GEOTECHNICAL REPORT

FOR

REFERENCE ONLY

SUBSURFACE EXPLORATION, LABORATORY TESTING PROGRAM AND GEOTECHNICAL RECOMMENDATIONS FOR THE PROPOSED CRESCENT PARK BOOSTER STATION 1 CRESCENT PARK SAN ANTONIO, TEXAS

RETL JOB NUMBER: G212234

PREPARED FOR:

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AUGUST 22, 2012

PREPARED BY:

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INTRODUCTION

This report presents the results of a soils exploration, laboratory testing program and geotechnical analysis for the proposed Crescent Park Booster Station to be constructed at 1 Crescent Park within The Dominion Subdivision in San Antonio, Texas. This study was conducted for LNV, Inc.

Authorization

The work for this project was performed in accordance with RETL proposal number P073112A dated July 31, 2012. The proposal included a proposed scope of work, fee and limitations. The proposal was signed by Derek Naiser, P.E., of LNV, Inc. on August 2, 2012 and returned to our office via email.

Purpose and Scope

The purpose of this exploration was to evaluate the soil and groundwater conditions at the site of the proposed booster station, and to provide geotechnical recommendations and foundation design parameters suitable for the proposed project.

The scope of the exploration and analysis included the subsurface exploration, field and laboratory testing, engineering analysis and evaluation of the subsurface soils, developing geotechnical recommendations and preparation of this report.

The scope of services did not include an environmental assessment. Any statements in this report, or on the boring logs, regarding odors, colors, unusual or suspicious items or conditions are strictly for the information of the client.

General

The exploration and analysis of the subsurface conditions reported herein are considered sufficient in detail and scope to provide geotechnical recommendations for the design and construction of the proposed booster station. The recommendations submitted for the proposed project are based on the available soil information and the preliminary design details provided by Jeffrey E. Reck, P.E. of LNV, Inc. If additional soil information is needed to complete the design of the structures, and this information can be obtained from the data obtained within the agreed upon scope of work, then RETL will provide this information in a supplemental report.

The Geotechnical Engineer states that the findings, recommendations, specifications or professional advice contained herein, have been presented after being prepared in a manner consistent with that level of care and skill ordinarily exercised by reputable members of the Geotechnical Engineer's profession practicing contemporaneously under similar conditions in the locality of the project.

RETL operates in general accordance with "*Standard Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction*", (ASTM D3740). No other representations are expressed or implied, and no warranty or guarantee is included or intended.

This report has been prepared for the exclusive use of LNV, Inc. for the specific purpose of the proposed Crescent Park Booster Station to be constructed at 1 Crescent Park within The Dominion Subdivision in San Antonio, Texas

FIELD EXPLORATION

Scope

The field exploration, to evaluate the engineering characteristics of the subsurface materials, included reconnaissance of the project site, drilling the test borings and recovering disturbed split barrel soil samples. Three borings were completed at the site, two were each drilled to a depth of 25-feet at the booster station location and one was drilled to a depth of 12-feet at the location of the water main and bore pit. The test borings were drilled using air-rotary drilling methods. LNV, Inc. determined the location and depth of the borings. Upon completion of the drilling operations the bore holes were backfilled with excavated soil. A Boring Location Plan is provided in the Appendix of this report.

Drilling and Sampling Procedures

The test borings were performed using a drilling rig equipped with a rotary head and air-rotary drilling methods were used to advance the boreholes to their desired termination depths. Disturbed soil samples were obtained employing split-barrel sampling procedures in general accordance with the procedures for "*Penetration Test and Split-Barrel Sampling of Soils*" (ASTM D1586). All of the soil samples were placed in plastic bags, marked according to boring number, depth and any other pertinent field data, stored in special containers and delivered to the laboratory for testing.

Field Tests and Measurements

Penetration Tests - During the sampling procedures, standard penetration tests (SPT) were performed to obtain the standard penetration value of the soil. The standard penetration value (N) is defined as the number of blows of a 140-pound hammer, falling 30-inches, required to advance the split-barrel sampler 1-foot into the soil. The sampler is lowered to the bottom of the previously cleaned drill hole and advanced by blows from the hammer. The numbers of blows are recorded for each of three successive 6-inch penetrations.

The "N" value is obtained by adding the second and third 6-inch increment number of blows. The results of standard penetration tests indicate the relative density of cohesionless soils and comparative consistency of cohesive soils, thereby providing a basis for estimating the relative strength and compressibility of the soil profile components.

Water Level Observations - Water level observations were obtained during the test boring operations and are noted on the boring logs provided in the Appendix. In relatively pervious soils, such as sandy soils, the indicated depths are usually reliable groundwater levels. In relatively impervious soils, a suitable estimate of the groundwater depth may not be possible, even after several days of observation. Drilling techniques, seasonal variations, temperature, land-use, proximity to a creek, river or body of water and recent rainfall conditions may influence the depth to groundwater. The amount of water in an open borehole largely depends on the permeability of the soils encountered at the boring locations.

Ground Surface Elevation – The ground surface elevations at the boring locations were not provided. Therefore, all depths referred to in this report are from the ground surface elevation at the boring locations during the time of our field investigation.

LABORATORY TESTING PROGRAM

In addition to the field investigation, a laboratory testing program was conducted to determine additional pertinent engineering characteristics of the subsurface materials.

The laboratory testing program included supplementary visual classification (ASTM D2487) and water content tests (ASTM D2216) on all samples. In addition, selected samples were subjected to Atterberg limits tests (ASTM D4318), percent material finer than the #200 sieve (ASTM D1140), and soil resistivity testing using the two-electrode box method (ASTM G187).

All phases of the laboratory testing program were conducted in general accordance with applicable ASTM Specifications. With the exception of the soil resistivity testing, the results of these tests are to be found on the accompanying borings log provided in the Appendix.

Soil Resistivity Testing

The testing conducted to identify the resistivity properties of the subsurface soil materials at the site of the proposed booster station included employment of the *Standard Test Method for Measurement of Soil Resistivity Using the Two-Electrode Soil Box Method* (ASTM G187) test on disturbed samples of the limestone rock in their as is moisture condition.

During the laboratory-based resistivity testing, the resistivity values were measured and recorded. As requested, the samples from depths of 0-feet to 15-feet from one test boring were subject to resistivity testing. RETL selected samples from boring B-1 to be subjected to the resistivity testing.

The soil resistivity values as determined by the two-electrode soil box method are included in the table below:

BORING B-1 SOIL RESISTIVITY	
Depth (ft.)	**Resistivity (Ω·ft)**
0-6	6,890
6-15	1,090

SUBSURFACE CONDITIONS

General

The types of subsurface materials encountered in the test borings have been visually classified and are described in detail on the boring logs. The results of the field penetration tests, water level observations and other laboratory tests are presented on the boring logs in numerical form. Representative samples of the soils were placed in polyethylene bags and are now stored in the laboratory for further analysis, if desired. Unless notified to the contrary, all samples will be disposed of three months after the issuance of this report.

The stratification of the soil, as shown on the boring logs, represents the soil conditions at the actual boring locations. Variations may occur between or beyond the boring locations. Lines of demarcation represent the approximate boundary between different soil types, but the transition may be gradual, or not clearly defined.

It should be noted that, whereas the test borings were drilled and sampled by experienced drillers, it is sometimes difficult to record changes in stratification within narrow limits. In the absence of foreign substances, it is also difficult to distinguish between discolored soils and clean soil fill.

Soil Conditions

The subsurface soils in Borings B-1 and B-2 at the site consist of hard to very hard slightly plastic weathered limestone and chalk materials that extend to the boring termination depths of 25-feet. The chalky soils are likely the residual materials leftover from carbonate weathering, and therefore bare similar properties to their parent carbonate materials in that they are inert, hard materials.

The subsurface materials in Boring B-3 consist of an upper stratum of low plasticity clayey/silty clayey gravel to approximately the 6.5-foot depth underlain by alternating layers of weathered limestone and lean clay which extend to the boring termination depth of 12-feet.

The generalized subsurface conditions encountered at the boring locations are summarized in the following table.

D	Description	LL	PI	C	Ø	γ_e	-#200	N
0-6	**LIMESTONE** and **SILTY CLAYEY GRAVEL**	37-17	5-16	3000	0	120	24-48	17-50/4"
6-10	**LIMESTONE** and Lean **CLAY**	24-26	11	0	38	125	34-75	50/5"-50/1"
10-25	**LIMESTONE** and **CHALK**	21	7	0	38	125	35	50/2"-50/0"

Where: D = Depth below existing grade, ft
LL = Liquid Limit (%)
PL = Plastic Limit (%)
PI = Plasticity Index
C = Average Soil Cohesion, psf (undrained)
Ø = Average Angle of Internal Friction, deg. (undrained)
γ_e = Effective Soil Unit Weight, pcf
-#200= Percent Material Finer than a #200 sieve
N = Standard Penetration value range

Detailed descriptions of the subsurface materials encountered at the boring locations are provided on the boring log included in the Appendix.

Groundwater Observations

No groundwater was observed during or immediately upon completion of the drilling operations. It should be noted that water levels in an open borehole may require several hours to several days to stabilize depending on the permeability of the soils and that groundwater levels at this site may be subject to seasonal conditions, recent rainfall, drought or temperature effects.

FOUNDATION & GEOTECHNICAL RECOMMENDATIONS

Project Description

Based on information provided to RETL, the project will consist of the construction of a potable water pump station to supply fire water flows to the Hidden Springs and Dominion Subdivisions, and will provide domestic (daily) flows to Hidden Springs.

The pump station will consist of three 5,000 gallon hydropneumatic tanks, four pumps (1-1000 gpm, 2-500 gpm, and 1-150 gpm), a standby emergency electric generator, electrical and control panels, a 40-feet tall antenna for radio communications, a pressure reducing valve in a concrete vault (below ground), and associated plant piping (above ground and below ground).

PVR Discussion

The laboratory test results indicate that the subsurface soils and rock in the active zone at this site are generally low in plasticity to non-expansive. **The calculated total potential vertical rise (PVR) at the booster station site is less than ½-inch.** The PVR was calculated using the Texas Department of Transportation Method TEX-124E and took into account the depth of active zone, estimated to extend to a depth of 15-feet, and the Atterberg limits test results of the soils encountered within the active zone.

The estimated PVR value provided is based on a lightly loaded foundation, floor or flatwork system founded near the surface applying a sustained surcharge load of approximately 1.0 pound per square inch on the subgrade soils. The value represents the vertical rise that can be experienced by dry subsoils if they are subjected to conditions that allow them to become saturated, such as poor drainage. Using dry soil conditions to calculate the PVR is generally considered the worst-case scenario. The actual movement of the subsoils is dependent upon their change in moisture content. Differential vertical movements can potentially be equal to the expected total movements. Worst case scenario can result in differential vertical movements at this site equal to the calculated PVR over a distance of 5-feet, or approximately the depth of the active zone if dry soil conditions exist and a localized water source such as ponding water occurs resulting in non-uniform moisture conditions.

Footing and Mat Foundations

Conventional spread and continuous strip footings are generally most economical when the existing soil conditions allow them to be founded at shallow depths. This type of foundation can be considered for lightly loaded structures and equipment at the booster station site. Mat foundations are used most advantageously when it is necessary to distribute comparatively heavy structural and working loads onto the supporting foundation materials. In view of the prevailing subsurface conditions, it appears that mat foundations will also be a suitable foundation system at the booster station site.

A net allowable unit bearing pressure of 5,000 psf may be used for footing and mat foundations founded on or in the natural weathered limestone materials and a net allowable unit bearing pressure of 2,500 psf may be used for footing and mat foundations founded on properly compacted structural fill soils. The foundations shall have a minimum depth of 12-inches below the final ground surface elevation.

The net allowable unit bearing pressures provided above include a safety factor of at least 3.0. The allowable unit soil pressures may be increased by 33-percent for maximum transient loads, such as wind loads. Footings shall have a minimum dimension of 2-feet.

Footings and mats designed using the net allowable unit soil pressures provided could expect total and differential settlements of less than 1-inch. In order to minimize the effects of any slight differential movement that may occur due to variations in the character of the supporting materials, it is recommended that all foundations be suitably reinforced to make them as rigid as possible.

It is recommended to model a mat using a modulus of subgrade reaction (K) of 140 pci for the natural in-situ subgrade soils and weathered rock. It should be noted that mat foundations supporting dynamically dominate machinery shall be founded on a structural fill pad with a minimum thickness of at least 2-feet and extending at least 2-feet beyond the perimeter of the mat foundation in all directions. Crushed limestone road base material is recommended for use as structural fill. **The Shear Modulus of crushed limestone utilized as structural fill at this site is 20,000 psi and the modulus of subgrade reaction (K) can be increased to 360 pci for a 2-foot thick pad of crushed limestone structural fill.**

Foundation excavations will likely require high powered rock excavation and jack hammering equipment. All loose material or improperly compacted fill in the bottom of the foundation excavations shall be removed to the level of competent bearing materials prior to concrete placement.

Excavations and Slopes

The soil parameters provided in the table below may be used for the design of braced excavations. The trench protection should be designed to provide the most conservative design.

D	Description	C	Ø	C'	Ø'	Ka	Kp
0-6	**LIMESTONE** and **SILTY CLAYEY GRAVEL**	3000	0	520	25	0.41	2.46
6-25	**LIMESTONE** and **CHALK**	0	38	0	38	0.32	4.2

Where: D = Depth below existing grade (ft)
C= Undrained Shear Strength (psf)
Ø = Undrained Angle of Internal Friction (degrees)
C'= Drained Shear Strength (psf)
Ø'= Drained Angle of Internal Friction (degrees)
K_a= Active Earth Pressure Coefficient
K_p= Passive Earth Pressure Coefficient

It should be noted that the values provided in the table above are based on the soil strengths and soil densities encountered in the field. Empirical formulas were used to correlate undrained shear strengths to drained shear strengths and the corresponding angle of internal friction for soils.

The active and passive earth pressure coefficients for the clay soils encountered were calculated using the drained angle of internal friction as recommended in "***FOUNDATION ANALYSIS AND DESIGN***", written by Mr. Joseph Bowles where he states, "Drained soil parameters for stiff clays and Ø-C soils in general may be appropriate for lateral pressures behind braced walls where the excavation is open for a considerable length of time".

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. All excavations should comply with applicable local, state and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. We are providing this information solely as a service to our client. Under no circumstances should the information provided herein be interpreted to mean that RETL is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

In no case should slope height, slope inclination or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. Specifically, the current OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926 should be followed. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor's "competent person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. For excavations, including a trench, extending to a depth of more than 20-feet, it will be necessary to have the side slopes designed by a professional engineer licensed in the State of Texas.

The contractor's "competent person" should establish a minimum lateral distance from the crest of the slope for all vehicles and spoil piles. Likewise, the contractor's "responsible person" should establish protective measures for exposed slope faces.

Excavations will require high powered rock sawing and/or rock jack hammering excavation equipment.

Retaining Wall and Below Grade Wall Design

Retaining and below grade walls must be designed to resist the loads imposed by the retained soil without exhibiting excessive deflection, rotational and/or sliding movements. In an attempt to reduce the load on the wall, granular backfill materials are typically used to fill the void between the natural soil materials and the wall. The load on the wall is transferred to the footing and resisted by the friction between the footing concrete and underlying soils, the resistance of the soil along the face of the footing and backfill materials placed above the footing.

Equivalent fluid density values for active, passive and at rest conditions were evaluated for various backfill materials. These values and their respective USCS soil classification symbols are presented in the table below:

Backfill Material	Equivalent Fluid Density (psf per foot of depth)		
	Active	**At Rest**	**Passive**
Conventional Select Fill (CL-ML, CL, ML & SC-SM, SC, SM) (PI= 4 to 18)	70	95	200
Sand (SP & SW) (PI < 4)	45	65	275
Free Draining Gravel (GW, GP or GM) (PI=0)	40	65	330

Retaining walls, which are allowed to move slightly, will develop an "active" earth pressure condition. If the wall is restrained from lateral movements such as when it is fixed at the top, the "at rest" earth pressure condition will be developed.

The design equivalent fluid density values assume the soils are in a moist condition and have been compacted. It is very important to note that these equivalent fluid densities do not include the effect of seepage or hydrostatic pressures, groundwater, and surcharge loads due to equipment, vehicular loads, paving or future storage near the walls. It is recommended that the backfill behind the walls be free draining gravels, with less than 5 percent passing the #200 sieve. Soils with USCS Classification OL, MH, CH and OH are unsuitable for use as backfill.

A coefficient of sliding friction of **0.50** between concrete and natural soils should be used to design the footing supporting the wall. In addition, the passive soil resistance on the face of the footing may be estimated using an equivalent fluid pressure of **200 psf**.

CONSTRUCTION CONSIDERATIONS

Site Preparation

Site preparation of the surface in any construction areas should initially consist of removing all vegetation, all loose or excessively organic materials and all soft or saturated soils from the surface. After the cut operations are performed, and prior to fill placement, the upper 6-inches of exposed subgrade soils should be compacted to a minimum density of 95-percent of the maximum dry density as determined by the standard Proctor (ASTM D 698) and the moisture content of the subgrade soils should be at, or above, the optimum moisture content. Any soft spots identified should be removed to firm soils and recompacted in-place. The exposed subgrade should not be allowed to dry out prior to placing properly compacted fill soils, select fill soils, structural fill soils or slabs. Limestone rock will not require compaction testing.

Select Fill

Select fill material used at this site shall have a maximum liquid limit of 40-percent and a plasticity index (PI) between 4 and 18. The select fill should be placed in no greater than 8-inch thick loose lifts and compacted to a minimum density of 95-percent of the maximum dry density as determined by the standard Proctor (ASTM D698) test. The moisture content shall be within –3 to +3-percent of the optimum moisture content. The Geotechnical Engineer shall approve the select fill soils utilized at this site.

Structural Fill Material

Crushed limestone base materials used as structural fill should meet the requirements set forth in the Texas Department of Transportation (TxDOT) 2004 Standard Specifications for Construction of Highways, Streets and Bridges; Item 247, Type A, Grade 2. The base material should be placed in maximum 8-inch thick loose lifts and compacted to a minimum density of 95-percent of the maximum dry density as determined by the modified Proctor test (ASTM D1557) and within –2 to +2-percent of the optimum moisture content.

Earthwork and Foundation Acceptance

Exposure to the environment may weaken the soils at the bearing level if the excavation remains open for a long period of time. Therefore, it is recommended that excavations be extended to final grade and foundations be constructed as soon as possible to minimize potential damage to the bearing soils. The bearing level should be free of loose soil, ponded water or debris and should be observed prior to concreting by the Geotechnical Engineer, or his designated representative.

Foundation concrete should not be placed on soils that have been disturbed by rainfall or seepage. If the bearing soils are softened by surface water intrusion, or by desiccation, the unsuitable soils must be removed from the excavation and shall then be replaced with properly compacted select fill, approved by the Geotechnical Engineer, prior to placement of concrete.

The Geotechnical Engineer, or his designated representative, should monitor subgrade preparation and fill placement operations. As a guideline, a minimum of one in-place density test per 2,000 square feet, or three in-place density tests per lift, whichever is greater, should be performed on the exposed subgrade soils and each subsequent lift of fill. Any areas not meeting the required compaction should be recompacted and retested until compliance is met.

Site Drainage

A majority of foundation related problems in the project area are attributable, at least in part, to poor drainage. Poor drainage, and the resulting ponded water, can increase the likelihood of highly plastic soils to change in volume with changes in moisture content and can even induce settlements in very low plasticity silts or coarser grained materials. We recommend that an effective site drainage plan be devised by others prior to commencement of construction to provide positive drainage away from the structures and off the site, both during, and after construction.

Dewatering Construction Considerations

Based on the groundwater observations made during the drilling operations, it appears that dewatering should not be required to construct the below grade structures. It should be noted that the depth to the groundwater is subject to change due to climatic conditions. The following discussion is general information that may be useful where dewatering operations are required.

For construction of shallow excavations, open drainage or interceptor ditches can be expedient and relatively inexpensive method for lowering the groundwater table a slight distance. The interceptor ditch has to penetrate deeper than the elevation of the work area. Water collecting in such ditches normally has to be pumped out of the ditch for disposal. Since gravity flow is relied upon to bring the water to the ditch, the continued inflow is dependent on the water level in the ditch being kept low. With this method, it is common to construct small pits in the ditch, termed sump pits, for locating the necessary pumps (sump pumps).

The drawing down of the water table can also be accomplished by constructing a series of sump pits, or, if greater depth is required, some type of drainage wells around the construction area and pumping the water from these pits or wells.

For dewatering to intermediate depths (to about 30-feet but more if sufficient area is available for installing the necessary equipment) well-point systems are normally used. To dewater an area, a series of well points is installed around the perimeter of the area. The groundwater level within the perimeter will be lowered when the well-point system is put in operation. The spacing of the well points varies according to the soil type and depth of dewatering. Spacing conventionally varies between 3 and 10-feet.

With the type of pumping equipment conventionally used for well points, the depth of dewatering that can usually be achieved by a single line of well points located around the perimeter of an excavation is about 18 to 20-feet. This is due to the limit on the practical lifting, or suction, capacity of the pumping equipment. Lowering the water table through a greater distance may require the use of a two (or more) stage (multistage) installation. Where a two-stage installation is required, the well points for the first stage of drawdown are located near the extreme perimeter limits of the area that can be excavated, and are put into operation. Well points for the second stage are subsequently located within the area that has been excavated, near to the bottom elevation that has been dewatered by the first stage. The second stage well points then lower the water table to the additional depth necessary to complete the excavation in dry conditions.

Subsurface water that flows in an upward direction into an excavation area that is being dewatered imparts a seepage force that tends to loosen the soil, reducing the soil strength. The change in strength should be considered in designing excavation bracing and foundations. Where excavations are to extend more than a few feet below groundwater level, open ditches or pits may not be practical, and more advanced methods may be required.

Other methods of dewatering are available and may be more cost effective than those mentioned above. Additional information concerning dewatering may be obtained from a contractor whose specialty is dewatering.

GENERAL COMMENTS

If significant changes are made in the character or location of the proposed booster station project, a consultation should be arranged to review any changes with respect to the prevailing soil conditions. At that time, it may be necessary to submit supplementary recommendations.

It is recommended that the services of RETL be engaged to test and evaluate the soils in the excavations to verify that the soils are consistent with those encountered in the borings. RETL cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the booster station if not engaged to also provide construction observation and testing for this project. If it is required for RETL to accept any liability, then RETL must agree with the plans and perform such observation during construction as we recommend.

All sheeting, shoring, and bracing of trenches, pits and excavations should be made the responsibility of the contractor and should comply with all current and applicable local, state and federal safety codes, regulations and practices, including the Occupational Safety and Health Administration.

APPENDIX

BORING LOCATION PLAN

NO SCALE

August 22, 2012
LNV, Inc. – Jeffrey E. Reck, P.E.
RETL Job No.: G212234

PROPOSED CRESCENT PARK BOOSTER STATION
1 Crescent Park
San Antonio, Texas

ROCK ENGINEERING AND TESTING LABORATORY, INC.
18847 REDLAND ROAD; SUITE 202
SAN ANTONIO, TEXAS 78259
(210) 495-8000

LOG OF BORING B-1

ROCK
Rock Engineering & Testing Laboratory
18847 Redland Rd., Suite 202
San Antonio, Texas 78259
Telephone: 210-495-8000
Fax: 210-495-8015

CLIENT: LNV, Inc.
PROJECT: Crescent Park Booster Station
LOCATION: 1 Crescent Park - San Antonio, Texas
NUMBER: G212234

DATE(S) DRILLED: 08/09/2012 - 08/09/2012

DRILLING METHOD(S):
Air Rotary

GROUNDWATER INFORMATION:
Groundwater was not encountered during drilling, nor measured in the boring upon completion of the drilling.

SURFACE ELEVATION: N/A

SOIL SYMBOL	DEPTH (FT)	SAMPLE NUMBER	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: TONS/SQ FT PERCENT RECOVERY/ ROCK QUALITY DESIGNATION	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ FT)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
		SPT S-1		N= 26-50/4"	3						40	**WEATHERED LIMESTONE**, light gray, dry, very hard.
		SPT S-2		N= 50/4"	5							Same as above.
	5	SPT S-3		N= 50/4"	8							Same as above, light tan, slightly moist.
		SPT S-4		N= 32-50/4"	8	24	13	11			34	**WEATHERED LIMESTONE**, light tan, slightly moist, very hard.
		SPT S-5		N= 50/2"	9							Same as above.
	10											
		SPT S-6		N= 50/2"	10	21	14	7			35	Same as above.
	15											
		SPT S-7		N= 50/3"	10							**WEATHERED LIMESTONE**, light tan, slightly moist, very hard.
	20											
		SPT S-8		N= 50/0"	5							**CHALK**, light tan, dry, very hard.
	25											Boring terminated at a depth of 25-feet.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - POCKET TORVANE SHEAR STRENGTH

REMARKS:
Boring location determined by LNV, Inc. Drilling operations performed by a drilling subcontractor to RETL.

LOG_OF_BORING G212234 CRESCENT PARK.GPJ ROCK_ETL.GDT 8/22/12

LOG OF BORING B-2

Rock Engineering & Testing Laboratory
18847 Redland Rd., Suite 202
San Antonio, Texas 78259
Telephone: 210-495-8000
Fax: 210-495-8015

CLIENT: LNV, Inc.
PROJECT: Crescent Park Booster Station
LOCATION: 1 Crescent Park - San Antonio, Texas
NUMBER: G212234

DATE(S) DRILLED: 08/09/2012 - 08/09/2012

DRILLING METHOD(S):
Air Rotary

GROUNDWATER INFORMATION:
Groundwater was not encountered during drilling, nor measured in the boring upon completion of the drilling.

SURFACE ELEVATION: N/A

SOIL SYMBOL	DEPTH (FT)	SAMPLE NUMBER	N: BLOWS/FT P: TONS/SQ FT T: TONS/SQ FT PERCENT RECOVERY/ ROCK QUALITY DESIGNATION	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ FT)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
		SPT S-1	N= 32-50/5"	4							**WEATHERED LIMESTONE**, light gray, dry, very hard.
		SPT S-2	N= 50/4"	3	17	12	5			24	Same as above.
	5	SPT S-3	N= 50/5"	6							Same as above, light gray and tan.
		SPT S-4	N= 50/5"	9	26	15	11			75	**LEAN CLAY**, with sand, light tan, slightly moist, very hard. (CL)
	10	SPT S-5	N= 50/3"	7							Same as above, dry.
	15	SPT S-6	N= 50/2"	11							**CHALK**, with limestone gravel, light tan, slightly moist, very hard.
	20	SPT S-7	N= 25/1"	7						37	Same as above.
	25	SPT S-8	N= 25/0"	5							**CHALK**, light tan, dry, very hard.
											Boring terminated at a depth of 25-feet.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - POCKET TORVANE SHEAR STRENGTH

REMARKS:
Boring location determined by LNV, Inc. Drilling operations performed by a drilling subcontractor to RETL.

LOG OF BORING B-3

Rock Engineering & Testing Laboratory
18847 Redland Rd., Suite 202
San Antonio, Texas 78259
Telephone: 210-495-8000
Fax: 210-495-8015

CLIENT: LNV, Inc.
PROJECT: Crescent Park Booster Station
LOCATION: 1 Crescent Park - San Antonio, Texas
NUMBER: G212234

DATE(S) DRILLED: 08/09/2012 - 08/09/2012

DRILLING METHOD(S):
Air Rotary

GROUNDWATER INFORMATION:
Groundwater was not encountered during drilling, nor measured in the boring upon completion of the drilling.

SURFACE ELEVATION: N/A

DEPTH (FT)	SAMPLE NUMBER	N: BLOWS/FT P: TONS/SQ FT T: TONS/SQ FT PERCENT RECOVERY/ ROCK QUALITY DESIGNATION	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ FT)	MINUS NO. 200 SIEVE (%)	DESCRIPTION OF STRATUM
0–1										**ASPHALT = 4"/ BASE = 7"**
1–3	SPT S-1	N= 66	12	37	21	16			48	**CLAYEY GRAVEL**, dark brown, slightly moist, hard. (GC)
3–4	SPT S-2	N= 17	13							Same as above, very stiff.
4–6.5	SPT S-3	N= 33	7	18	13	5			25	**SILTY CLAYEY GRAVEL**, tan, slightly moist, dense. (GC-GM)
6.5–8	SPT S-4	N= 50/1"	6							**WEATHERED LIMESTONE**, tan, dry, very hard.
8–10	SPT S-5	N= 50/5"	9							**LEAN CLAY**, with sand, light tan, slightly moist, very hard.
10–12	SPT S-6	N= 50/0"	5							**WEATHERED LIMESTONE**, tan, dry, very hard.
12										Boring terminated at a depth of 12-feet.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - POCKET TORVANE SHEAR STRENGTH

REMARKS:
Boring location determined by LNV, Inc. Drilling operations performed by a drilling subcontractor to RETL.

ROCK Engineering & Testing Laboratory, Inc.

4910 Neptune Street
Corpus Christi, Texas 78405
Office: (361) 883-4555 Fax: (361) 883-4711

KEY TO SOIL CLASSIFICATIONS AND SYMBOLS

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Letter	Symbol: Hatching	Symbol: Color	NAME
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		RED	Well – graded gravels or gravel – sand mixtures, little or no fines
		GP			Poorly-graded gravels or gravel – sand mixtures, little or no fines
		GM		YELLOW	Silty gravels, gravel – sand – silt mixtures
		GC			Clayey gravels, gravel - sand – clay mixtures
	SAND AND SANDY SOILS	SW		RED	Well - graded sands or gravelly sands, little or no fines
		SP			Poorly – graded sands or gravelly sands, little or no fines
		SM		YELLOW	Silty sands, sand – silt mixtures
		SC			Clayey sands, sand – clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS LL < 50	ML		GREEN	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL			Organic silts and organic silt-clays of low plasticity
	SILTS AND CLAYS LL > 50	MH		BLUE	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH			Inorganic clays of high plasticity, fat clays
		OH			Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		PT		ORANGE	Peat and other highly organic soils

TERMS CHARACTERIZING SOIL STRUCTURE

SLICKENSIDED – having inclined planes Of weakness that are slick and glossy in appearance

FISSURED – containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical

LAMINATED (VARVED) – composed of thin layers of varying color and texture, usually grading from sand or silt at the bottom to clay at the top.

CRUMBLY – cohesive soils which break into small blocks or crumbs on drying

CALCAREOUS – containing appreciable quantities of calcium carbonate, generally nodular.

WELL GRADED – having wide range in grain sizes and substantial amounts of all intermediate particle sizes.

POORLY GRADED - predominantly of one grain size (uniformly graded) or having a range of sizes with some intermediate size missing (gap or skip graded)

SYMBOLS FOR TEST DATA

M/C = 15 – Natural moisture content in percent.

γ = 95 – Dry unit weight in lbs/cu ft.

Qu = 1.23 – Unconfined compression strength in tons/ sq ft.

51 – 21 – 30 – Liquid limit, Plastic limit, and Plasticity index.

30% FINER – Percent finer than No. 200 mesh sieve

30 B/F – Blows per foot, standard penetration test.

▼ – Ground water table.

TERMS DESCRIBING CONSISTENCY OF SOIL (2)

COARSE GRAINED SOILS

DESCRIPTIVE TERM	NO. BLOWS / FT. STANDARD PEN. TEST
Very loose	0 – 4
Loose	4 – 10
Firm (medium)	10 – 30
Dense	30 – 50
Very Dense	over 50

FINE GRAINED SOILS

DESCRIPTIVE TERM	NO. BLOWS / FT. STANDARD PEN. TEST	UNCONFINED COMPRESSION TONS PER SQ. FT.
Very Soft	< 2	< 0.25
Soft	2 – 4	0.25 – 0.50
Plastic (med. Stiff)	4 – 8	0.50- 1.00
Stiff	8 – 15	1.0 – 2.00
Very Stiff	15 – 30	2.00 – 4.00
Hard	over 30	over 4.00

Field classification for "Consistency" is determined with a 0.25" diameter penetrometer.