operations.

SUBGRADE PREPARATION

The exposed subgrade should be prepared with the recommendations in the *Site Preparation* section under *Foundation Construction Considerations*.

DRAINAGE CONSIDERATIONS

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
2) Final site grading should eliminate isolated depressions which may allow surface water to pond and infiltrate into the underlying soils.
3) Pavement surfaces should be maintained to help minimize surface ponding. These measures will help reduce infiltration of surface water downward through the pavement section.

ONSITE SOILS (PAVEMENTS)

As discussed previously, the pavement recommendations presented in this report were prepared assuming that on-site soils will be used for fill grading in proposed pavement areas. If used, we recommend that on-site soils be placed in loose lifts not exceeding 8 in. in thickness and be compacted to at least 95 percent of maximum dry density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of optimum water content to 3 percentage points above the optimum water content until permanently covered. We recommend that fill materials be free of roots and other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to TxDOT 2014 Standard Specifications, Item 247, Type A or B, Grade 1-2, or 3. Base course should be placed in lifts with a maximum thickness of 8 in. and compacted to a minimum of 95 percent of the maximum dry density at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E, or 98 percent of maximum density as determined by ASTM D698.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, RKCI is retained to perform construction observation and testing services during the construction of the project. This is because:

- RKCI has an intimate understanding of the geotechnical engineering report's findings and recommendations. RKCI understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- RKCI knows what subsurface conditions are anticipated at the site.
- RKCI is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables RKCI to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- RKCI has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- RKCI cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKCI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKCI looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

* * * * * * * * * * * * * * * * * *

ATTACHMENTS

RABA KISTNER

12821 West Golden Lane
San Antonio, TX 78249

(210)699-9090 TEL
(210)699-6426 FAX

www.rkci.com

TBPE Firm Number 3257

SOURCE: Aerial photograph obtained from Google Earth Pro - 2018

BORING LOCATION MAP

RIO NOGALES GSU SPARE FOUNDATION
SEGUIN, TEXAS

PROPOSAL No.:	ASA19-128-00
ISSUE DATE:	03/09/2020
DRAWN BY:	JMR/KRB
CHECKED BY:	RBW
REVIEWED BY:	EJN

FIGURE

1

NOTE: This Drawing is Provided for Illustration Only, May Not be to Scale and is Not Suitable for Design or Construction Purposes

LOG OF BORING NO. B-1

Rio Nogales GSU Spare Foundation
Seguin, Texas

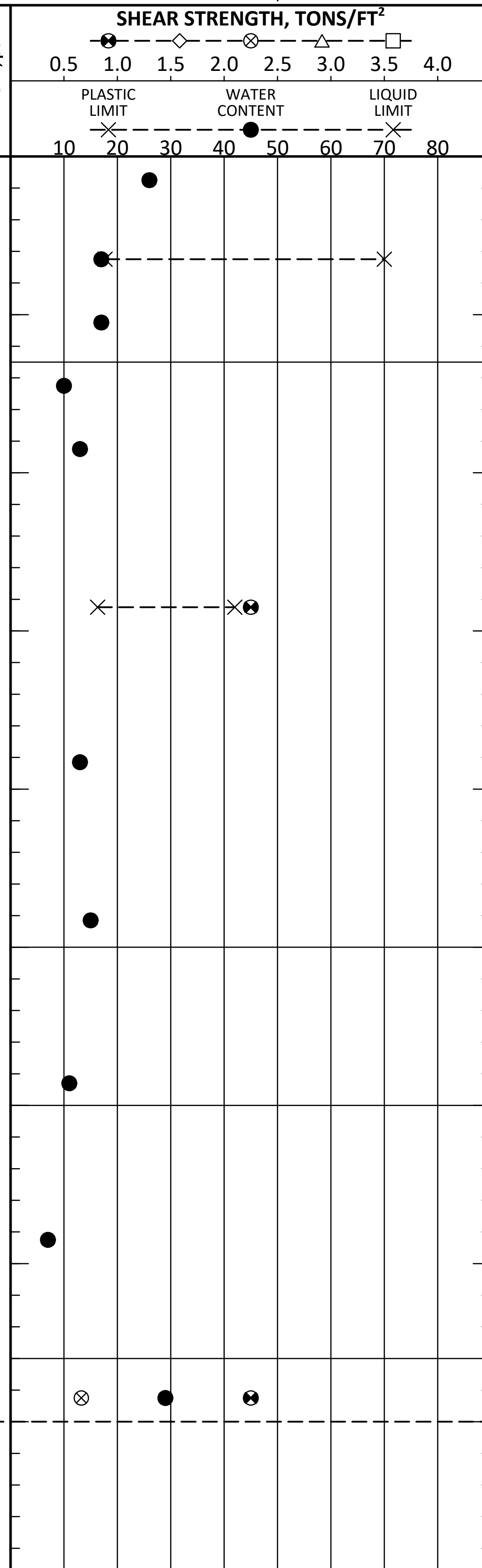
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.59152; W 97.97458

SURFACE ELEVATION: 556.09 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	CLAY, Stiff to Very Stiff, Dark Brown	12			
		14		52	
5		15			
6.5	CLAY, Very Stiff to Hard, Reddish Brown, with calcareous deposits	16			
10		16			
15				26	96
18	- tan below 18 ft	50/9"			
23	- blocky at 23 ft	50/10"			
25	MARL, Hard, Reddish Tan	ref/1"			
30	CLAY, Gravelly, Very Stiff, Tan, with ferrous stains, chert, and sand	29			
38	CLAY, Blocky, Hard, Tan and Gray		94		
40	Boring Terminated				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 40.0 ft
DATE DRILLED: 1/20/2020
DEPTH TO WATER: DRY
DATE MEASURED: 1/20/2020
PROJ. No.: ASA19-128-00
FIGURE: 2

LOG OF BORING NO. B-2

Rio Nogales GSU Spare Foundation
Seguin, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.59150; W 97.97440

SHEAR STRENGTH, TONS/FT²: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

SURFACE ELEVATION: 556.40 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	CLAY, Stiff, Dark Brown	9			
		9			
5		12		52	
	CLAY, Stiff to Hard, Reddish-Brown, with calcareous deposits	15			
			65	18	
10					
15		43			
20		50/10"			
	MARL, Hard, Light Tan	50/0"			
25	GRAVEL, Clayey, Medium Dense, Tan, with ferrous stains and chert				
		29			25
30	Boring Terminated				
35					
40					

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	30.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA19-128-00
DATE DRILLED:	1/20/2020	**DATE MEASURED:**	1/20/2020	**FIGURE:**	3

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

- CALCAREOUS
- CALICHE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY
- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

ROCK TERMS

- CHALK
- CLAYSTONE
- CLAY-SHALE
- CONGLOMERATE
- DOLOMITE
- IGNEOUS
- LIMESTONE
- MARL
- METAMORPHIC
- SANDSTONE
- SHALE
- SILTSTONE

OTHER

- ASPHALT
- BASE
- CONCRETE/CEMENT
- BRICKS / PAVERS
- WASTE
- NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

- BLANK PIPE
- SCREEN
- BENTONITE
- CEMENT GROUT
- BENTONITE & CUTTINGS
- CONCRETE/CEMENT
- CUTTINGS
- GRAVEL
- SAND
- VOLCLAY

SAMPLE TYPES

- AIR ROTARY
- GRAB SAMPLE
- CORE
- GEOPROBE SAMPLER
- ROTOSONIC -DAMAGED
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC -INTACT
- SHELBY TUBE
- SPLIT BARREL
- SPLIT SPOON
- TEXAS CONE PENETROMETER
- DISTURBED

STRENGTH TEST TYPES

- POCKET PENETROMETER
- TORVANE
- UNCONFINED COMPRESSION
- TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. ASA19-128-00

RABAKISTNER

REVISED 04/2012

FIGURE 4a

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
EI = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Rio Nogales GSU Spare Foundation
Seguin, Texas

FILE NAME: ASA19-128-00.GPJ 3/4/2020

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-1	0.0 to 1.5	12	26								
	2.5 to 4.0	14	17	70	18	52	CH				
	4.5 to 6.0	15	17								
	6.5 to 8.0	16	10								
	8.5 to 10.0	16	13								
	13.5 to 15.0			42	16	2	CL		96	2.25	PP
	18.5 to 19.8	50/9"	13								
	23.5 to 24.8	50/10"	15								
	28.5 to 28.6	ref/1"									
	28.6 to 30.0		11								
	33.5 to 35.0	29	7								
	38.5 to 40.0		29					94		0.66	UC
B-2	0.0 to 1.5	9	20								
	2.5 to 4.0	9	19								
	4.5 to 6.0	12	21	71	19	52	CH				
	6.5 to 8.0	15	15								
	8.5 to 10.0		18	34	16	18	CL	65		0.75	UC
	13.5 to 15.0	43	18								
	18.5 to 19.8	50/10"	12								
	23.5 to 24.0	50/0"	11								
	28.5 to 30.0	29	10						25		

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA19-128-00

RABA KISTNER

FIGURE 5

Project Number: ASA19-128-00
Test Date: January 20, 2020

DCP TEST DATA

DCP-1

Rio Nogales GSU Spare Foundation
Seguin, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	1	80	3.1	2	3	0.87
1	1	110	7.5	2	3	0.87
1	2	105	11.6	3	4.5	1.13
1	1	55	13.8	3	4.5	1.13
1	1	35	15.2	5	7.5	1.59
1	1	25	16.1	8	12	2.17
1	1	20	16.9	10	15	2.52
1	2	30	18.1	14	21	3.15
1	2	30	19.3	14	21	3.15
1	3	35	20.7	19	28.5	3.86
1	5	55	22.8	20	30	3.99
1	5	60	25.2	18	27	3.72
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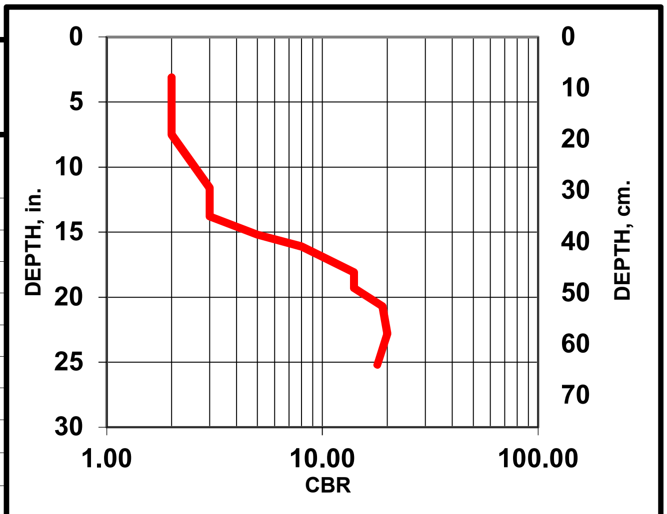
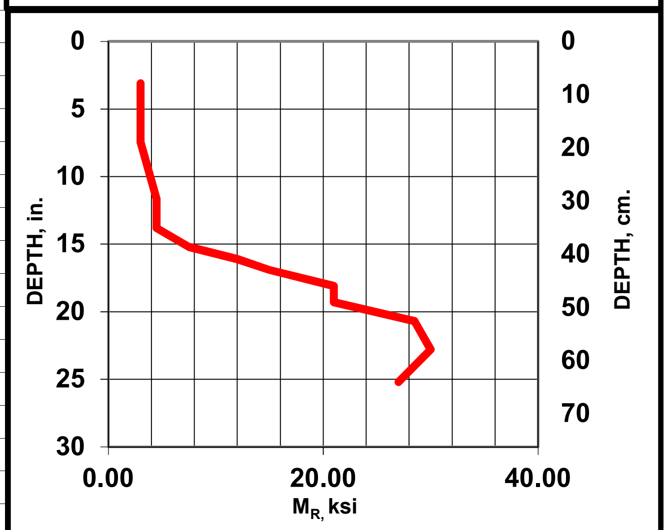
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





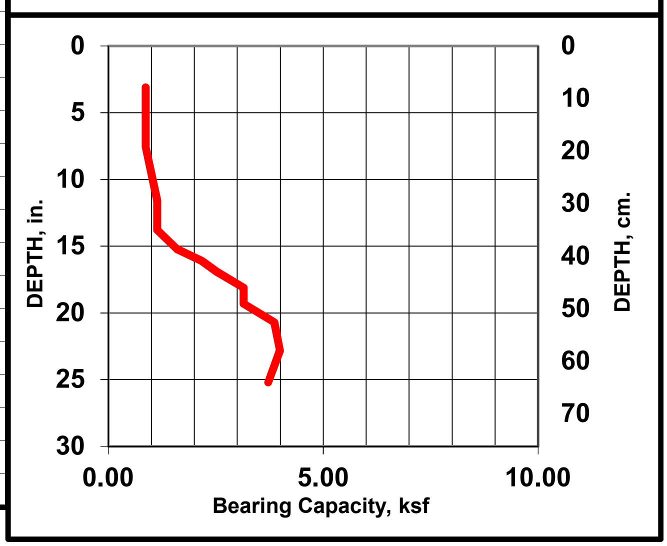


Figure 6a

Project Number: ASA19-128-00
Test Date: January 20, 2020

DCP TEST DATA

DCP-2

Rio Nogales GSU Spare Foundation
Seguin, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	1	80	3.1	2	3	0.87
1	1	60	5.5	3	4.5	1.13
1	2	85	8.9	4	6	1.37
1	1	55	11	3	4.5	1.13
1	1	30	12.2	6	9	1.80
1	2	45	14	9	13.5	2.35
1	2	35	15.4	12	18	2.84
1	2	30	16.5	14	21	3.15
1	2	25	17.5	17	25.5	3.59
1	2	25	18.5	17	25.5	3.59
1	3	30	19.7	22	33	4.25
1	5	60	22	18	27	3.72
1	5	60	24.4	18	27	3.72
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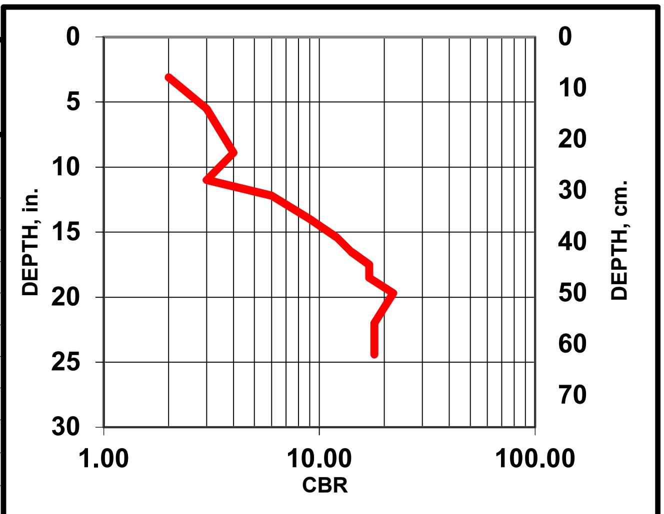
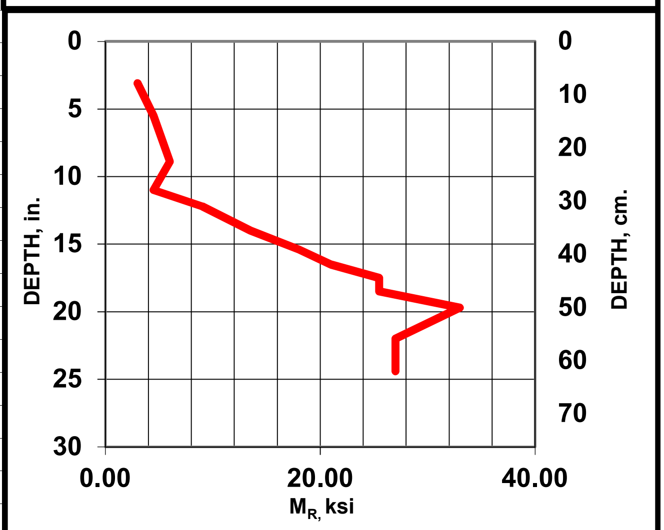
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





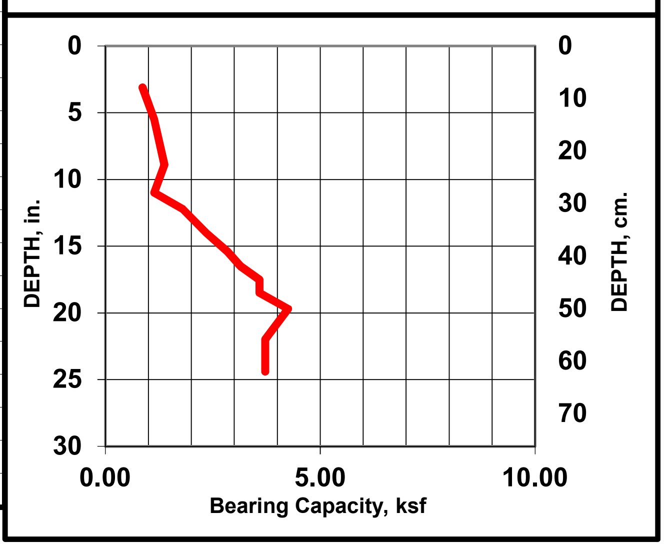


Figure 6b

Project Number: ASA19-128-00
Test Date: January 20, 2020

DCP TEST DATA

DCP-3

Rio Nogales GSU Spare Foundation
Seguin, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	1	70	2.8	3	4.5	1.13
1	1	80	5.9	2	3	0.87
1	1	70	8.7	3	4.5	1.13
1	1	50	10.6	4	6	1.37
1	1	30	11.8	6	9	1.80
1	1	35	13.2	5	7.5	1.59
1	1	25	14.2	8	12	2.17
1	1	30	15.4	6	9	1.80
1	2	35	16.7	12	18	2.84
1	2	35	18.1	12	18	2.84
1	2	25	19.1	17	25.5	3.59
1	2	25	20.1	17	25.5	3.59
1	2	25	21.1	17	25.5	3.59
1	2	20	21.9	22	33	4.25
1	2	20	22.6	22	33	4.25
1	2	20	23.4	22	33	4.25
1	2	20	24.2	22	33	4.25
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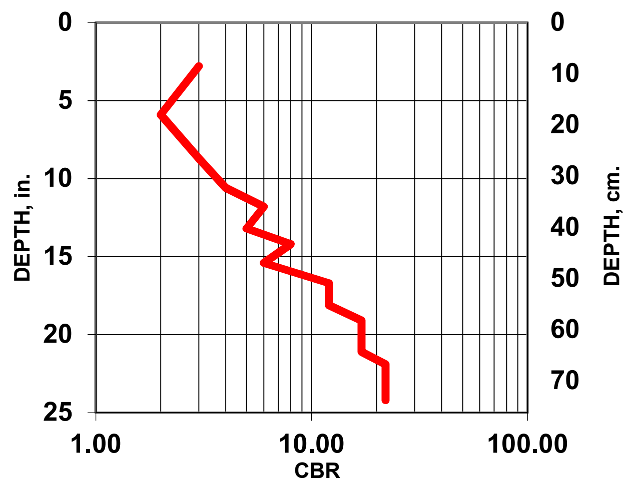
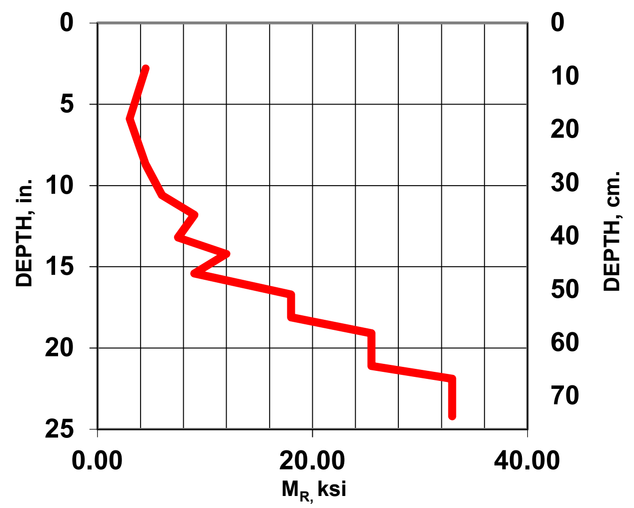
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





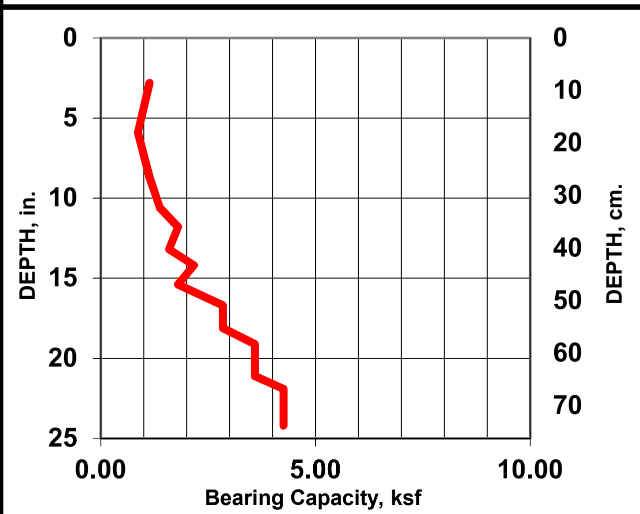
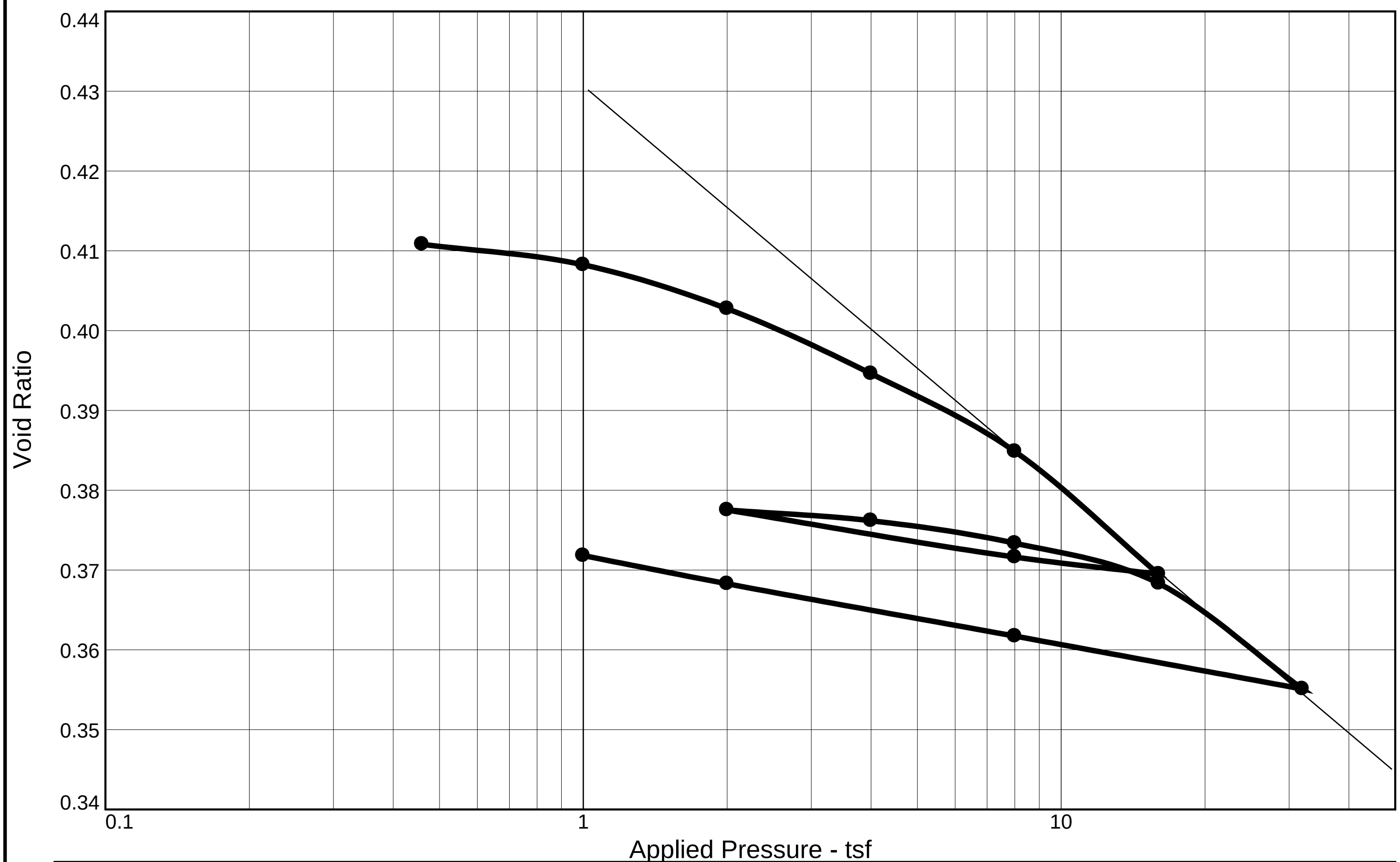


Figure 6c

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	1.00	0.409		9	4.00	1.508					
3	2.00	0.422		10	8.00	0.432					
4	4.00	0.328		11	16.00	0.693					
5	8.00	0.668		12	32.00	0.183					
6	16.00	0.205		13	8.00	0.339					
7	8.00	0.480		14	2.00	0.347					
8	2.00	0.097		15	1.00	0.349					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture								
99.3 %	15.0 %	120.7	42	26	2.72708	1.00	3.5	0.05	0.411

MATERIAL DESCRIPTION	USCS	AASHTO
Light brown clay with calcareous pockets and gravel		

Project No. ASA19-128 **Client:** Structural Engineering Associates, Inc.

Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 **Depth:** 13.5-15ft **Sample Number:** 7

Remarks:

ASTM D2435

Weight added to prevent swell after inundation= 0.46 tsf

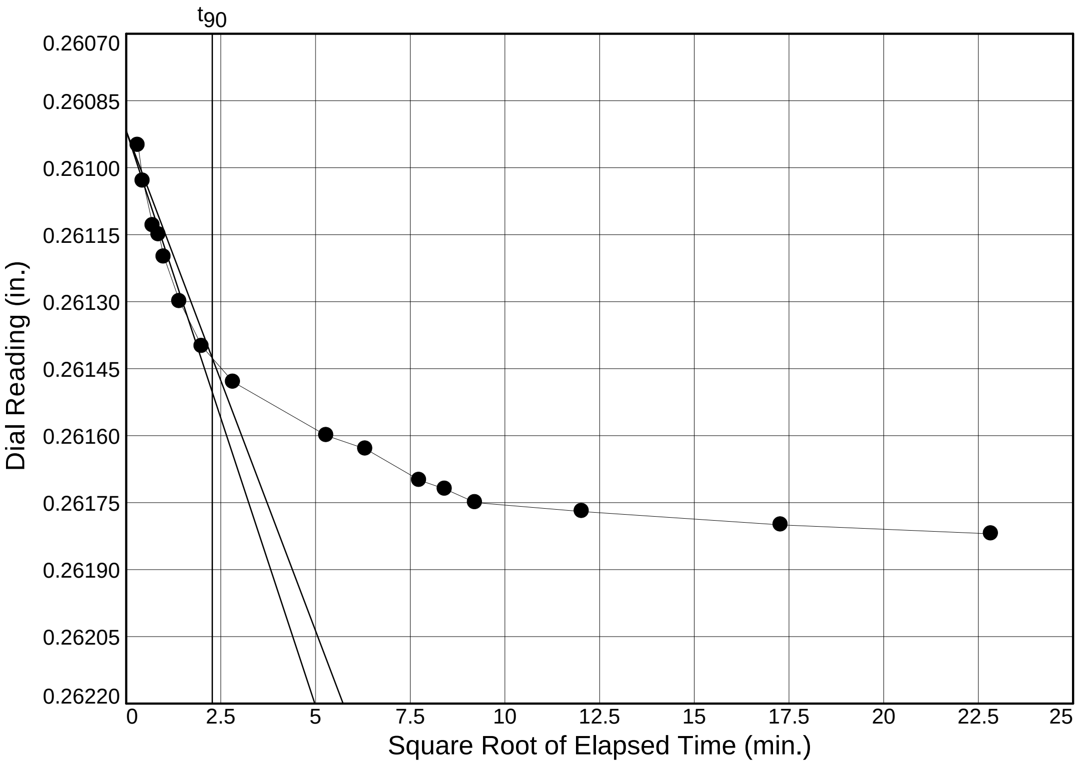
RABA KISTNER CONSULTANTS, INC.

Figure 7a

Dial Reading vs. Time

Project No.: ASA19-128
Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 Depth: 13.5-15ft Sample Number: 7



Load No.= 2

Load= 1.00 tsf

D_0 = 0.2609

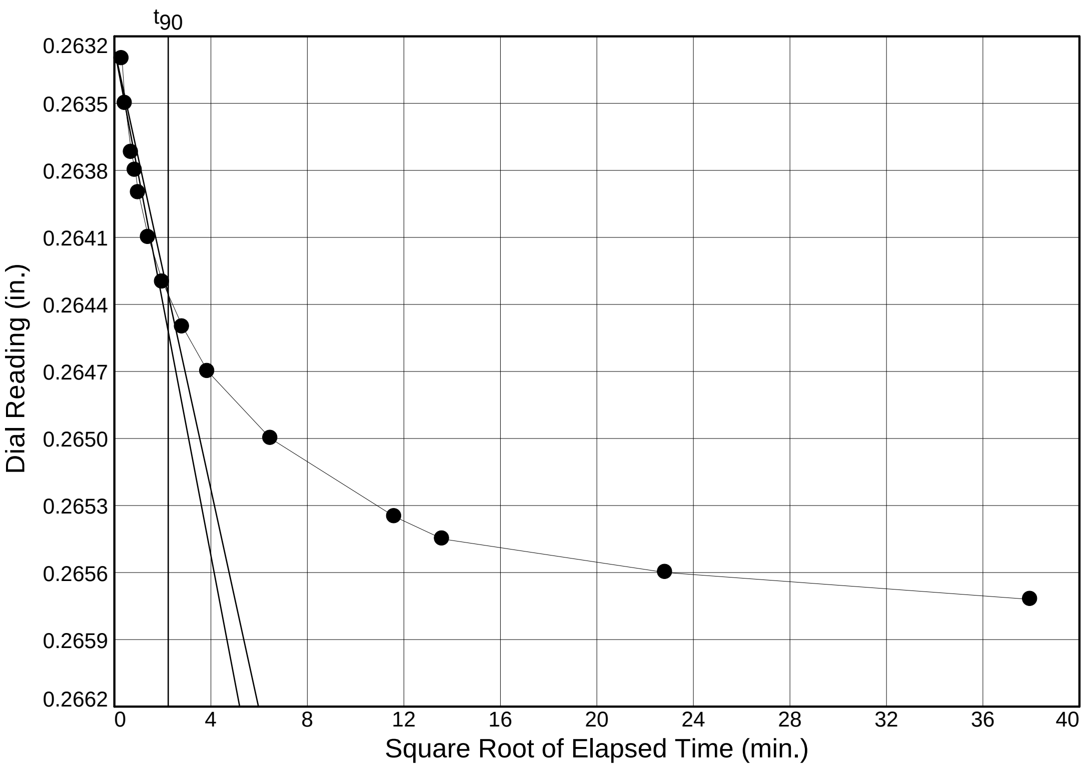
D_{90} = 0.2614

D_{100} = 0.2615

T_{90} = 5.17 min.

C_v @ T_{90}

0.409 ft.^{2}/day



Load No.= 3

Load= 2.00 tsf

D_0 = 0.2633

D_{90} = 0.2644

D_{100} = 0.2645

T_{90} = 4.99 min.

C_v @ T_{90}

0.422 ft.^{2}/day

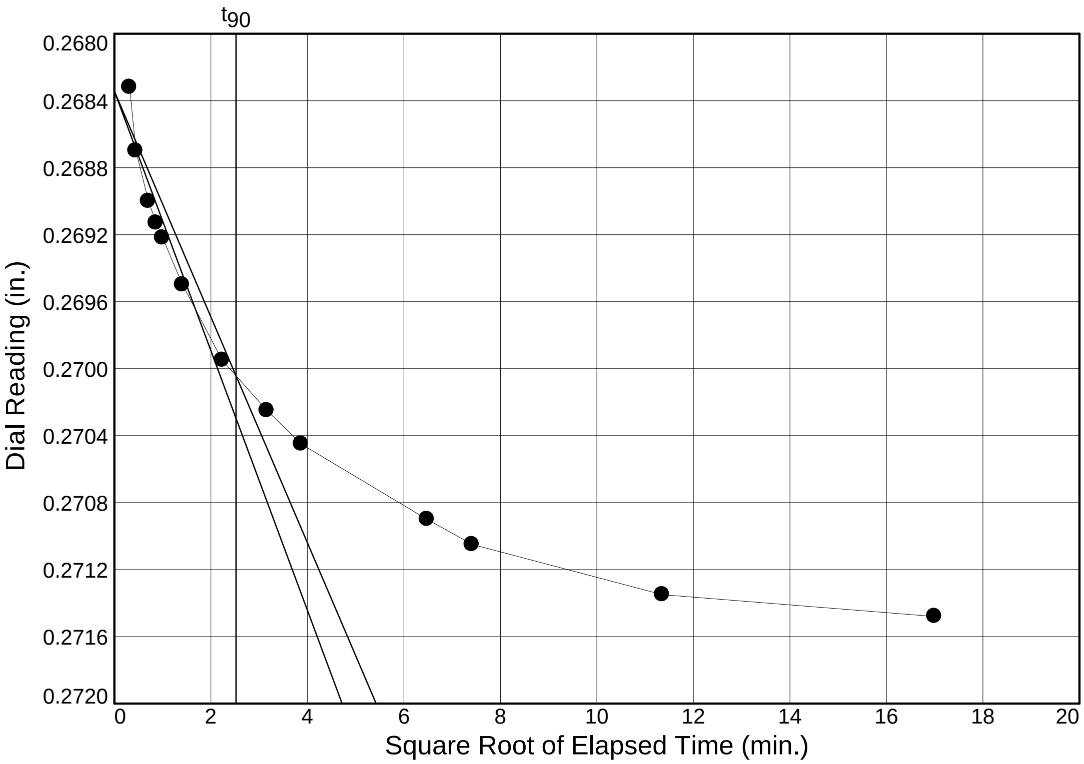
Figure 7b

RABA KISTNER CONSULTANTS, INC.

Dial Reading vs. Time

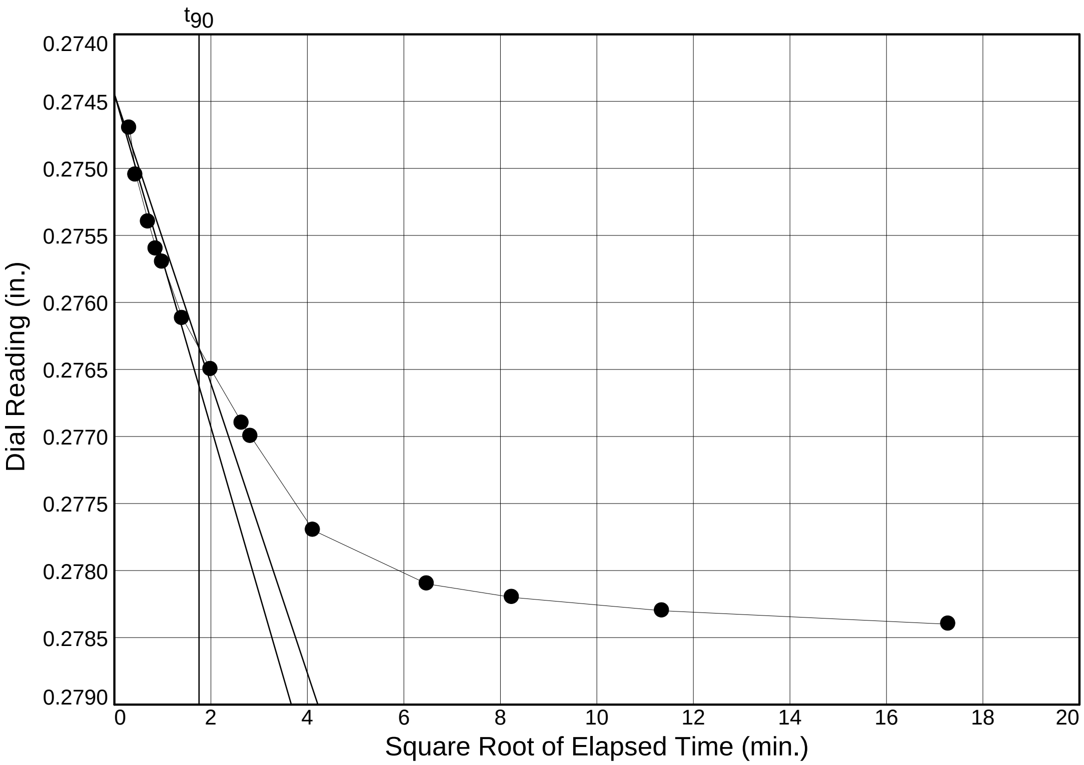
Project No.: ASA19-128
Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 Depth: 13.5-15ft Sample Number: 7



Load No.= 4
Load= 4.00 tsf
D_0 = 0.2683
D_{90} = 0.2700
D_{100} = 0.2702
T_{90} = 6.35 min.

C_v @ T_{90}
0.328 ft.^{2}/day



Load No.= 5
Load= 8.00 tsf
D_0 = 0.2744
D_{90} = 0.2763
D_{100} = 0.2766
T_{90} = 3.08 min.

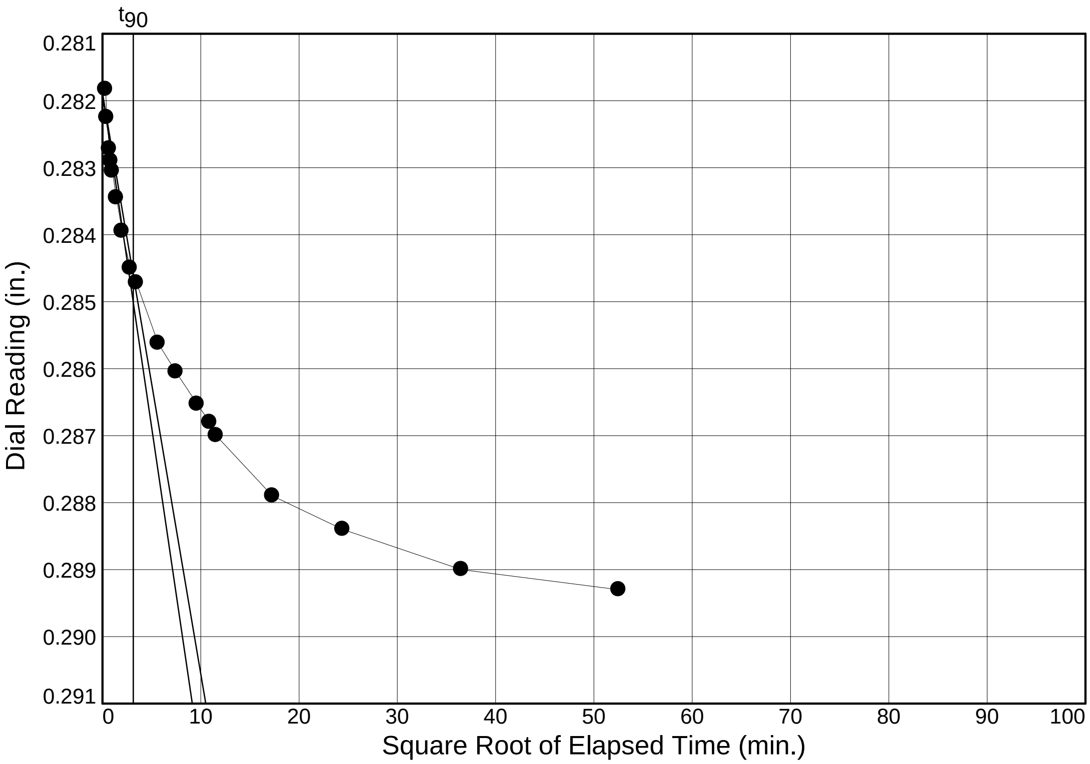
C_v @ T_{90}
0.668 ft.^{2}/day

Figure 7c

Dial Reading vs. Time

Project No.: ASA19-128
Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 Depth: 13.5-15ft Sample Number: 7



Load No.= 6

Load= 16.00 tsf

D_0 = 0.2819

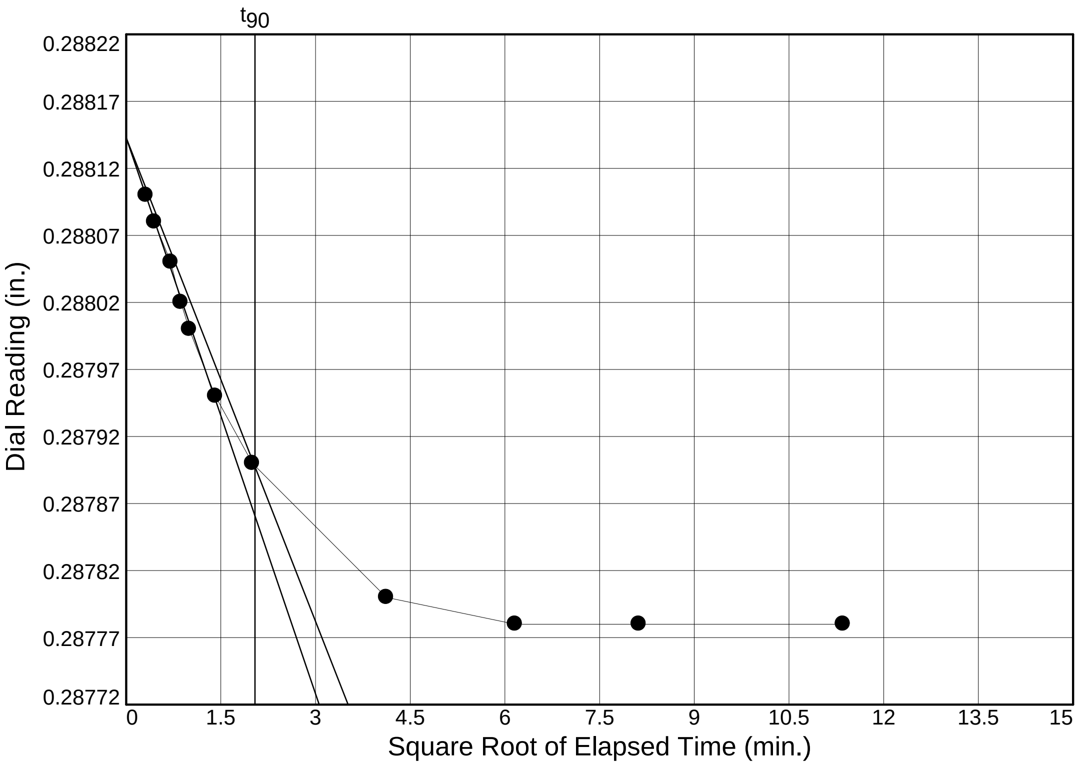
D_{90} = 0.2846

D_{100} = 0.2849

T_{90} = 9.86 min.

C_v @ T_{90}

0.205 ft.^{2}/day



Load No.= 7

Load= 8.00 tsf

D_0 = 0.2881

D_{90} = 0.2879

D_{100} = 0.2879

T_{90} = 4.17 min.

C_v @ T_{90}

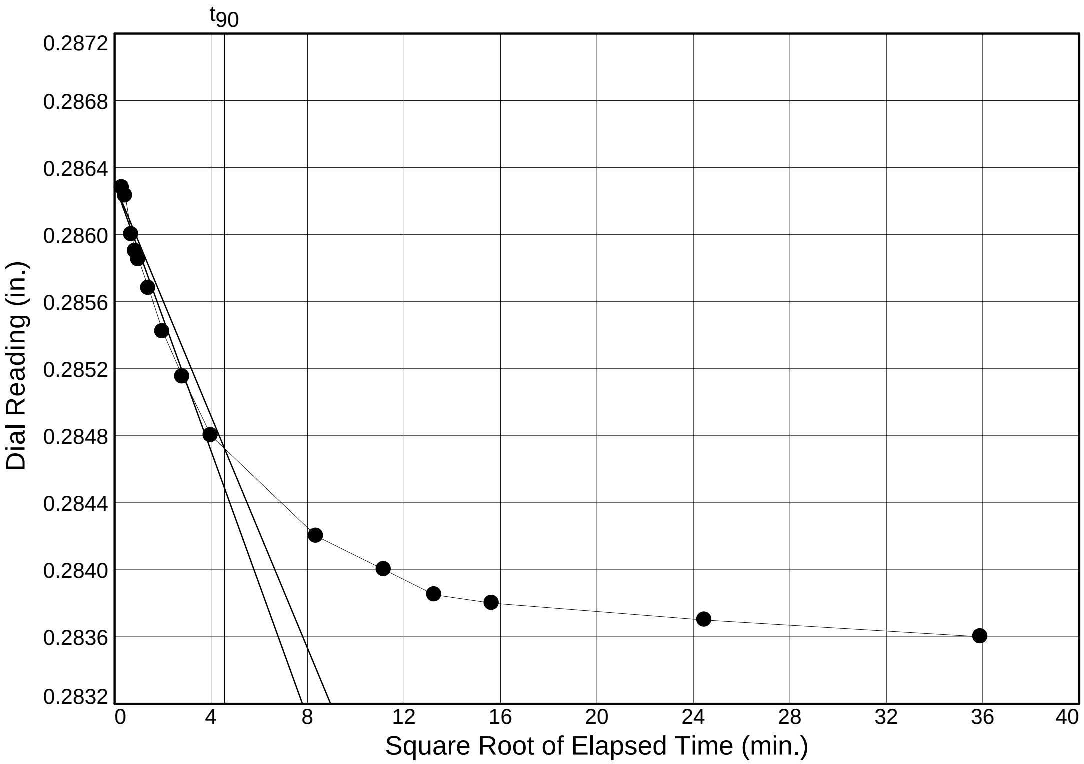
0.480 ft.^{2}/day

Figure 7d

Dial Reading vs. Time

Project No.: ASA19-128
Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 Depth: 13.5-15ft Sample Number: 7



Load No.= 8

Load= 2.00 tsf

D_0 = 0.2863

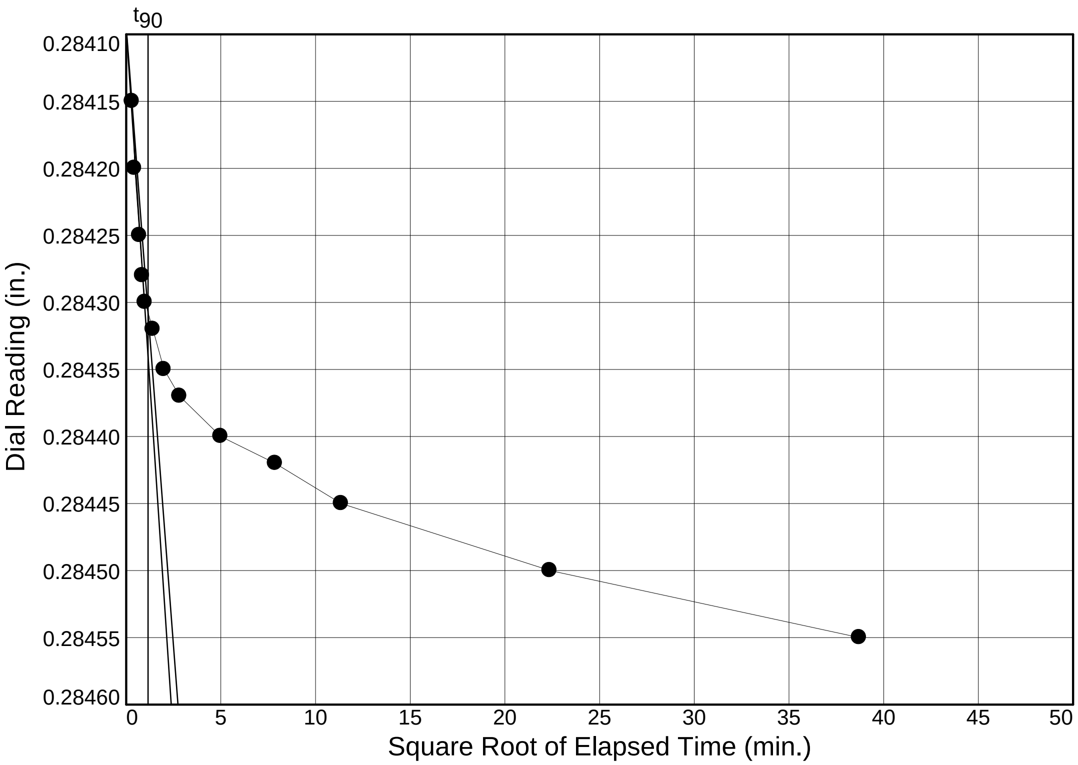
D_{90} = 0.2847

D_{100} = 0.2845

T_{90} = 20.78 min.

C_v @ T_{90}

0.097 ft.^{2}/day



Load No.= 9

Load= 4.00 tsf

D_0 = 0.2841

D_{90} = 0.2843

D_{100} = 0.2843

T_{90} = 1.34 min.

C_v @ T_{90}

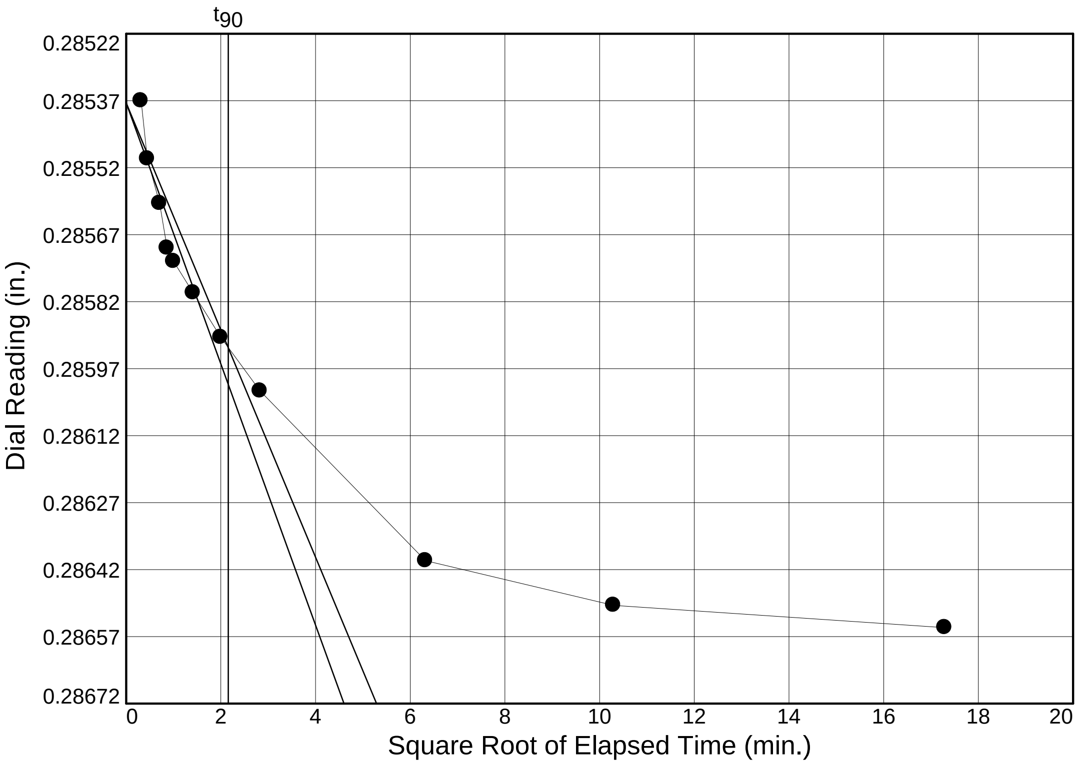
1.508 ft.^{2}/day

Figure 7e

Dial Reading vs. Time

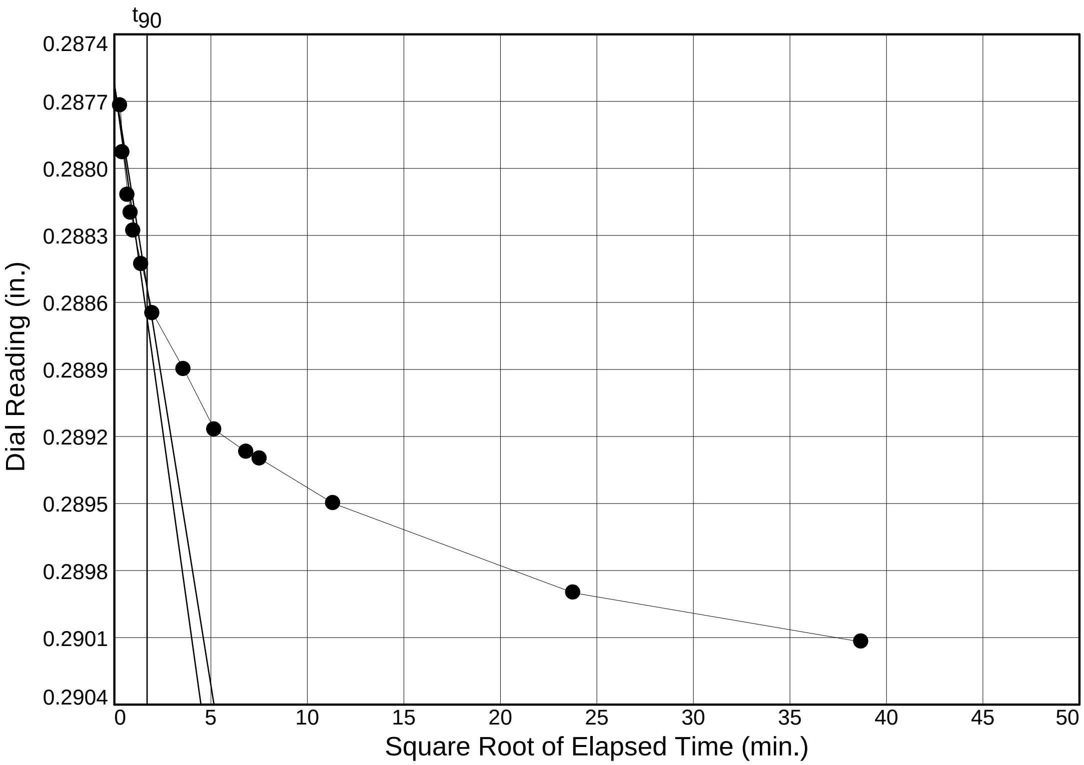
Project No.: ASA19-128
Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 Depth: 13.5-15ft Sample Number: 7



Load No.= 10
Load= 8.00 tsf
D_0 = 0.2854
D_{90} = 0.2859
D_{100} = 0.2860
T_{90} = 4.66 min.

C_v @ T_{90}
0.432 ft.^{2}/day



Load No.= 11
Load= 16.00 tsf
D_0 = 0.2876
D_{90} = 0.2885
D_{100} = 0.2886
T_{90} = 2.89 min.

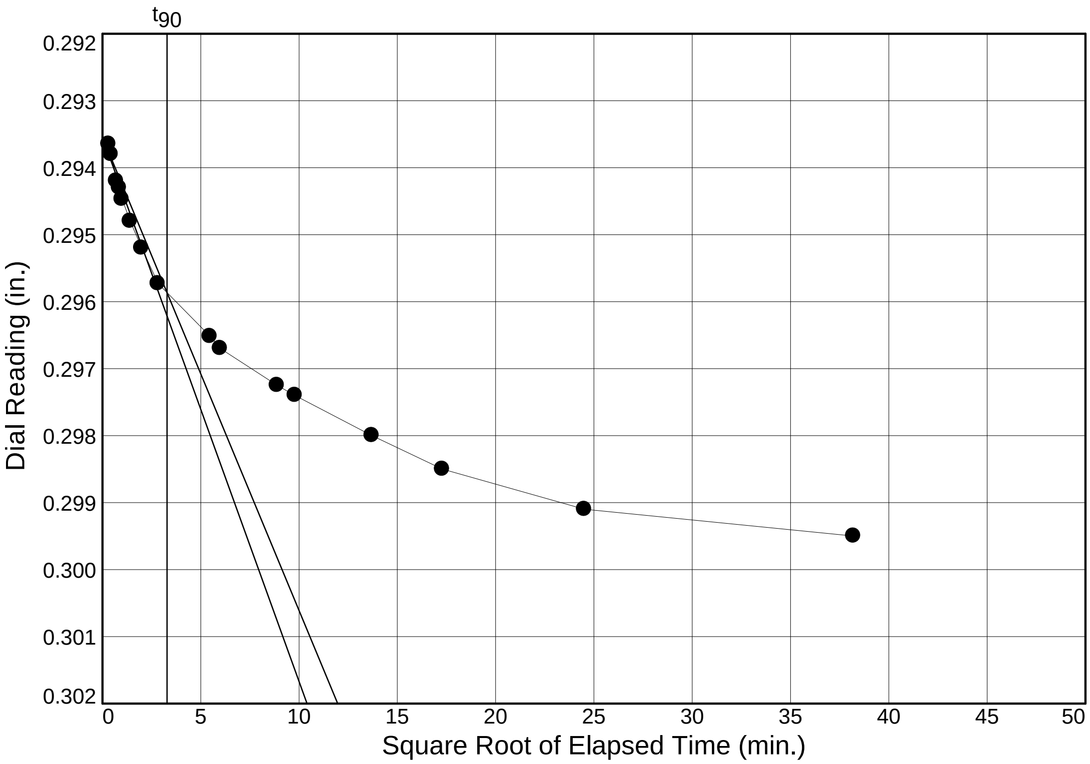
C_v @ T_{90}
0.693 ft.^{2}/day

Figure 7f

Dial Reading vs. Time

Project No.: ASA19-128
Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 Depth: 13.5-15ft Sample Number: 7



Load No.= 12

Load= 32.00 tsf

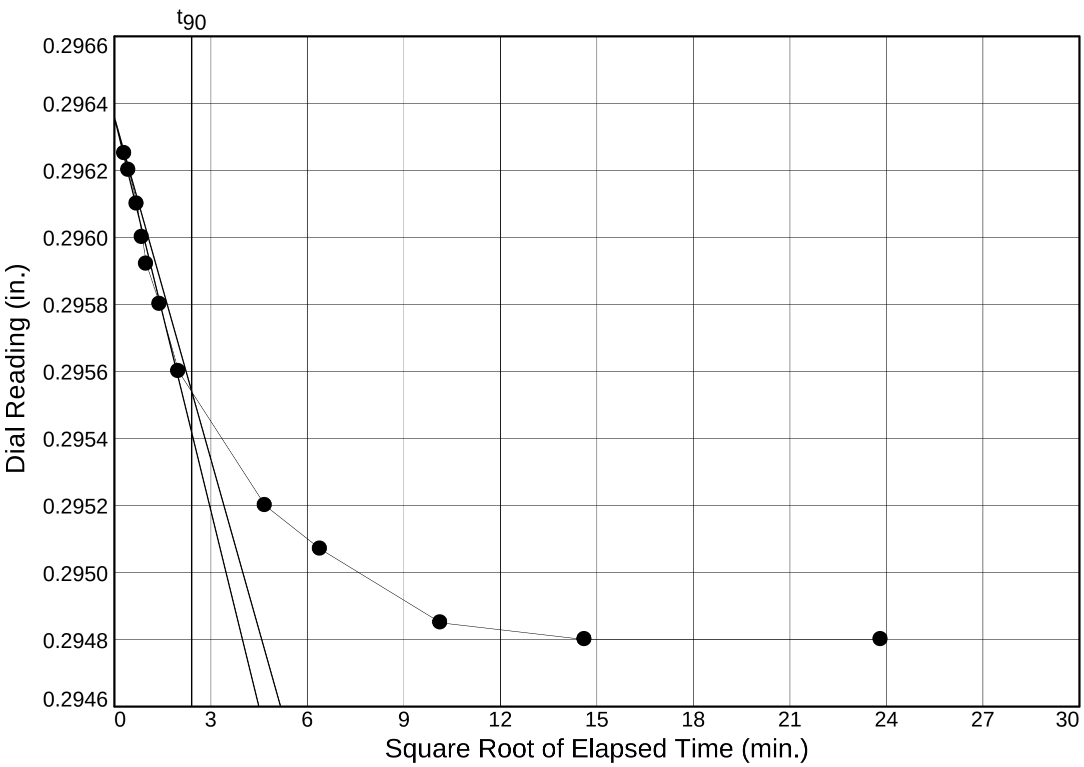
D_0 = 0.2935

D_{90} = 0.2959

D_{100} = 0.2961

T_{90} = 10.79 min.

C_v @ T_{90}
0.183 ft.^{2}/day



Load No.= 13

Load= 8.00 tsf

D_0 = 0.2964

D_{90} = 0.2955

D_{100} = 0.2954

T_{90} = 5.79 min.

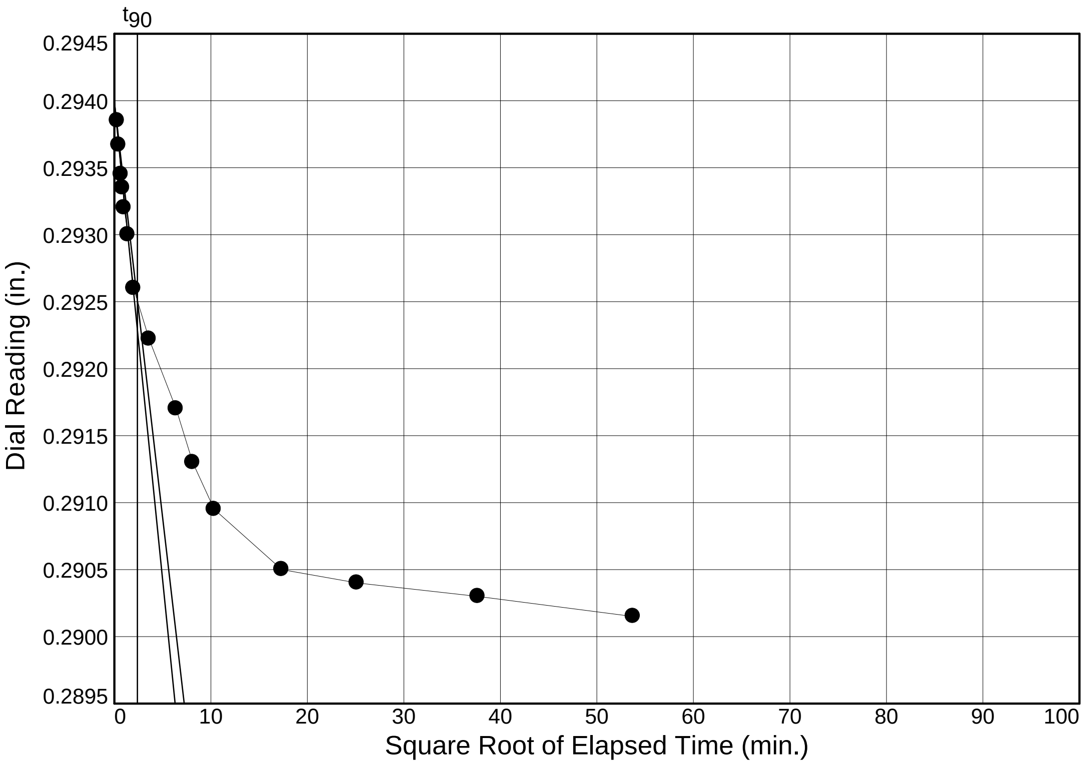
C_v @ T_{90}
0.339 ft.^{2}/day

Figure 7g

Dial Reading vs. Time

Project No.: ASA19-128
Project: Rio Nogales GSU Spare Foundation

Location: Boring 1 Depth: 13.5-15ft Sample Number: 7



Load No.= 14

Load= 2.00 tsf

D_0 = 0.2940

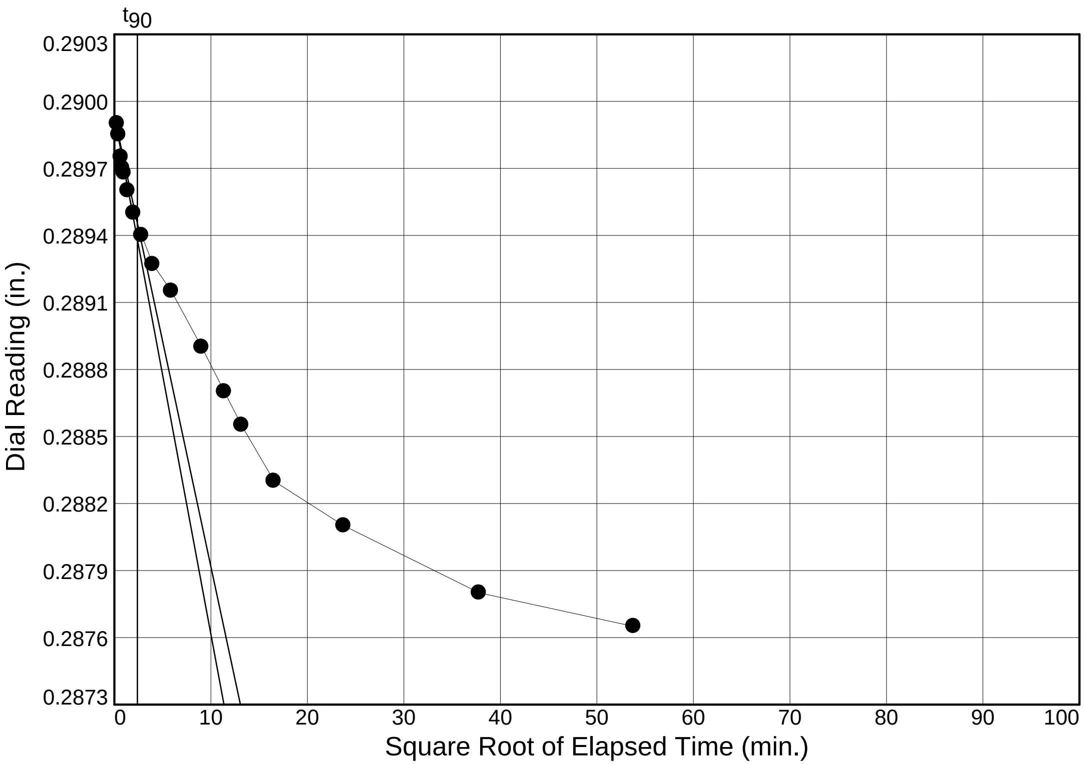
D_{90} = 0.2925

D_{100} = 0.2923

T_{90} = 5.73 min.

C_v @ T_{90}

0.347 ft.^{2}/day



Load No.= 15

Load= 1.00 tsf

D_0 = 0.2899

D_{90} = 0.2895

D_{100} = 0.2894

T_{90} = 5.72 min.

C_v @ T_{90}

0.349 ft.^{2}/day

Figure 7h

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL of the Geoprofessional Business Association

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

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