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GEOTECHNICAL ENGINEERING STUDY

FOR

HIDDEN SPRINGS WATER IMPROVEMENTS
SAN ANTONIO, TEXAS

Austin, TX
Brownsville, TX
Dallas, TX
El Paso, TX
Houston, TX
McAllen, TX
México
San Antonio, TX

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Project No. ASA11-032-00
May 9, 2011

Mr. Jeffrey E. Reck, P.E.
LNV Engineering, Inc.
8918 Tesoro Drive, Suite 401
San Antonio, Texas 78217

RE: Geotechnical Engineering Study
Hidden Springs Water Improvements
San Antonio, Texas

Dear Mr. Reck:

Raba-Kistner Consultants Inc. (R-K) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with **R-K** Proposal No. PSA11-044-00, dated March 15, 2011. The purpose of this study was to drill borings within the proposed water main improvements, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the proposed improvements.

The following report contains our design recommendations and considerations based on our current understanding of the project information provided to us. There may be alternatives for value engineering of the foundation systems, and **R-K** 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.

Ian Perez
T. Ian Perez, ~~P.E.~~
Project Engineer

GARLAND L. BURCH
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Garland L. Burch, P.E.
Senior Geotechnical Consultant
5-9-11

TIP/GLB/mem

Attachments

Copies Submitted: Above (3)

GEOTECHNICAL ENGINEERING STUDY

For

HIDDEN SPRINGS WATER IMPROVEMENTS
SAN ANTONIO, TEXAS

Prepared for

LNV ENGINEERING, INC.
San Antonio, Texas

Prepared by

RABA-KISTNER CONSULTANTS, INC.
San Antonio, Texas

PROJECT NO. ASA11-032-00

May 9, 2011

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INTRODUCTION

Raba-Kistner Consultants Inc. (R-K) has completed the authorized subsurface exploration and foundation analysis for the proposed water system improvements located within the Hidden Springs subdivision in San Antonio, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations.

PROJECT DESCRIPTION

The facilities being considered in this study include two new booster pump stations/tanks and new 8 to 16 in. diameter water mains within the Hidden Springs subdivision in San Antonio, Texas. Relatively light loads are anticipated to be carried by the booster pump station and tank foundation systems which will be located on the east and west sides of the intersection of Brewer Drive and Aue Road. It is our understanding that at the time of this study, site grading plans and proposed structural loads for the booster pump stations and tanks were not yet available. The following is our understanding of proposed water mains planned in the Hidden Springs Subdivision:

- 930 ft of 12 in. main on Rocky Hill Boulevard west of Manor Hill Road;
- 2,730 ft of 12 in. main on Rocky Hill Boulevard east of Manor Hill Road;
- 1,700 ft of 12 in. main on Aue Road north of Whistling Wind;
- 870 ft of 16 in. main on Aue Road south of Whistling Wind;
- 2,300 ft of 12 in. main on Black Creek;
- 1,200 ft of 8 in. main on Whistling Wind east of Black Creek;
- 860 ft of 12 in. main on Cedar Brush; and
- 200 ft of 12 in. main between Cedar Brush and Crescent Ledge in the subdivision south and adjacent to Hidden Springs Subdivision.

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 LNV Engineering, 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 recommendations submitted in this report are based on the data obtained from 8 widely spaced borings drilled at this site, our understanding of the project information provided to us, and the assumption that site grading will result in only minor changes in the existing topography. 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 from existing grades (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, FIELD AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by 8 widely spaced borings drilled at the locations shown on the Boring Location Map, Figure 1. These locations are approximate and distances were measured using tape, angles, pacing, etc. The borings were drilled using a truck-mounted drilling rig to maximum approximate depths of 25 and 10 ft below the existing ground surface for Borings B-1 through B-3 and B-4 through B-8, respectively. During drilling operations, the following samples were collected:

Type of Sample	Number Collected
Auger (grab samples)	7
Split-Spoon (with Standard Penetration Test)	49

Each sample was visually classified in the laboratory by a member of our Geotechnical Engineering staff. The geotechnical engineering properties of the strata were evaluated by the following tests:

Type of Test	Number Conducted
Natural Moisture Content	49
Atterberg Limits	13
Percent Passing a No. 200 Sieve	4

The results of all laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 3 through 10. A key to classification terms and symbols used on the logs is presented on Figure 11. The results of the laboratory and field testing are also tabulated on Figure 12 for ease of reference.

Standard penetration test results are noted as “blows per ft” on the boring logs and Figure 12, where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft of penetration into the soil/weak rock. Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal “ref” for 6 in. or less will be noted on the boring logs and on Figure 12.

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.

FIELD RESISTIVITY TESTING

Field resistivity testing of shallow subsurface strata (15 to 50 feet below the ground surface) was conducted at two locations (resistivity sounding stations) at the proposed booster pump stations. These tests were conducted in accordance with procedures described in *Standard Test Method for Field Measurement of Soil Resistivity using the Wenner Four Electrode Array (ASTM G-57)*. This testing was performed by an **R-K** environmental professional. Each resistivity test was conducted in the vicinity of two geotechnical borings (Borings B-1 and B-2) along the project alignment. The locations of these tests are indicated as Resistivity Sounding Stations on the Resistivity Sounding Station Location Map on Figure 2.

In all instances, resistivity measurements were obtained along sounding station arrays oriented along the proposed Hidden Valley water improvements project alignment, although additional measurements were obtained periodically along orthogonal arrays in order to evaluate potential anisotropy associated with ground resistivity measurements and provide for periodic checks on the quality of data obtained.

Resistivity Sounding Data Sheets containing resistance values (ohms) measured directly in the field and the calculated apparent resistivity values (ohm-feet and ohm-centimeters) for various electrode-spacing configurations (A-spacings) are presented in the Attachments of this report on Figure 17.

GENERAL SITE CONDITIONS

SITE DESCRIPTION

The project site(s) include various locations and alignments within the Hidden Springs Subdivision in San Antonio, Texas. The site is an existing subdivision with paved roads and residential lots.

GEOLOGY

A review of the *Geologic Atlas of Texas, San Antonio Sheet*, indicates that this site is naturally underlain with the soils/rocks of the Glen Rose formation. The Glen Rose formation is generally characterized as limestone, dolomite and marl as alternating resistant and recessive beds that form stairstep topography. The limestone is generally fine grained and marly while the dolomite is fine grained, porous and fossiliferous. The Glen Rose formation is divided into and upper and a lower part. The upper part is relatively thinner bedded, more dolomitic and less fossiliferous while the lower part is more massive in nature. Key geotechnical engineering concerns for development supported on this formation are the porous zones and the hardness of the limestone as it impacts excavation operations

SEISMIC COEFFICIENTS

Based upon a review of Section 1613 *Earthquake Loads – Site Ground Motion* of the 2006 International Building Code, the following information has been summarized for seismic considerations associated with this site.

- Site Class Definition (Table 1613.1.1): **Class B**. Based on the soil borings conducted for this investigation, the upper 100 feet of soil may be characterized as rock.
- Mapped Maximum Considered Earthquake Ground Motion for a 0.2 sec Spectral Response Acceleration (Figure 1613(1)): $\mathbf{S_s = S_{ms} = 0.093g}$. Note that the value taken from Figure 1613(1) is based on Site Class B and no adjustment is required.
- Mapped Maximum Considered Earthquake Ground Motion for a 1 sec Spectral Response Acceleration (Figure 1613(1)): $\mathbf{S_1 = S_{m1} = 0.030g}$. Note that the value taken from Figure 1613(2) is based on Site Class B and no adjustment is required
- Values of Site Coefficient (Table 1613.1.2(1)): $\mathbf{F_a = 1.2}$
- Values of Site Coefficient (Table 1613.1.2(2)): $\mathbf{F_v = 1.7}$

The Design Spectral Response Acceleration Parameters are as follows:

- 0.2 sec, based on equation 16-40: $\mathbf{S_{DS} = 0.062}$
- 1 sec, based on equation 16-41: $\mathbf{S_{D1} = 0.020}$

Based on the parameters listed above, Tables 1616.3(1) and 1616.3(2), the Seismic Design Category for both short period and 1 second response accelerations is **A**. However, without more information, we are not able to discern the Seismic Use Group, which will be one of the following four choices; I, II, III, or IV.

STRATIGRAPHY

The subsurface stratigraphy at this site can generally be described as a thin veneer of dark brown clay overlying hard, tan marl or very stiff, tan clay. Limestone was encountered in Boring B-3 at a depth of 15 ft below the existing ground surface. Limestone may be encountered as rock outcrops at the surface in other areas of the pipeline alignment. The boring logs should be consulted for more specific stratigraphic information. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The lines designating the interfaces between strata on the boring logs represent approximate boundaries. Transitions between strata may be gradual.

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 at the clay/marl interface and 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 ANALYSIS

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 ranging from 1 to 2 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 above PVR values.

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....) 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- BOOSTER PUMP STATIONS AND TANKS

To reduce expansive soil-related movements in at-grade construction of the booster pump stations and tanks, a portion of the upper highly expansive subgrade clays can be removed by overexcavating and backfilling with a suitable select fill material. We recommend that all of the dark brown clays or the upper 2 ft of soil from the existing ground surface, whichever results in the lower elevation, be completely removed from the proposed booster pump station and tank foundation areas. To maintain negligible PVR values, subsequent fill placed in the foundation area should consist of select fill material in accordance with the *Select Fill* Section of this report.

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 structures. Water entering the fill surface during construction or entering the fill exposed beyond the building lines after construction may create problems with fill moisture control during compaction and increased access for moisture 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 building 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 building) over the select fill material and any perimeter drain extending beyond the building lines, 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 building 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.

FOUNDATION RECOMMENDATIONS

SITE GRADING

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

SHALLOW FOUNDATION SYSTEMS

The proposed booster pump stations may be founded on rigid-engineered beam and slab foundations and the storage tanks may be founded on conventional, continuous footing foundations (ringwall foundations), 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 the structures. If shallow foundations are to be considered, we recommend that overexcavation and select fill replacement be utilized to reduce expansive soil-related movements.

Allowable Bearing Capacity

Shallow foundations founded on compacted, select fill or native undisturbed soil should be proportioned using the design parameters tabulated below. If movement tolerances are such that overexcavation and select fill replacement is not required, the recommendations provided in the table below may be utilized for the in-situ, surficial soils.

Minimum depth below final grade	18 in.
Minimum beam width	12 in.
Minimum widened beam width	18 in.
Maximum allowable bearing pressure for grade beams/conventional continuous footings (ringwall)	2,500 psf
Maximum allowable bearing pressure for widened beams/conventional continuous footings (ringwall)	3,000 psf

The above presented maximum allowable bearing pressures will provide a factor of safety of about 3 with respect to the measured shear strength, provided that fill is selected and placed as recommended in the *Select Fill* section of this report or provided that the subgrade is prepared in accordance with the recommendations outlined in the *Site Preparation* section of this report.

We recommend that a vapor barrier comprised of polyethylene or polyvinylchloride (PVC) sheeting be placed between the supporting soils and the concrete floor slab.

B.R.A.B./W.R.I. Criteria

Beam and slab foundations are sometimes designed using criteria developed by the Building Research Advisory Board (B.R.A.B.) or the Wire Reinforcement Institute (W.R.I.). Based on an email from the structural engineer via the CLIENT, it is our understanding that B.R.A.B. and W.R.I. design criteria will be utilized to design the foundations at this site. Recommended values for the Climatic Rating (C_w) and minimum unconfined compressive strength (q_u) are as listed in the table below:

Climatic Rating, C_W	16
Unconfined Compressive Strength, q_u	3,000 psf

The recommended design plasticity index (PI) for the booster pump station sites (in the vicinity of Borings B-1 and B-2) is 27. The corresponding soil support index, C, is 0.86 for a design PI of 27.

If overexcavation and select fill replacement is implemented to reduce expansive soil-related movements to approximately 1 in. or less, a reduced B.R.A.B. design PI of 20 may be utilized with a corresponding C value of 0.94.

BELOW GRADE WALLS – BOOSTER PUMP STATIONS

Depending on the final elevation of the booster pump station foundation, below grade walls may be necessary. The following sections provide information for evaluating lateral earth pressures, backfill compaction, and drainage issues.

LATERAL EARTH PRESSURES

Equivalent fluid density values for computation of lateral soil pressures acting on retaining walls were evaluated for various types of backfill materials that may be placed behind the retaining walls. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented below in preferential order for use as backfill materials.

Back Fill Type	Estimated Total Unit Weight (pcf)	Active Condition		At Rest Condition	
		Earth Pressure Coefficient, k_a	Equivalent Fluid Density (pcf)	Earth Pressure Coefficient, k_o	Equivalent Fluid Density (pcf)
Washed Gravel	135	0.29	40	0.45	60
Crushed Limestone	145	0.24	35	0.38	55
Clean Sand	120	0.33	40	0.5	60
Pit Run Clayey Gravels or Sands	135	0.32	45	0.48	65
Clays	120	0.59	70	0.74	90

The values tabulated above under "Active Conditions" pertain to flexible retaining walls free to tilt outward as a result of lateral earth pressures. For rigid, non-yielding walls the values under "At-Rest Conditions" should be used.

The values presented above assume the surface of the backfill materials to be level. Sloping the surface of the backfill materials will increase the surcharge load acting on the structures. The above values also do not include the effect of surcharge loads such as construction equipment, vehicular loads, or future storage near the structures. Nor do the values account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the backfill materials. However, these surcharge loads and groundwater pressures should be considered in designing any structures subjected to lateral earth pressures.

The on-site clays exhibit significant shrink/swell characteristics. The use of these soils as backfill against the proposed retaining structures is not recommended. These soils generally provide higher design active earthen pressures, as indicated above, but may also exert additional active pressures associated with swelling. Controlling the moisture and density of these materials during placement will help reduce the likelihood and magnitude of future active pressures due to swelling, but this is no guarantee.

BACKFILL COMPACTION

Placement and compaction of backfill behind the retaining walls will be critical, particularly at locations where backfill will support adjacent near-grade foundations and/or flatwork. If the backfill is not properly compacted in these areas, the adjacent foundations/flatwork can be subject to settlement.

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed and compacted as recommended in the *Select Fill* section of this report. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the walls, we recommend that compaction be accomplished using hand-guided compaction equipment capable of achieving the maximum density in a series of 3 to 5 passes.

DRAINAGE

The use of drainage systems is a positive design step toward reducing the possibility of hydrostatic pressure acting against the retaining structures. Drainage may be provided by the use of a drain trench and pipe. The drain pipe should consist of a slotted, heavy duty, corrugated polyethylene pipe and should be installed and bedded according to the manufacturer's recommendations. The drain trench should be filled with gravel (meeting the requirements of ASTM D 448 coarse concrete aggregate Size No. 57 or 67) and extend from the base of the structure to within 2 ft of the top of the structure. The bottom of the drain trench will provide an envelope of gravel around the pipe with minimum dimensions consistent with the pipe manufacturer's recommendations. The gravel should be wrapped with a suitable geotextile fabric (such as Mirafi 140N or equivalent) to help minimize the intrusion of fine-grained soil particles into the drain system. The pipe should be sloped and equipped with clean-out access fittings consistent with state-of-the-practice plumbing procedures.

As an alternative to a full-height gravel drain trench behind the proposed retaining structures, consideration may be given to utilizing a manufactured geosynthetic material for wall drainage. A number of products are available to control hydrostatic pressures acting on earth retaining structures, including Amerdrain (manufactured by American Wick Drain Corp.), Miradrain (manufactured by Mirafi, Inc.), Enkadrain (manufactured by American Enka Company), and Geotech Insulated Drainage Panel (manufactured by Geotech Systems Corp.). The geosynthetics are placed directly against the retaining structures and are hydraulically connected to the gravel envelope located at the base of the structures.

With the exception of basement or subfloor walls, weepholes may be provided along the length of the proposed retaining structures, if desired, in addition to one of the two alternative drainage measures presented above. Based on our experience, weepholes, as the only drainage measure, often become clogged with time and do not provide the required level of drainage from behind retaining structures. We recommend that **R-K** review the final retaining structure drainage design before construction.

BURIED PIPE RECOMMENDATIONS

The following sections provide our recommendations with respect to buried pipe design including the loads imposed on buried pipe, guidelines for thrust restraint, and our recommendations for bedding and backfill. In addition, installation considerations and guidelines are also provided with respect to trench safety, excavation dewatering and equipment.

GENERAL

Loads on buried pipes result from a combination of material properties of the pipe and surrounding soils, the methods and techniques used during the installation process (i.e. material used for the haunch, the amount of compactive effort in the backfill materials, etc.), live loads such as roadway traffic, and internal forces due to the transmission of fluids within the pipe. As such, care should be taken to assure design assumptions are validated by review of project specifications prior to construction and appropriate quality control/ quality assurance monitoring during construction.

EARTH LOADS

The weight of the soil over the top of a buried pipe is dependant upon the installation method, the backfill materials, and the degree of compaction achieved during construction. The soil prism method is a common way to describe the weight of the soil directly over the top of a buried pipe. The soil prism load per foot of alignment may be defined as:

$$W_{sp} = \gamma_s (H + 0.11B_c)B_c$$

where:

W_{sp} = soil prism load, lbs/ft
B_c = outside diameter of the pipe, ft
H = depth of fill over the pipe, ft
γ_s = total unit weight, pcf (no less than 110 pcf[1])

For example, assuming fill depths over the pipe ranging from 6 to 10 ft, a soil backfill total unit weight of 115 pcf, and an outside pipe diameter of 36 in., the approximate soil prism load ranges from 2,180 to 3,560 lbs per linear foot of alignment.

VEHICULAR TRAFFIC LOADS

The Project Civil Engineer should review anticipated traffic loading and frequencies to appropriately account for traffic loading and frequency for buried pipes crossing underneath or near roadways. We recommend using the simplified load distribution method suggested in the AASHTO Standard Specifications for Highway Bridges[1]. That is, AASHTO assumes the stress induced by traffic at the ground surface is uniformly distributed to an area with sides equal to 1-3/4 times the depth of fill above the buried pipe. As an example, a 16,000 lb load can be modeled with the following:

$$LiveLoad = {}^{P}\!\!\diagup\!_{(0.83 + 1.75H)(1.67 + 1.75H)}$$

where:

P = applied load (lbs)
H = depth of fill above the buried pipe (ft)

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[1] "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)," (2000) ASCE 15-98, American Society of Civil Engineers, Reston, VA, Section 9.1.2, Page 9.

Assuming a fill depth above the buried pipe of 6 ft, the resulting live load for a 16,000 lb load is about 117 psf.

PIPING SYSTEM THRUST RESTRAINT

Changes in fluid direction within the pipeline generate an increase in horizontal stress due to thrust forces along the pipe. This increase in horizontal stress translates into unbalanced forces that need to be resisted by the soil mass, typically by means of a concrete thrust block. A diagram for the calculation of pipe thrust resultant (unbalanced) force on a pipe bend and example calculation are presented on Figure 13.

Thrust blocks are often installed at pipe bends and or at a number of different pipe fittings such as tees, wyes, reducers, valves, offsets, etc., where unbalanced thrust forces are expected to be significant. The main purpose of the thrust blocks is to transfer the pipe thrust force to the soil structure. Thrust blocks allow for an increase of the area of contact between the soil and the pipe system, distributing the thrust load in a way that will not cause separation of unrestrained joints.

A convenient method to dimension a thrust block is based on the bearing capacity of the soils where the reaction is being generated. We recommend an allowable Equivalent Fluid Density (EFD) of 200 pcf or 400 pcf for blocks bearing a minimum of 2 ft into the hard, tan marl or the very stiff, tan clay, respectively, up to a maximum lateral pressure of 4,000 psf. The excavations should be observed by the Geotechnical Engineer or his qualified representative. If the thrust block is to be installed against disturbed soils, we recommend placing granular materials compacted to a minimum of 95 percent of maximum standard Proctor. Additional details in reference to fill materials and placement are presented in the Bedding and Backfill section below.

BEDDING AND BACKFILL

Bedding and backfill recommendations for the proposed storm sewer lines should be in accordance with San Antonio Water System (SAWS) Standard Specifications for Construction, Item 804 – Excavation, Trenching, and Backfill.

Bedding

Bedding is the material used along the bottom of the trench to provide uniform support for the buried pipe. Bedding may be compacted or uncompacted, depending on the recommendations of the design engineer. Bedding that is uncompacted allows the pipe to sink into the bedding soil allowing for a more uniform distribution of stress on the bottom of the pipe. When rock or other unyielding foundation material is encountered, a more compressible material should be used to bed the pipe.

Under installed conditions, the vertical load on a pipe is distributed over its width and the reaction is distributed in accordance with the type of bedding. When the pipe strength used in design has been determined by controlled, laboratory testing, a factor must be applied that relates the in-place supporting strength to those found in the lab. We recommend the pipe designer use a bedding factor to account for the width of the soil reaction at the bottom of the pipe.

Foundation

The bottoms of trench excavations should expose strong competent soils and should be dry and free of loose, soft, or disturbed soil. Soft, wet, weak, or deleterious materials should be over-excavated to expose strong competent soils. At locations where soft or weak soils extend for some depth, overexcavation to stronger soils may prove infeasible and/or uneconomical. In the event of encountering these areas of deep soft or weak soils, we recommend that the bottom of the trench excavation be over-excavated by 1 to 2 ft, and replaced with an open-graded aggregate that will allow for drainage of water, as well as provide a stable working platform.

Materials

The bedding materials should be selected to ensure the most uniform contact between the pipe and the foundation as possible. The bedding should be selected and placed in accordance with SAWS Item 804.

Backfill

We recommend that backfill material selection and placement be in accordance with SAWS Item 804. In addition, backfill for trenches should not be started until the pipeline is properly bedded in accordance with the above recommendations. Materials removed from the trench excavations will generally be suitable as backfill, provided they are not saturated and do not contain organic matter, debris, or other deleterious material.

To reduce potential settlements of the ground surface resulting from consolidation of the trench backfill, we recommend that trench backfill be placed in 6-in. thick loose lifts and compacted to at least 95 percent of the maximum dry density as determined by ASTM D 698.

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 foundations and to facilitate rapid drainage away from the foundations. Failure to provide positive drainage away from the structure can result in localized differential vertical movements in soil supported foundations.

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

SITE PREPARATION

The foundation areas and all areas to support select fill should be stripped of all vegetation and organic topsoil. Furthermore, as discussed in a previous section of this report, we recommend that 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 foundation 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 from TxDOT, Tex-114-E, 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 final site grading preferably should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2004 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Types A or C, Grades 1 through 3.

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

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 TxDOT, Tex-113-E, Compaction Test. 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.

SHALLOW FOUNDATION EXCAVATIONS

Shallow foundation excavations should be observed by the Geotechnical Engineer or his representative prior to placement of reinforcing steel and concrete. This is necessary to verify that the bearing soils 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 pockets of soil 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

Due to the shallow nature of the surficial soils, excavations at this site will require removal of the underlying rock (like) formation. Rock may also be encountered at the surface in some areas of this site. Thus, the need of rock excavation equipment should be anticipated for construction at this site. 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

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 (such as fractures within a rock mass or at contacts between rock and clay formations). 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.
- Curbs should completely penetrate base materials and be installed to a sufficient depth to reduce water infiltration beneath the curbs into the pavement base materials.
- 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.

BURIED PIPE CONSTRUCTION CONSIDERATIONS

TRENCH SAFETY

If utility trenches or other excavations extend to or beyond 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, is 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.

Critical Height

Cuts in clays will stand with vertical slopes for a period of time before failure occurs. However, changes in the shear strength of the clay with time and stress release resulting from the excavation can lead to progressive deterioration of stability. This process can be rapid in stiff, fissured clays and slower in softer clays.

Bearing Pressures

The stability of the bottom of excavations is dependent on the excavation geometry, soil strength parameters of the bearing soils, and most importantly the location of groundwater. Excavations within cohesive soils are not susceptible to a reduction in effective stress conditions due to the relatively short period of time excavations are open. That is, the stress conditions within cohesive soils generally do not move from an undrained (short-term) to drained (long-term) condition. As such, the bottom stability of excavations within cohesive soils is controlled by the shear strength of the bearing soils. Figure 14 presents the Department of the Navy's method[2] for determining the factor of safety for base stability of excavations in cohesive soils.

It is important to note that the above discussion does not consider layered soil profiles. For example, a charged granular layer that lies below a clay layer will cause significant seepage pressures on the bottom of an excavation within the overlying clay layer, especially as the excavation approaches the granular layer. When encountered, these areas must be considered on a case-by-case basis by the Geotechnical Engineer.

Bracing Pressures

In order to properly design the supports for shielding workers within an excavation, the type of shoring planed for use must be known as well as the geometry (i.e., the vertical spacing of struts). Once the Contractor has chosen a method for shoring, the bracing pressures can be determined using a cohesion value of 1,600 psf for the clays. Figure 15 presents the Department of the Navy's method[3] for determining the pressure distribution for internally braced flexible walls.

---

[2] "Foundations and Earth Structures," Department of the Navy, (1982), Naval Facilities Engineering Command, DM-7.2, Alexandria, VA, p 7.2-104.

[3] "Foundations and Earth Structures," Department of the Navy, (1982), Naval Facilities Engineering Command, DM-7.2, Alexandria, VA, p 7.2-100.

Figure 16 presents an example calculation for the factor of safety of base stability, the bracing pressures, and the force applied on a buried length of sheeting.

Lateral Earth Pressures

Equivalent fluid density values for computation of lateral soil pressures acting on temporary retention systems were evaluated for the natural materials that were encountered in our borings. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented below.

Back Fill Type	Estimated Total Unit Weight (pcf)	Active Condition		At Rest Condition	
		Earth Pressure Coefficient, k_a	Equivalent Fluid Density (pcf)	Earth Pressure Coefficient, k_o	Equivalent Fluid Density (pcf)
Clay	120	0.59	70	0.74	90
Marl	135	0.24	30	0.38	50

The values tabulated above under "Active Conditions" pertain to flexible retention systems free to tilt inward as a result of lateral earth pressures. For rigid, non-yielding walls the values under "At-Rest Conditions" should be used.

The values presented above do not include the effect of surcharge loads such as construction equipment, vehicular loads, or storage near the structures. Nor do the values account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the cut soils. However, these surcharge loads and groundwater pressures should be considered, if applicable, in designing any structures subjected to lateral earth pressures.

The above recommendations are for temporary retention systems to be in use 30 days or less. If the retention systems are to be in place for longer than 30 days, higher earth pressures may be warranted.

EXCAVATION DEWATERING

Typically, the Contractor is responsible for designing, installing and maintaining a dewatering system for groundwater control and taking precautions to avoid distress to nearby existing structures, as a result of dewatering. We recommend the Contractor consider retaining a dewatering expert to assist in identifying, implementing and monitoring the most suitable and cost-effective method to control groundwater.

In cohesive soils where seepage is usually low, groundwater is generally managed by collection in trench bottom sumps for pumped disposal. Care should be taken to have a redundant pumping system that allows for overnight pumping. Water must not be allowed to pond in the trench bottoms. The softening of soils can lead to instability and sloughing of trench side walls. In addition, if cohesive soils contain lenses/layers of water-bearing granular or cohesionless soils, they may have to be dewatered using techniques for cohesionless soils.

Generally, the groundwater depth should be lowered to a depth of at least 3 feet below the planned excavation bottom to provide a firm working surface. Extended and/or extensive dewatering can result in settlement of existing structures in the vicinity; the Contractor is to take necessary precautions to minimize the effects on these structures.

Based on the results of our borings, we do not anticipate encountering groundwater seepage during construction. However, seasonal variations and/or unforeseen environmental conditions may result in fluctuations in the depth-to-water. We suggest the Contractor provide a line item for dewatering in the bid package in the event that dewatering is required.

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, Raba-Kistner, is retained to perform construction observation and testing services during the construction of the project. This is because:

- **R-K** has an intimate understanding of the geotechnical engineering report's findings and recommendations. **R-K** understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- **R-K** knows what subsurface conditions are anticipated at the site.
- **R-K** 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 **R-K** to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- **R-K** 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.
- **R-K** 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 **R-K** 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. **R-K** 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.

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

The following figures are attached and complete this report:

ATTACHMENTS

Raba Kistner

Engineering • Testing • Environmental
Facilities • Infrastructure

12821 West Golden Lane
San Antonio, Texas 78249
(210)699-9090 TEL
(210)699-6426 FAX
www.rkci.com
TBPE Firm Number 3257

SOURCE: 2009 Aerial Photograph Provided by The City of San Antonio (COSA)

BORING LOCATION MAP

HIDDEN SPRINGS WATER IMPROVEMENTS
SAN ANTONIO, TEXAS

REVISIONS:

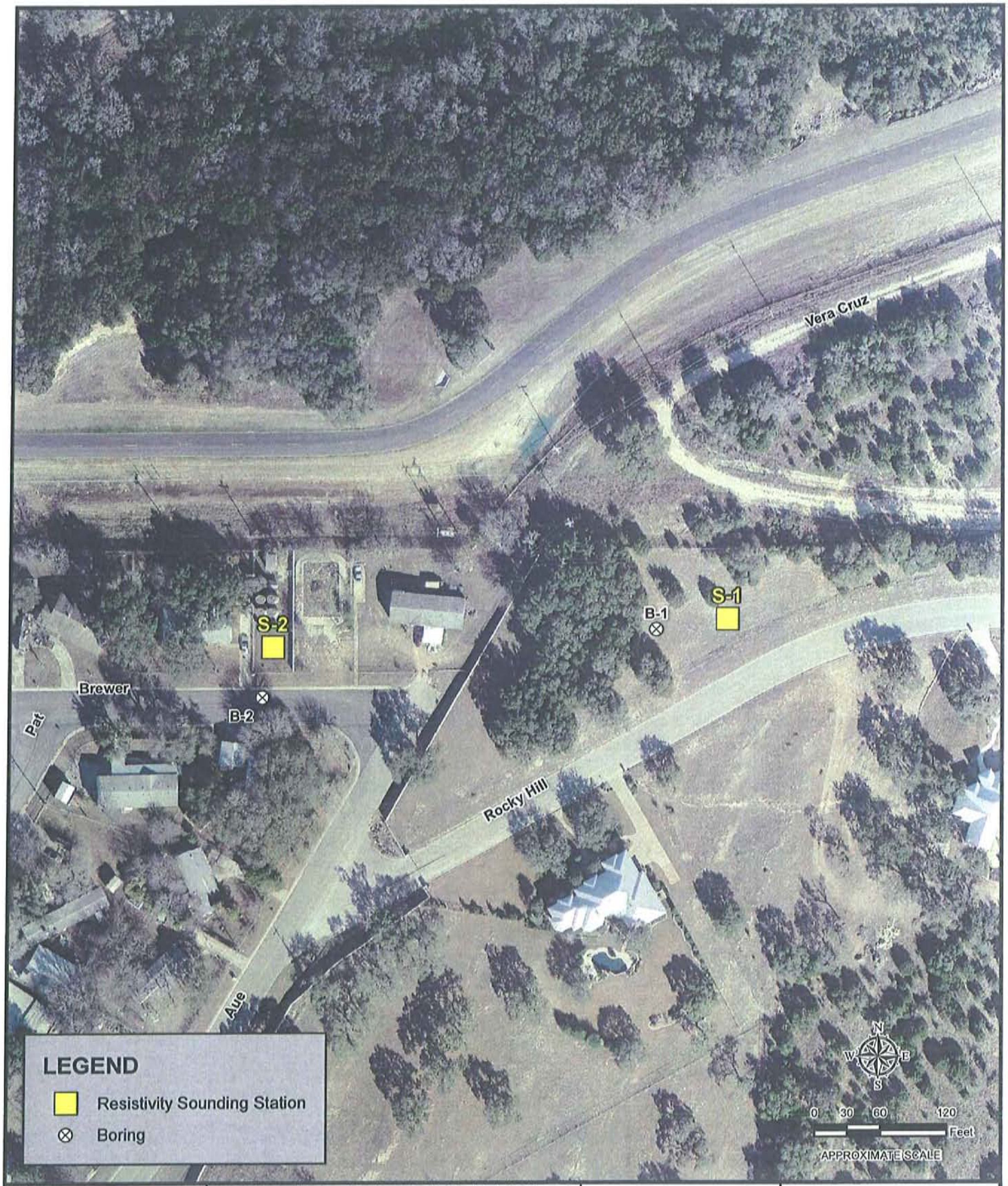
No.	DATE	DESCRIPTION

PROJECT No.:
ASA11-032-00

ISSUE DATE:	3-28-11
DRAWN BY:	CCL
CHECKED BY:	TIP
REVIEWED BY:	GLB

FIGURE

1



Raba Kistner

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TBPE Firm Number 3257

SOURCE: 2009 Aerial Photograph Provided by The City of San Antonio (COSA)

RESISTIVITY SOUNDING STATION LOCATION MAP

HIDDEN SPRINGS WATER IMPROVEMENTS
SAN ANTONIO, TEXAS

REVISIONS:

No.	DATE	DESCRIPTION

PROJECT No.:	ASA11-032-00
ISSUE DATE:	3-30-11
DRAWN BY:	CCL
CHECKED BY:	TIP
REVIEWED BY:	GLB
FIGURE	2

LOG OF BORING NO. B-1

Hidden Springs Water Improvements
San Antonio, Texas

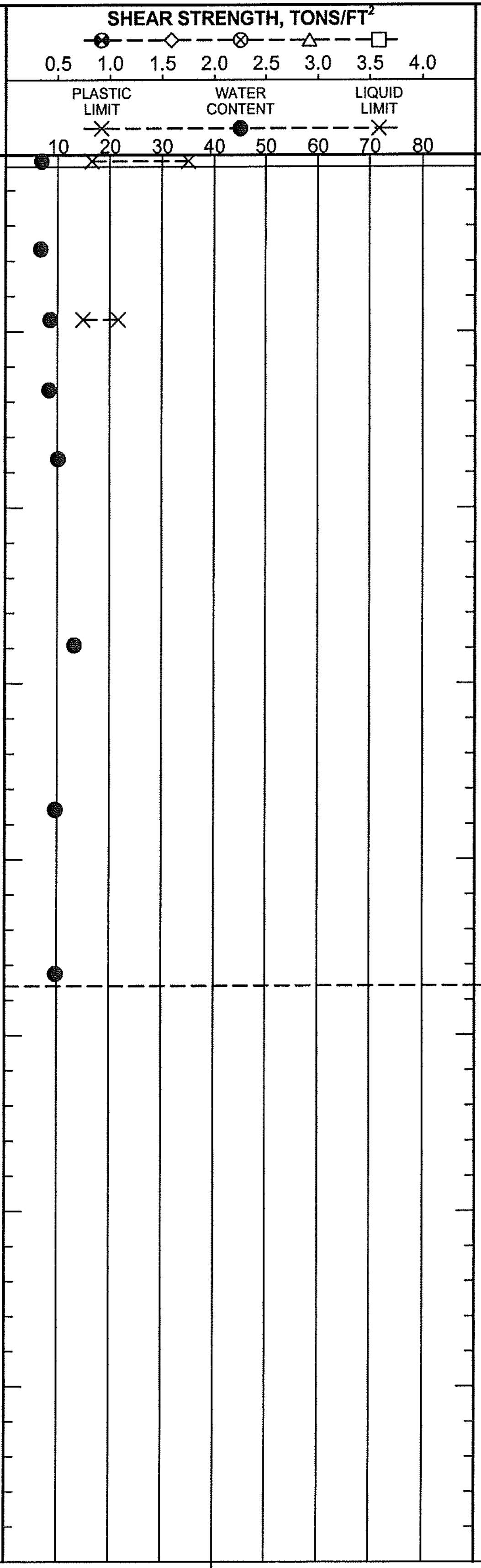
Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67666; W 98.62493

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	CLAY, Very Stiff, Dark Brown, with limestone fragments	21		18	
	MARL, Hard, Tan, with calcareous deposits	ref/4"			
5		ref/4"		7	
		ref/4"			
		ref/3"			
	- clayey seam at 13.5 ft	50/4"			
		ref/2"			
		ref/1"			
	Boring Terminated				

NOTES:
Redrilled to 23.6 ft on 05/04/11



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

DEPTH DRILLED: 23.6 ft
DATE DRILLED: 4/5/2011

DEPTH TO WATER: Dry
DATE MEASURED: 4/5/2011

PROJ. No.: ASA11-032-00
FIGURE: 3

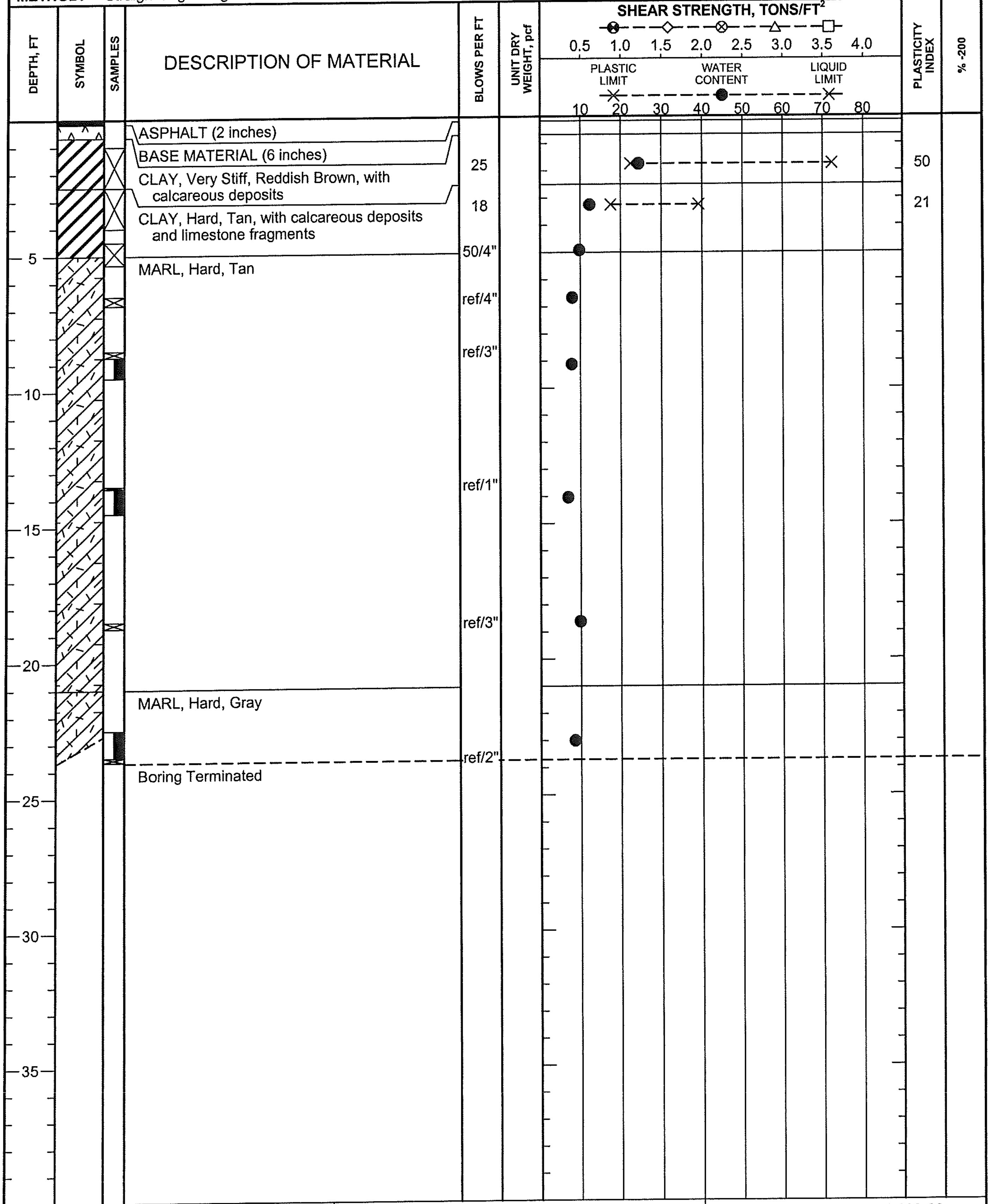
LOG OF BORING NO. B-2

Hidden Springs Water Improvements
San Antonio, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67653; W 98.62613



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
ASPHALT (2 inches)				
BASE MATERIAL (6 inches)				
CLAY, Very Stiff, Reddish Brown, with calcareous deposits	25		50	
CLAY, Hard, Tan, with calcareous deposits and limestone fragments	18		21	
MARL, Hard, Tan	50/4" ref/4" ref/3" ref/1" ref/3"			
MARL, Hard, Gray	ref/2"			
Boring Terminated				

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

DEPTH DRILLED: 23.7 ft
DATE DRILLED: 4/21/2011

DEPTH TO WATER: Dry
DATE MEASURED: 4/21/2011

PROJ. No.: ASA11-032-00
FIGURE: 4

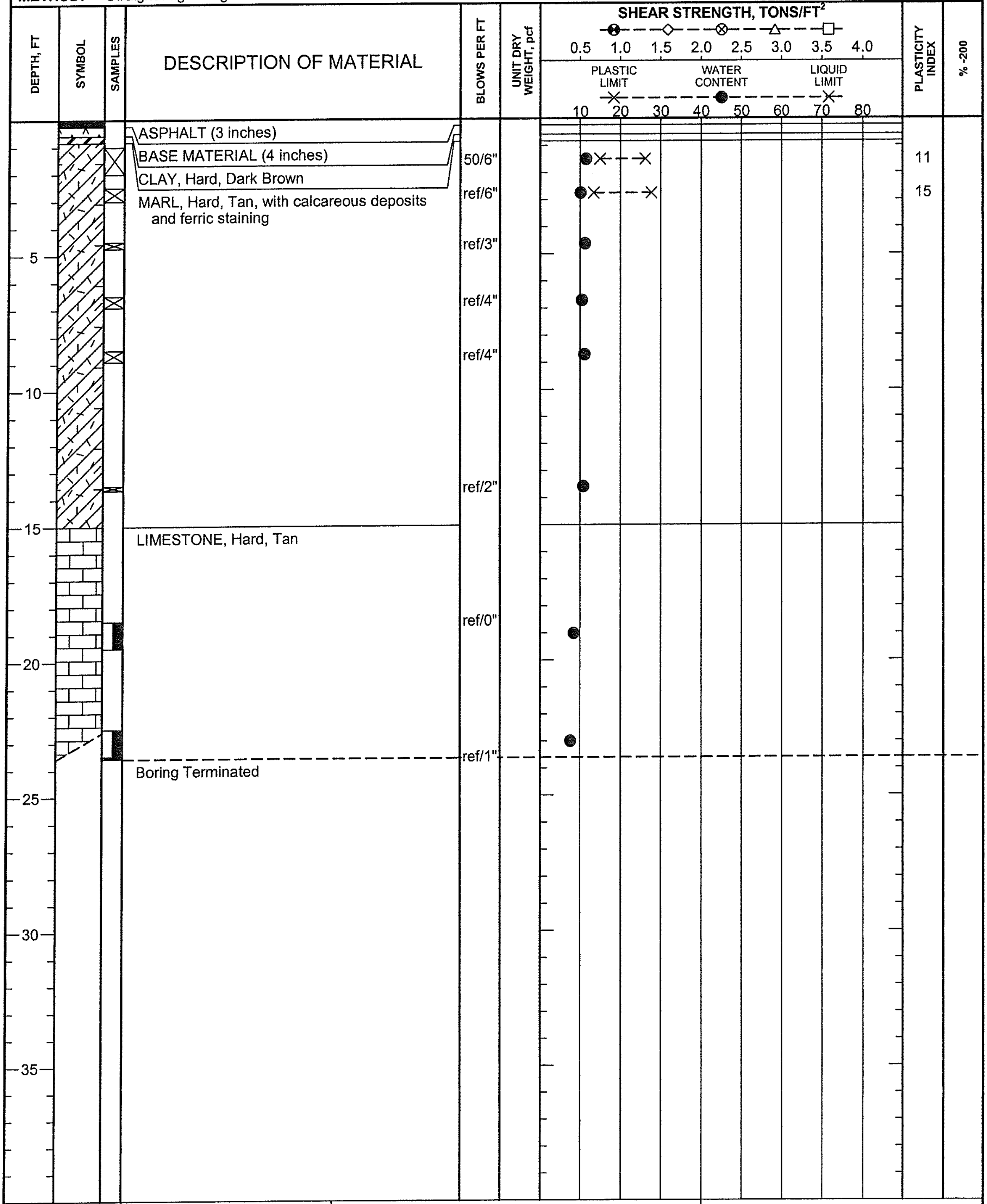
LOG OF BORING NO. B-3

Hidden Springs Water Improvements
San Antonio, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67235; W 98.62894



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

DEPTH DRILLED: 23.6 ft
DATE DRILLED: 4/21/2011

DEPTH TO WATER: Dry
DATE MEASURED: 4/21/2011

PROJ. No.: ASA11-032-00
FIGURE: 5

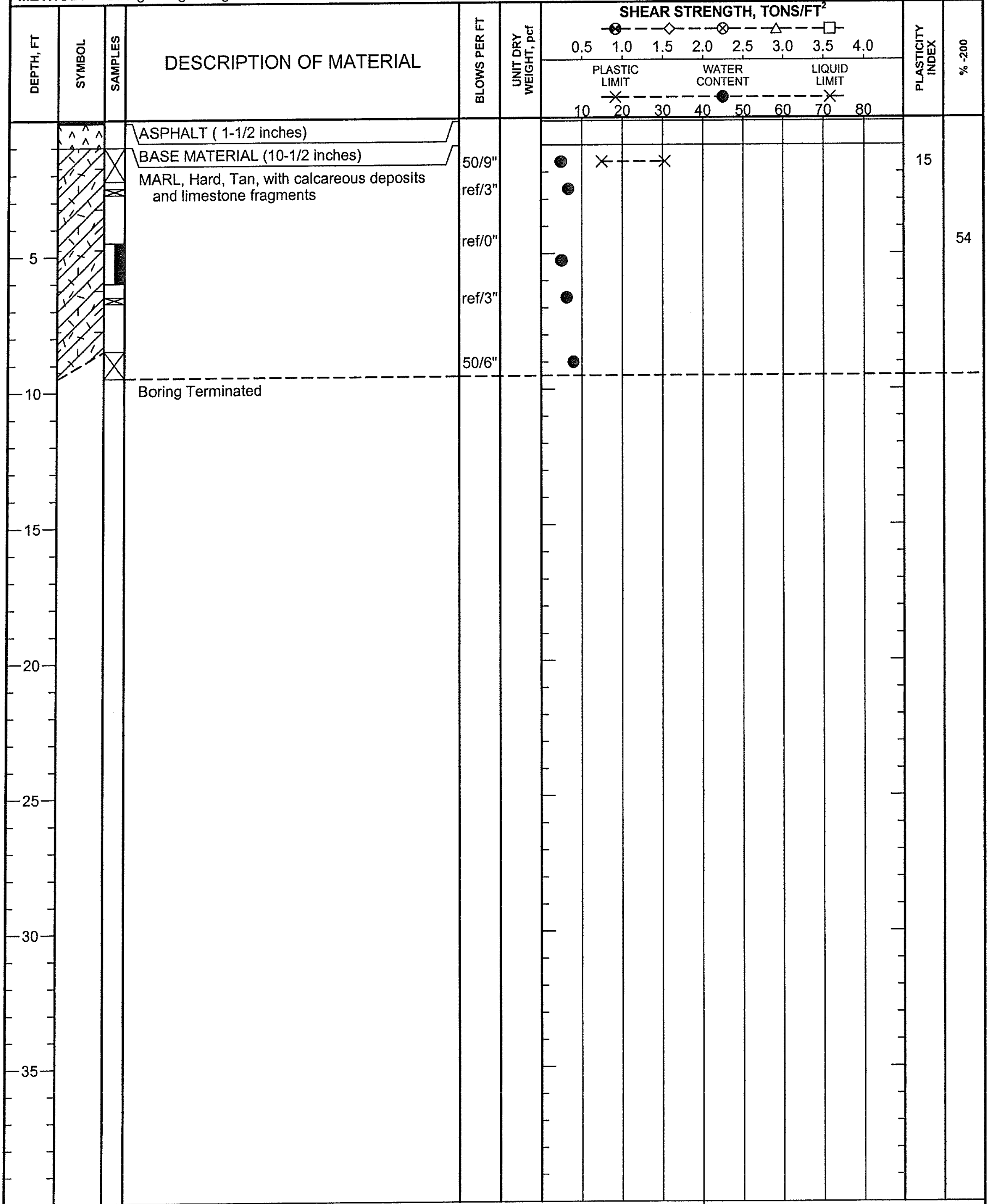
LOG OF BORING NO. B-4

Hidden Springs Water Improvements
San Antonio, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67363; W 98.62666



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
ASPHALT (1-1/2 inches)				
BASE MATERIAL (10-1/2 inches)	50/9"		15	
MARL, Hard, Tan, with calcareous deposits and limestone fragments	ref/3"			
	ref/0"			54
	ref/3"			
	50/6"			
Boring Terminated				

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

DEPTH DRILLED: 9.5 ft
DATE DRILLED: 4/5/2011

DEPTH TO WATER: Dry
DATE MEASURED: 4/5/2011

PROJ. No.: ASA11-032-00
FIGURE: 6

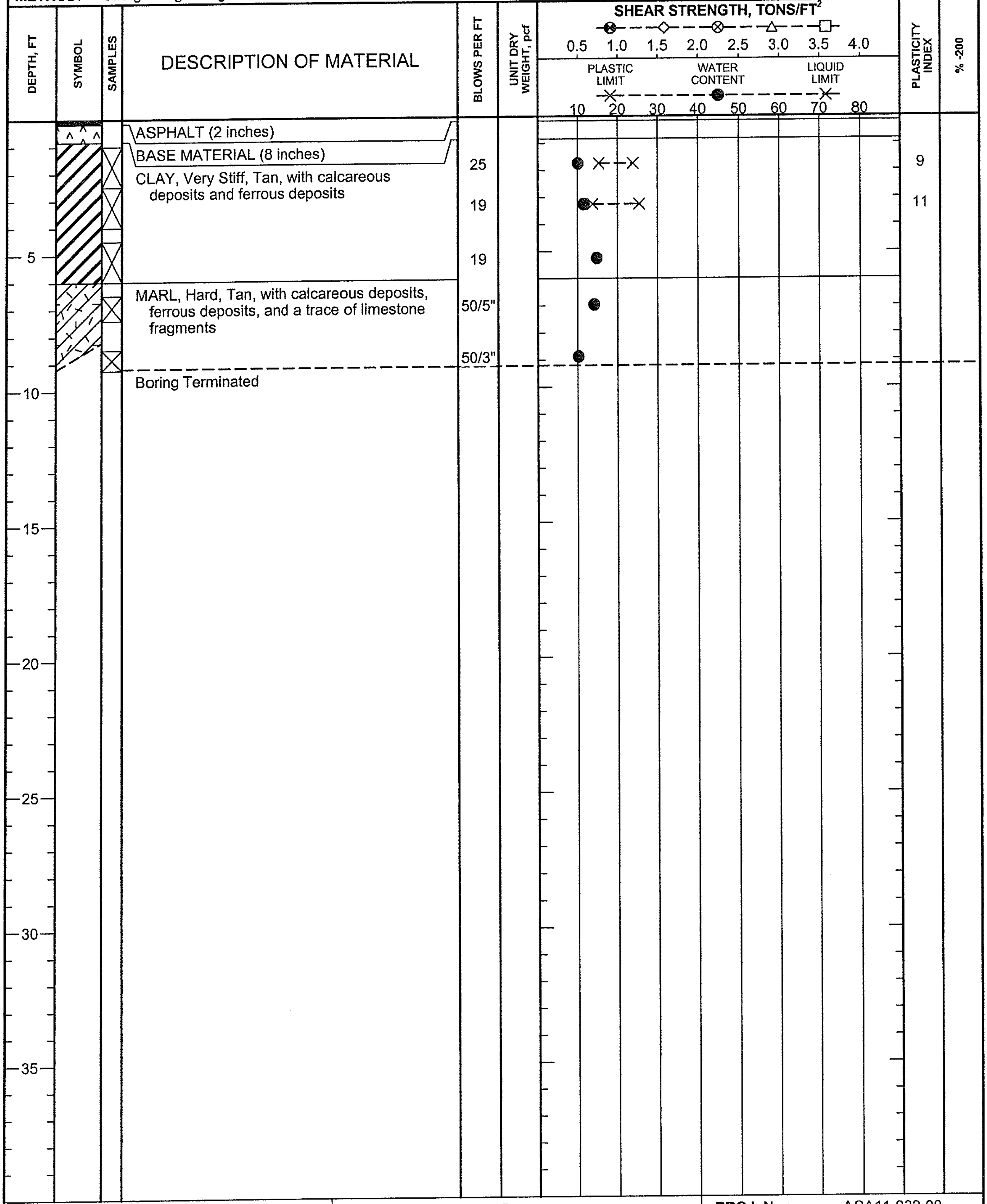
LOG OF BORING NO. B-5

Hidden Springs Water Improvements
San Antonio, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67423; W 98.62276



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
ASPHALT (2 inches)				
BASE MATERIAL (8 inches)				
CLAY, Very Stiff, Tan, with calcareous deposits and ferrous deposits	25		9	
	19		11	
	19			
MARL, Hard, Tan, with calcareous deposits, ferrous deposits, and a trace of limestone fragments	50/5"			
	50/3"			
Boring Terminated				

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

DEPTH DRILLED:	9.2 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA11-032-00
DATE DRILLED:	4/5/2011	**DATE MEASURED:**	4/5/2011	**FIGURE:**	7

LOG OF BORING NO. B-6

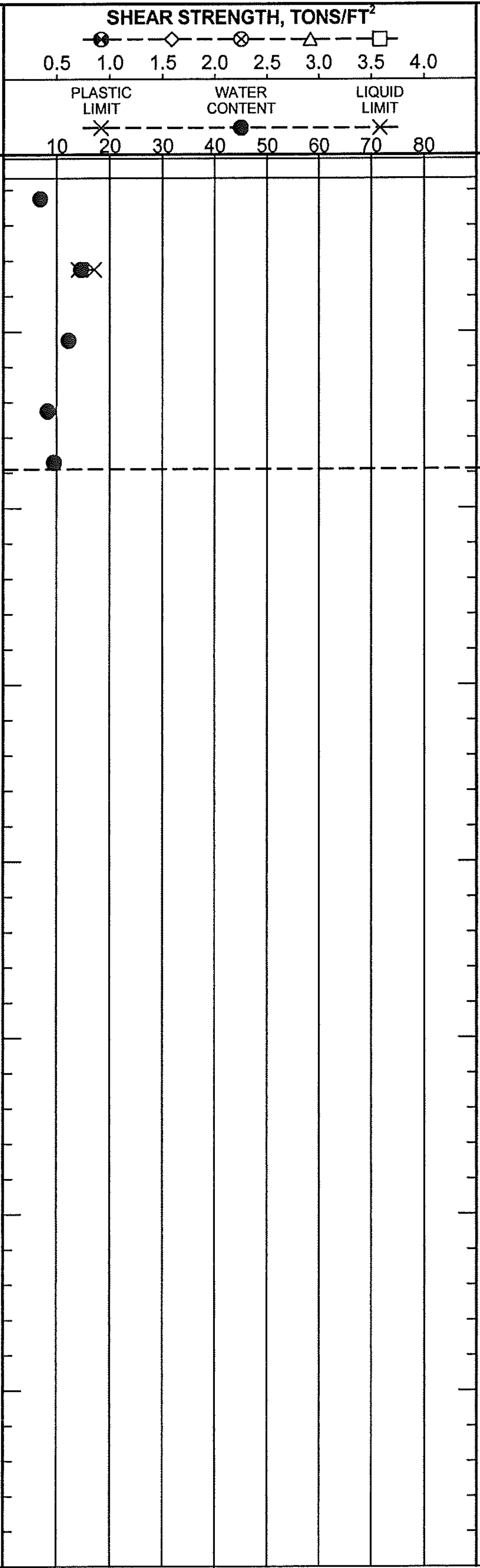
Hidden Springs Water Improvements
San Antonio, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67637; W 98.62023

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
ASPHALT (1-1/2 inches)				
BASE MATERIAL (6-1/2 inches)	ref/6"			
CLAY, Very Stiff to Hard, Tan, with calcareous deposits	25		3	
	47			
- with limestone fragments below 6 ft	23			45
- marl below 8 ft	ref/5"			
Boring Terminated				



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

DEPTH DRILLED: 8.9 ft
DATE DRILLED: 4/5/2011
DEPTH TO WATER: Dry
DATE MEASURED: 4/5/2011
PROJ. No.: ASA11-032-00
FIGURE: 8

LOG OF BORING NO. B-7

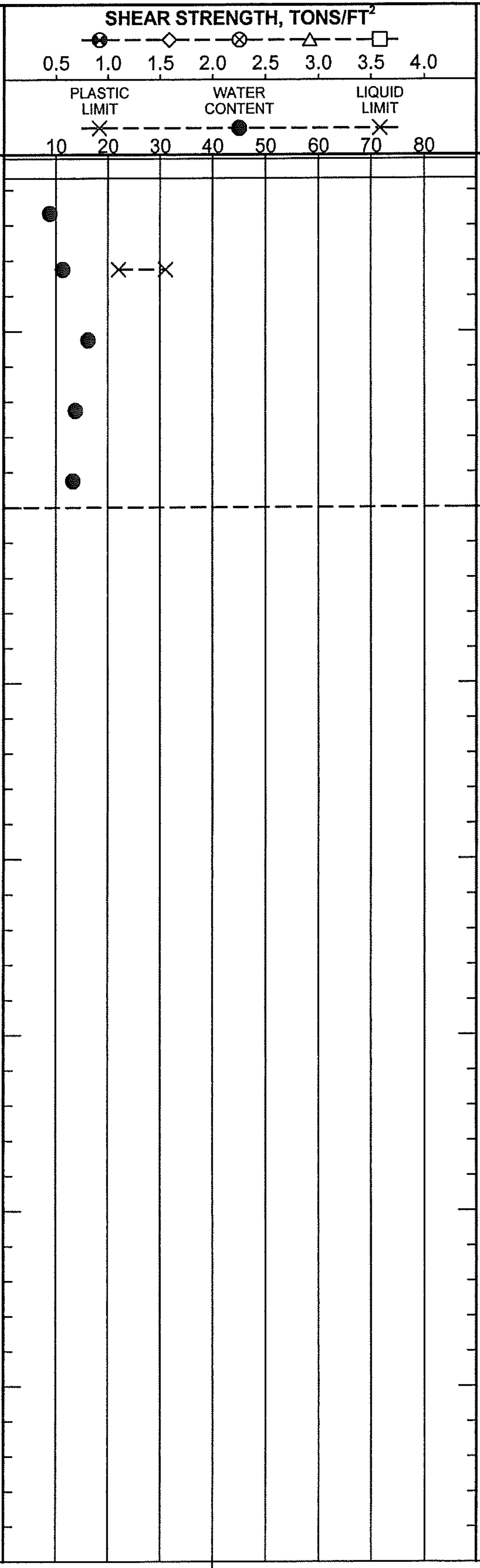
Hidden Springs Water Improvements
San Antonio, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67485; W 98.61839

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	ASPHALT (1-1/2 inches)				
	BASE MATERIAL (6-1/2 inches)				
	CLAY, Hard, Tan, with calcareous deposits, ferrous deposits, and limestone fragments	50/10"			
		31		9	
5		49			
		43			78
		50			
10	Boring Terminated				



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

DEPTH DRILLED: 10.0 ft
DATE DRILLED: 4/5/2011

DEPTH TO WATER: Dry
DATE MEASURED: 4/5/2011

PROJ. No.: ASA11-032-00
FIGURE: 9

LOG OF BORING NO. B-8

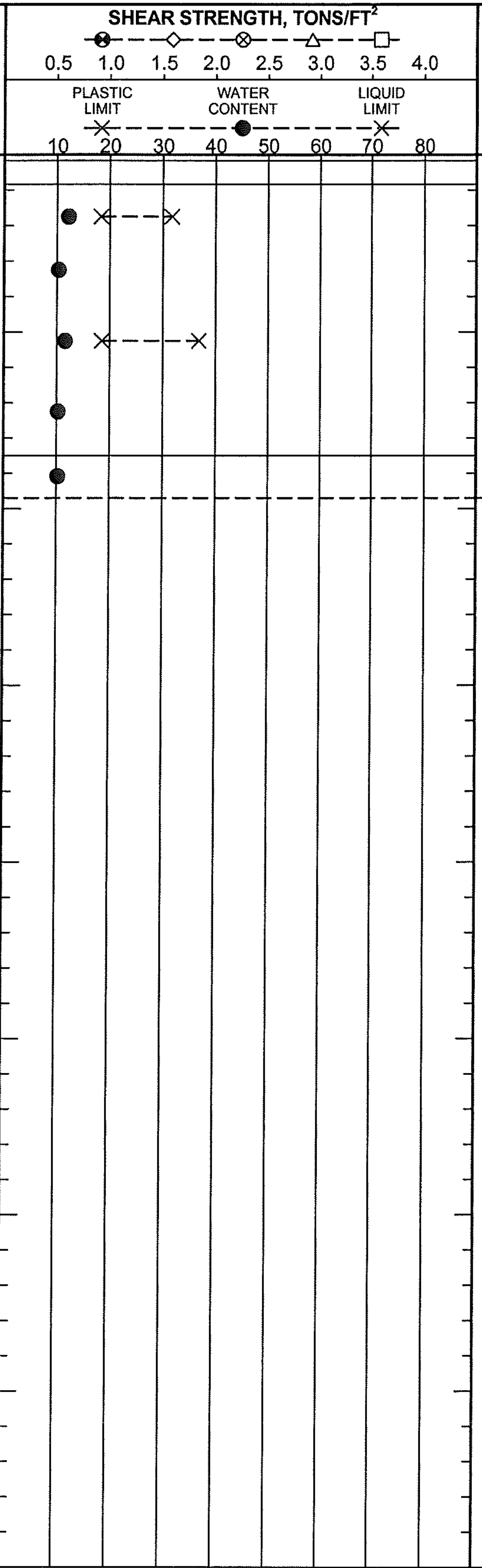
Hidden Springs Water Improvements
San Antonio, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.67249; W 98.61724

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			ASPHALT (2 inches)				
			BASE MATERIAL (8 inches)				
			CLAY, Very Stiff to Hard, Tan, with calcareous deposits, limestone fragments, and a trace of sand	19		14	
				15			
5				15		19	
			- with limestone fragments below 6 ft				
				16			40
			MARL, Hard, Tan, with limestone fragments	50/8"			
10			Boring Terminated				
15							
20							
25							
30							
35							

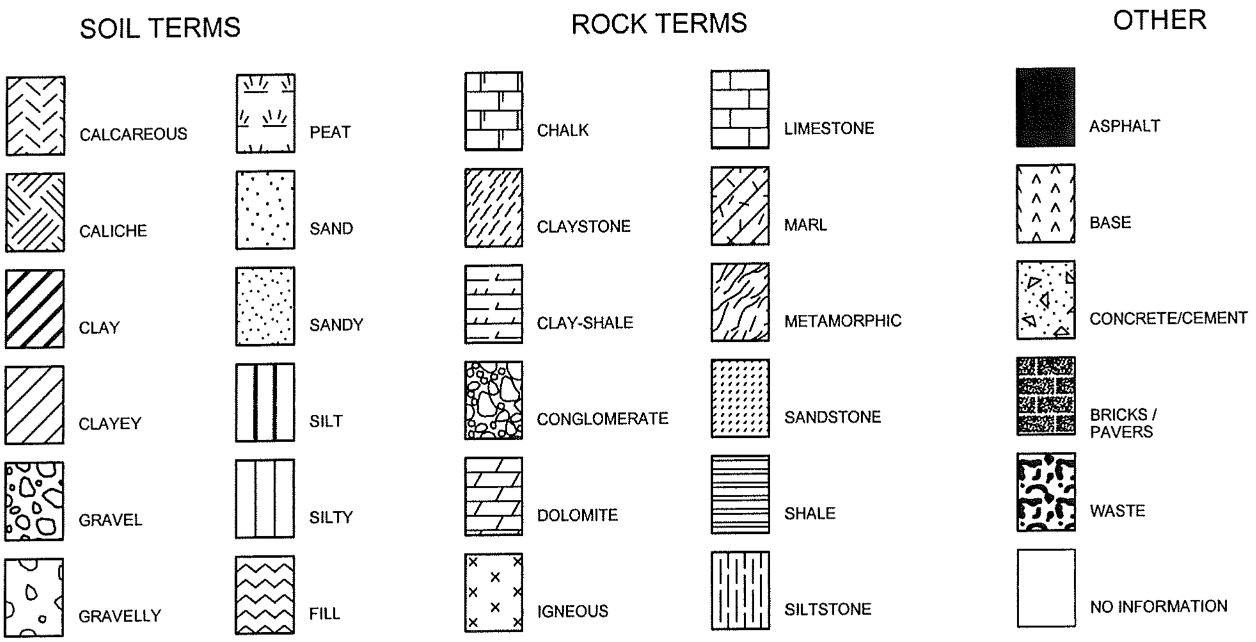


NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

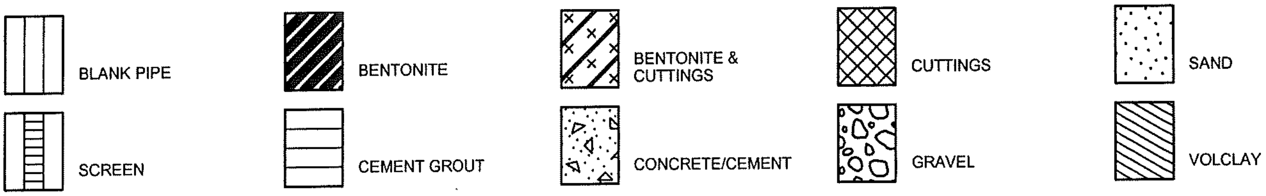
DEPTH DRILLED:	9.7 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA11-032-00
DATE DRILLED:	4/5/2011	**DATE MEASURED:**	4/5/2011	**FIGURE:**	10

KEY TO TERMS AND SYMBOLS

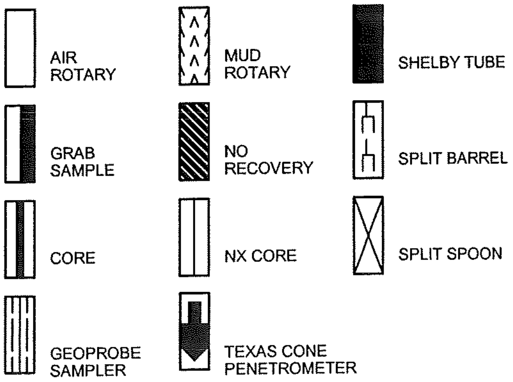
MATERIAL TYPES



WELL CONSTRUCTION AND PLUGGING MATERIALS



SAMPLE TYPES



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. ASA11-032-00

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
El = 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-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-BARRELL 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.

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

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Hidden Springs Water Improvements
San Antonio, Texas

FILE NAME: ASA11-032-00.GPJ

5/5/2011

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	21									
	0.0 to 0.3		7	35	17	18	CL				
	2.5 to 2.8	ref/4"	7								
	4.5 to 4.8	ref/4"	9	22	15	7	CL-ML				
	6.5 to 6.8	ref/4"	8								
	8.5 to 8.7	ref/3"	10								
	13.5 to 14.3	50/4"	13								
	18.5 to 18.7	ref/2"	10								
	23.0 to 23.5		10								
	23.5 to 23.6	ref/1"									
B-2	1.0 to 2.5	25	24	72	22	50	CH				
	2.5 to 4.0	18	12	39	18	21	CL				
	4.5 to 5.3	50/4"	10								
	6.5 to 6.8	ref/4"	8								
	8.5 to 8.7	ref/3"									
	8.7 to 9.5		8								
	13.5 to 13.6	ref/1"									
	13.6 to 14.5		7								
	18.5 to 18.7	ref/3"	10								
	22.5 to 23.5		8								
	23.5 to 23.7	ref/2"									
B-3	1.0 to 2.0	50/6"	11	26	15	11	CL				
	2.5 to 3.0	ref/6"	10	28	13	15	CL				
	4.5 to 4.7	ref/3"	11								
	6.5 to 6.9	ref/4"	10								
	8.5 to 8.9	ref/4"	11								
	13.5 to 13.7	ref/2"	11								
	18.5 to 18.5	ref/0"									
	18.5 to 19.5		8								
	22.5 to 23.5		8								
	23.5 to 23.6	ref/1"									
B-4	1.0 to 2.2	50/9"	5	30	15	15	CL				
	2.5 to 2.7	ref/3"	7								
	4.5 to 4.5	ref/0"							54		
	4.5 to 6.0		5								
	6.5 to 6.7	ref/3"	6								
	8.5 to 9.5	50/6"	8								
B-5	1.0 to 2.5	25	10	24	15	9	CL				
	2.5 to 4.0	19	12	25	14	11	CL				

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

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA11-032-00

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Hidden Springs Water Improvements
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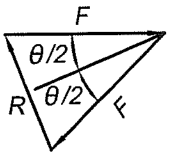
Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-5	4.5 to 6.0	19	15								
	6.5 to 7.4	50/5"	14								
	8.5 to 9.3	50/3"	10								
B-6	1.0 to 1.5	ref/6"	7								
	2.5 to 4.0	25	15	17	14	3	ML				
	4.5 to 6.0	47	12								
	6.5 to 8.0	23	8						45		
	8.5 to 8.9	ref/5"	10								
B-7	1.0 to 2.3	50/10"	9								
	2.5 to 4.0	31	11	31	22	9	CL				
	4.5 to 6.0	49	16								
	6.5 to 8.0	43	14						78		
	8.5 to 10.0	50	13								
B-8	1.0 to 2.5	19	12	32	18	14	CL				
	2.5 to 4.0	15	10								
	4.5 to 6.0	15	12	37	18	19	CL				
	6.5 to 8.0	16	10						40		
	8.5 to 9.7	50/8"	10								

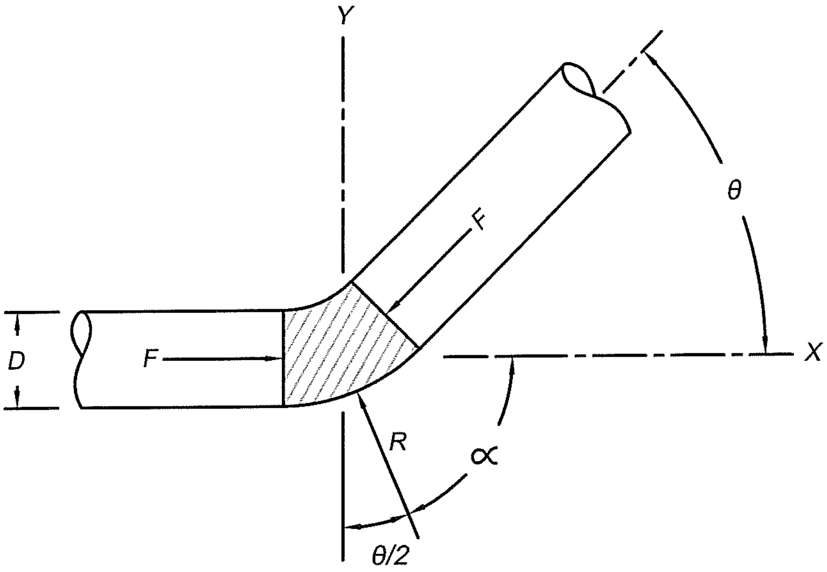
PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA11-032-00



Free vector diagram



$$R = 2 * F * SIN(\theta/2)$$
$$R_x = F * (1 - COS\theta)$$
$$R_y = F * SIN\theta$$
$$\alpha = 90 - (\theta/2)$$

Where:

R = resultant thrust force
R_x = thrust force component along the X axis
R_y = thrust force component along the Y axis
F = impulse force = $P * A$
P = maximum sustained pressure
A = cross-sectional area of pipe = $(\pi * D^2)/4$
D = internal diameter conduit
θ = angle of bend
α = angle between X axis and R

THRUST CALCULATION EXAMPLE

Given:

$D = 20$ in
$P = 200$ psi
$\theta = 45^\circ$

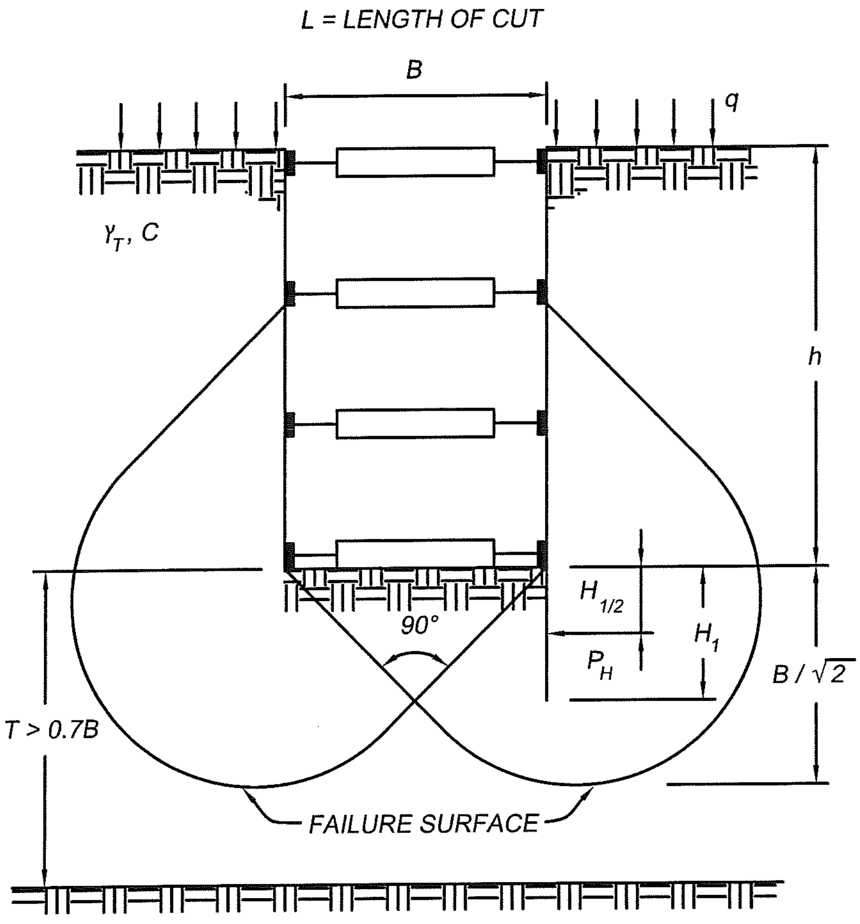
Determine:

R, R_x and R_y

Procedure:

$A = (\pi * 20^2)/4 = 314.16$ in^2
$F = 200 * 314.16 = 62{,}832$ lbs
$R = 2 * 62{,}832 * SIN(45/2) = 48{,}090$ lbs
$R_x = 62{,}832 * (1 - COS45) = 18{,}403$ lbs
$R_y = 62{,}832 * SIN45 = 44{,}428$ lbs

THRUST FORCE RESTRAINT



When sheeting terminates at the base of cut:

$$FS = \frac{N_c C}{\gamma_T h + q}$$

where: N_c = Terzaghi's Bearing Capacity Factor
C = Undrained Shear Strength of Clay in Failure Zone
Q = Surcharge Load

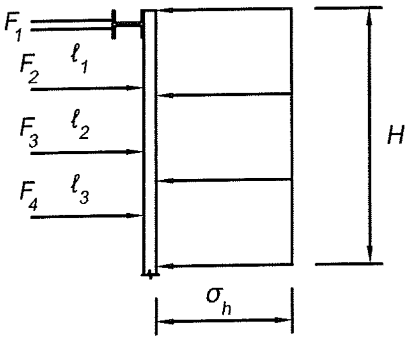
If factor of safety is less than 1.5, sheeting must be carried below the base of cut to insure stability. The force, P_H, on the buried length of sheeting can be calculated by:

IF $H_1 > \frac{2}{3}\frac{B}{\sqrt{2}}$, then $P_H = 0.7(\gamma_T hB - 1.4CH - \pi CB)$

IF $H_1 \leq \frac{2}{3}\frac{B}{\sqrt{2}}$, then $P_H = 1.5H_1\left(\gamma_T h - \frac{1.4CH}{B} - \pi C\right)$

BASE STABILITY FOR BRACED CUT IN CLAY

Ref: Department of the Navy, (1982) "Foundations and Earth Structures"
Naval Facilities Engineering Command, Alexandria, VA

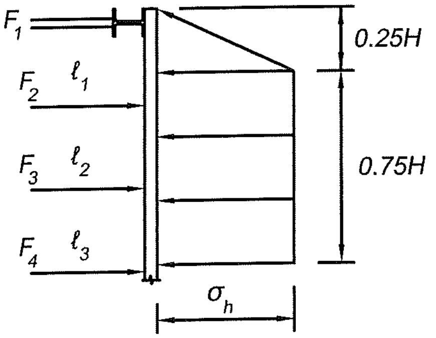


SAND PROFILE

$$\sigma_H = 0.65 K_A \gamma H$$

where

$$K_A = \tan^2\left(45 - \phi/2\right)$$



$F_3 = (\frac{\ell_2}{2} + \frac{\ell_3}{2})\, \sigma_h$

ASSUME HINGES AT STRUT LOCATIONS FOR CALCULATING STRUT FORCES

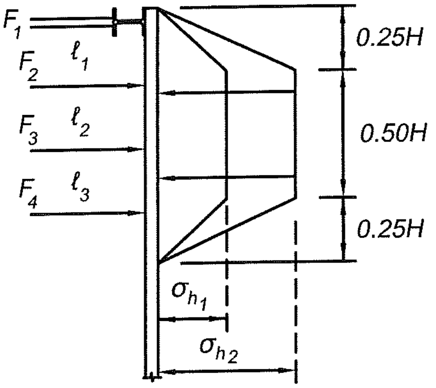
SOFT TO MEDIUM CLAY PROFILE
($N_o > 6$)

For clays base the selection on $N_O = \gamma H / c$

$$\sigma_H = K_A \gamma H$$

$$K_A = 1 - m \frac{4c}{\gamma H}$$

m = 1 except where cut is underlain by deep soft normally consolidated clay, then m = 0.4FS (against bottom instability)



STIFF CLAY PROFILE
($N_o < 4$)

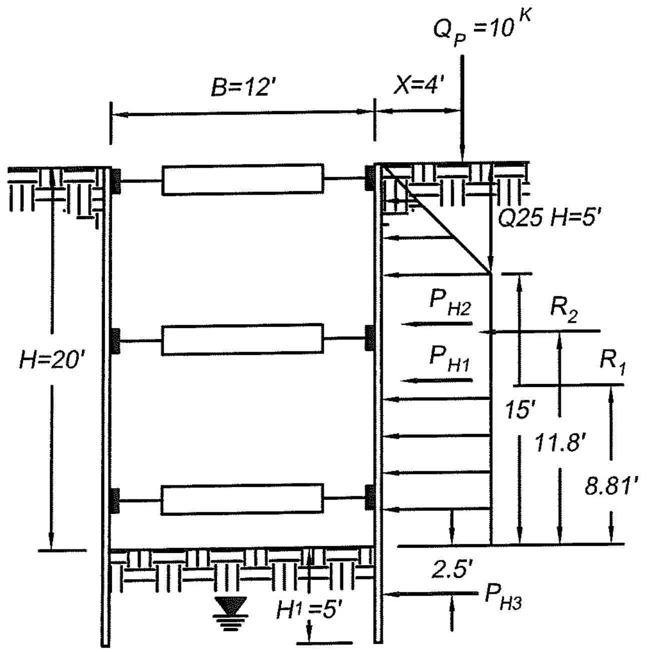
For 4 < N_o <6, use the larger diagram representing soft to medium clay or stiff clay.

$$\sigma_{h1} = 0.2\gamma H \; ; \quad \sigma_{h2} = 0.4\gamma H$$

Use lower value (σ_{h1} or σ_{h2}) when movements are minimal and short construction period.

PRESSURE DISTRIBUTION FOR INTERNALLY BRACED FLEXIBLE WALLS

Ref: Department of the Navy, (1982) "Foundations and Earth Structures"
Naval Facilities Engineering Command, Alexandria, VA



PRESSURE ON WALL FROM SOIL

$K_A = 1 - m\frac{4c}{\gamma H}$; Where m = 0.4 FS = 0.4 x 1.18 = 0.47; and $K_A = 1 - (0.47)\left(\frac{4 \times 400}{120 \times 20}\right) = 0.69$

$$\sigma_h = K_A \gamma H = 0.69 \times 0.12 \times 20 = 1.66 ksf$$

$$P_{H1} = \frac{(15+20)(1.66)}{2} = 29.05 kips$$

LOCATION OF RESULTANT

$$R_1 = \frac{1.66 \times 5/2 \times (15 + 5/3) + 1.66 \times 15 \times 15/2}{29.05} = 8.81 ft$$

PRESSURES ON WALL FROM SURCHARGE

$$m = \frac{x}{H} = \frac{4}{20} = 0.2$$

$$P_{H2} = 0.78\frac{Q_P}{H} = 0.78\frac{10}{20} = 0.39 kips$$

GIVEN CONDITIONS:

Excavation in SILTY CLAY

C = 400 psf; $\phi = 0$; $\gamma_T = 120$ pcf

L = length of excavation (into the page) = 80 ft

DETERMINE:

Factor of safety w/ respect to base stability; bracing pressures; force on buried sheeting

STABILITY OF BASE OF CUT

$FS = \frac{N_c c}{\gamma_T h + q}$; q = 0 (no uniform surcharge)

when $H/B = z/B = 20/12 = 1.67$ and

$B/L = 12/80 = 0.15$, then $N_c = 6.9_{(square)}$

$N_{c(rectangular)} = N_{c(square)}$ x (1+0.2 B/L) = 7.1

$$FS = \frac{7.1 \times 400}{120 \times 20 + 0} = 1.18 < 1.5$$

LOCATION OF RESULTANT

$$R_2 = 0.59H = 0.59 \times 20 = 11.8 ft$$

FORCE ON BURIED LENGTH OF SHEETING

Assume $H_1 = 5 < \frac{2}{3}\frac{B}{\sqrt{2}}$

$$P_{H3} = 1.5H_1(\gamma_T H - \frac{1.4cH}{B} - \pi c)$$

$$P_{H3} = 1.5 \times 5(0.12 \times 20 - \frac{1.4 \times 0.4 \times 20}{12} - \pi 4)$$

$$P_{H3} = 1.6 kips$$

EXAMPLE OF ANALYSIS OF PRESSURE ON FLEXIBLE WALL IN CLAY

Ref: Department of the Navy, (1982) "Foundations and Earth Structures"
Naval Facilities Engineering Command, Alexandria, VA

RESISTIVITY SOUNDING DATA SHEET
Wenner Array, Method ASTM G-57
HIDDEN SPRINGS WATER IMPROVEMENTS
SAN ANTONIO, TEXAS

R-K Project Number: ASA11-032-00

Date: 3/30/2010

Meter: MiniRes (L&R Instruments, Inc.)

Weather Conditions: CLOUDY/COLD - 50°

Time: 10:42

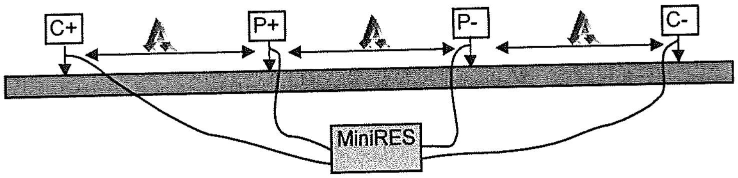
Units: feet

Observers: CCL/CRM

SOUNDING No.: S-1

Location Description: *EAST OF BORING B-1*

Electrode Spacing (Feet)			Factor	Meter Reading (Ohms)		Apparent Resistivity			
						(Ohm-Feet)		(Ohm-Centimeters)	
A	A/2	3A/2	(2 * PI * A)	N-S	E-W	N-S	E-W	N-S	E-W
2.5	*1.3*	*3.8*	15.71	56.700	57.100	890.6	896.9	27145.1	27336.6
5	*2.5*	*7.5*	31.42	20.100	21.000	631.5	659.7	19245.8	20107.5
10	*5.0*	*15.0*	62.83	6.173	6.539	387.9	410.9	11821.3	12522.2
15	*7.5*	*22.5*	94.25	3.266	3.206	307.8	302.2	9381.6	9209.2
20	*10.0*	*30.0*	125.66	2.078	2.046	261.1	257.1	7958.7	7836.2
30	*15.0*	*45.0*	188.50	1.010	1.007	190.4	189.8	5802.5	5785.2
50	*25.0*	*75.0*	314.16	0.525	0.471	164.9	148.0	5026.9	4509.8



Notes:

GPS LOCATIONS:

Center of Array
536303 E
3283018 N

UTM Zone 14
NAD 83
(horizontal position error ~±3 meters)

- * Hard Limestone
- * Exposed Bedrock
- * Resistor test = 19.005 ohm

Important Information about Your

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 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 your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you —* should apply the 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.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of 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,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- 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 study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the 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 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 construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual