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GEOTECHNICAL INVESTIGATION REPORT

Prepared for:

ASSURED REAL ESTATE
160 WEST HORIZON DRIVE, SUITE B
HENDERSON, NEVADA 89015

RESIDENTIAL DEVELOPMENT
SEC ATHENS AVENUE AND MILAN STREET
APN 179-04-503-001
HENDERSON, NEVADA

DEI No.: 19-0437

JUNE 30, 2019

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Assured Real Estate
160 West Horizon Drive, Suite B
Henderson, Nevada 89015

June 30, 2019

Attention: Mr. Joe Yakubik

Subject: Geotechnical Investigation Report
Residential Development
SEC Athens Avenue and Milan Street
APN 179-04-503-001
Henderson, Nevada

DEI No.: 19-0437

Sir:

DuPont Engineering, Inc. is pleased to present this geotechnical investigation report for the proposed residential development. This report discusses the investigation we performed and provides recommendations for items such as grading, foundations, and drainage. We have concluded that the structures may be supported on shallow foundation system consisting of continuous and spread footings bearing on on-site soils and/or structural fill. Detailed descriptions of our findings, conclusions, and recommendations are contained within the body of this report.

We appreciate the opportunity to work with you on this project and trust that we have provided you with the information you require at this time. If you have any questions, comments, or concerns please give us a call at your convenience.

Respectfully submitted,
DuPont Engineering, Inc.

David R. DuPont, P.E.
President

DRD
Dist: 3/Addressee

GEOTECHNICAL INVESTIGATION REPORT
RESIDENTIAL DEVELOPMENT
SEC ATHENS AVENUE AND MILAN STREET
HENDERSON, NEVADA

Table of Contents

I. INTRODUCTION

This report describes the results of DuPont Engineering, Inc.'s geotechnical investigation for the proposed residential development.

A. Purpose and Scope of Services

The purpose of this investigation was to:

1. Determine the subsurface soil conditions at the site.
2. Provide geotechnical recommendations for the residences pertaining to:
   - i. Grading
   - ii. Excavation Difficulties
   - iii. Foundations
   - iv. Slabs
   - v. Concrete Durability
   - vi. Site Drainage
   - vii. Landscaping

B. Project Description

The proposed development consists of a residential development that is anticipated to include between 3 and 6 residences. The site is approximately 4.89 acres located at the southeast corner of Athens Avenue and Milan Street in Henderson, Nevada. The APN for the site is 179-04-503-001. The structures will be one- or two-stories in height of wood-frame construction and have slab-on-grade concrete floors. Design loads have not been provided to us; therefore, we have assumed loads typical for this type of construction. The site location is shown on the Vicinity Map and a Site Plan has been provided which illustrates the lot layout.

II. FIELD INVESTIGATION

A. Site Description

At the time of our exploration the site contained a minor growth of desert vegetation. The site was undulating with grade changes on the order of 15 feet across the site. Drainage was by sheet flow and via the washes. The elevation of the site varied considerably but was generally a few feet to several feet above Milan Street, a partially paved road. No trash or debris was noted.

B. Drilling and Sampling Methods

A subsurface exploration of the site was conducted by drilling three 15-foot deep borings at the approximate locations shown on the Site Plan. The borings were made using a truck-mounted drill rig. The drill rig uses air-rotary drilling techniques that send soil particles to the surface by pressure created from compressed air allowing for instant soil identification. Samples of the soil were obtained and the consistency of the soil determined using a split-barrel sampler. The sampler has a 2.4-inch diameter inner opening and a 3-inch outer diameter. The sampler is driven into the ground by raising a 320-pound weight to a height of 30 inches, then dropping it on the sampler. The number of blows to drive the sampler a distance of 12 inches is recorded and is used to determine the soil's consistency. Our analysis of the soil profile was determined from the soils encountered, drilling and sampling characteristics, and our experience in the vicinity.

C. Subsurface Conditions

The soils encountered at the site consisted of sand that was generally loose in the upper 4 to 7 feet then medium dense. Groundwater was not encountered within the borings at the time of our exploration.

III. GEOLOGY AND SEISMICITY

A. Regional Geology

The Las Vegas Valley lies within the southern portion of the Walker Lane Belt of the Basin and Range Province. The Basin and Range Province extends from southern Idaho to northern Mexico and is an area of tilted northwest trending mountain ranges divided by sediment filled valleys.

The Las Vegas Valley is bounded by several of these mountain ranges. To the west are the Spring Mountains which have an elevation up to 12,000 feet at Charleston Peak. The northern portion of the Spring Mountains were deposited in deep ocean environments and are predominantly limestone and dolomite deposits of the Ordovician through Mississippian. Jurassic Sandstone deposits occur along the eastern flank of this range.

To the north of the valley lie the Las Vegas and Sheep Mountain Ranges. These ranges consist mostly of deep ocean limestone deposits from the Ordovician through Mississippian. The Las Vegas Range reaches an elevation of about 7000 feet, while the Sheep Range extends up to 10,000 feet.

Frenchman and Sunrise Mountains are located to the east of the valley and reach heights of about 4500 and 3500 feet, respectively. The exposed bedrock at the base of Frenchman Mountain is Precambrian schist which lies below Tapeats Sandstone, Bright Angel Shale, and Muav Limestone. The contact between the sandstone and the schist represents the great unconformity where an age gap between the rocks of 1.7 billion years exists. The upper portion of Frenchman Mountain has Jurassic sandstone and Tertiary volcanic outcrops exposed. The exposed sedimentary materials along the face of these two mountains match the exposures in the Grand Canyon. It is theorized that faulting during the last 13 million years has moved Frenchman Mountain from its original position in the Grand Canyon/Virgin Mountains to its present position.

To the south and southeast of the valley lie the River Mountains and the McCullough Mountain Range. The McCullough Range is the higher of the two and reaches an elevation of about 5000 feet. These mountains predominantly consist of Tertiary volcanics (mostly andesite and basalt) of the Miocene. These ranges are the remnants of the western and northern flanks of two large strato volcanoes that have mostly eroded away. The Black Hills located to the east of the McCullough Range are the remains of the central portion of one of the volcanoes.

The Las Vegas Valley itself is filled with sediment from the adjacent mountains and mountain ranges. Near the bases of the mountains where gradients are steeper the soils are typically granular. Finer grained sediments dominate the center and flatter portion of the valley. The current arid climate of Las Vegas produces an average rainfall of less than 3 inches per year, hence erosion and deposition rates are minimal at this time.

Overall drainage in the valley is toward the east and southeast via numerous large washes. The trunk wash where these washes converge is known as the Las Vegas Wash, normally an ephemeral stream that, due to development, carries water year round.

B. Regional Seismicity

During the Mesozoic the once passive west coast of the North American continent began to undergo compression as the East Pacific Rise pushed the Farallon Plate into and under the North American Plate. This process created regional uplift leading to the formation of mountains along the west coast including the Spring Mountains and the Las Vegas Range. The compressional forces led to thrust faulting which occurred locally in both of these mountain ranges. Some of the most notable thrust faults in these mountains include the Keystone, Lee Canyon, Kyle Canyon, Deer Creek, Wheeler Pass, and Mack Canyon Thrusts.

During the mid-Oligocene, about 30 million years ago, the Farallon Plate had completely subducted and the North American Plate began to pass over the East Pacific Rise. Extensional forces now began pulling apart the western portion of the North American Plate. Dip-slip faulting resulted and began breaking apart and tilting the mountain ranges forming the basins and ranges of the Basin and Range Province. The melting of the Farallon Plate as it subducted renewed volcanic activity along the west coast forming, among others, the Sierra Nevada Batholith, the River Mountains, Black Hills, and the McCullough Range.

Seismic activity in the Las Vegas Valley continued from this time until about 8 million years ago. Most faults in the vicinity of the valley during this time were dip-slip faults; however, a few major strike-slip faults were present. Most notably was the left lateral fault known as the Las Vegas Shear Zone located in the northern portion of the valley. Offset along this fault is more than 40 miles and is responsible for the bends in the southern portion of the Las Vegas and Sheep Ranges. It is likely responsible for much of the movement of Frenchman Mountain.

Between about 8 million and 4 million years ago, during the late Miocene, seismic and volcanic activity in the Las Vegas area was minimal. Seismic activity, however, resumed about 4 million years ago and continues to the present day. Many Quaternary faults have been mapped throughout the Valley. These faults step downward from the surrounding mountains toward the center of the valley and were originally thought to be caused by differential settlement of the subsurface soils. They were given the name compaction faults by Maxey and Jameson in 1948. Although this terminology exists to this day, more recent research indicates that these faults are actually tectonic and that differential settlement plays only a small contributing factor. None of these faults within the interior of the valley has been shown to be active within the Holocene. One of these faults, however, the Valley View Fault, did experience movement as little as 14,000 years ago. There are at least four known Holocene faults within the perimeter of the valley. These are the Frenchman Mountain Fault located at the base of Frenchman Mountain, the Black Hills Fault located at the eastern flank of the Black Hills, the California Wash Fault which lies along the western flank of the Muddy Mountains, and the Mead Slope Fault northeast of Boulder City. Each of these faults is capable of Richter magnitude of 6.5 to 7.0. Although no known movement has occurred on these faults within the past 2000 years, it is believed that they are long overdue for producing large earthquakes.

The nearest known Holocene fault is the Frenchman Mountain fault and is located approximately 6 miles north-northwest of the site. The nearest mapped Quaternary fault is located approximately 1/2 mile to the south-southeast of the site. The Quaternary fault locations were originally mapped by Bell and Price in the 1980's. Their study was published in 1993 and was subsequently used by the Clark County Building Department to create the "Clark County Soil Guidelines Map" in 1997. The Soil Guidelines Map is the reference used for determining the distance to the nearest Quaternary fault. There are no known geologic features that would require mitigation at this site. No faults are known to traverse the site, nor are any fissures observed.

Liquefaction should not be a concern at this site. For liquefaction to occur it is necessary for loose saturated granular soils to be present. This soil condition is not known to exist in the near surface soils.

The latitude and longitude of the site are 36.0701 and -114.9444, respectively. The Clark County Seismic Map indicates that the Site Classifies as Site Class C. Based Upon our knowledge of the soils in the vicinity of the site, we concur with the County map. The Site Class and Seismic Design Category (ASCE 7-16) are C based on the following data:

SITE CLASS	ASSUMED SEISMIC USE GROUP	S_S	S_1	S_{DS}	S_{D1}	SEISMIC DESIGN CATEGORY
C	II	0.486	0.163	0.421	0.163	C

C. Clark County Soil Guidelines Maps

The Clark County Building Department created two maps which illustrate potential soils hazard areas within the Las Vegas Valley. The intent of the maps is to provide guidance to engineers performing investigations within any of these areas.

The Clark County Soil Guidelines Map (August 2001) delineates four types of potential soil hazards and a non-hazard type generally consisting of mixed alluvial sand and gravel. The four hazard types are:

(i) Areas within 2000 feet of compaction or tectonic faults. These areas include 90 percent of all mapped fissure zones. Soil subsidence is the general hazard associated with this type of soil. These phenomena are discussed in the previous report section, Regional Seismicity.

(ii) Areas within 1000 feet of mapped washes. Aside from evaluating possible erosional damage to the property, the general hazards associated with these areas include recent sediment deposits and soils with a potential for solubility, clay swell, corrosion, gypsum salts, or hydro collapse.

(iii) Areas with the same potential hazards as described in Paragraph (ii) except for the recent sediment deposits and possible erosional damage.

(iv) Areas with ground slopes in excess of 15 percent and the potential for shallow bedrock.

Per the Clark County Soil Guidelines Map the subject site is located in the area defined as Area i.

The Clark County Expansive Soil Guidelines Map (September 2006) delineates potential expansive soil areas. According to the map, the map is intended to show general trends of near surface soils in the Las Vegas Valley. The soil conditions for a specific site could vary considerably from those described on the map. The areas are defined on the map as follows:

(i) Areas with greater than 12 percent expansion potential which is considered critical.

(ii) Areas with 8 to 12 percent expansion potential which is considered high.

(iii) Areas with 4 to 8 percent expansion potential which is considered moderate.

(iv) Areas with 0 to 4 percent expansion potential which is considered none to low.

Per the map the subject site is located within an area not described by the map.

When developing the recommendations for this report, we reviewed both maps and utilized the information they contain in conjunction with our field and laboratory testing data as well as our experience in the vicinity.

IV. LABORATORY TESTING

Representative samples were tested in the laboratory to obtain pertinent engineering properties for our analyses.

Laboratory testing on selected samples included moisture content and unit weight, Atterberg Limits (ASTM D4318), gradation (ASTM D1140), and collapse. The results of these tests are included on the boring logs and in the Appendix.

Chemical tests made by Silver State Analytical Laboratories to determine concrete durability requirements had the following results. The testing by Silver State included soluble sulfate content (SM 4500 S04 E), and sodium content (ASTM D2791). The results of Silver State's testing is contained in the Appendix.

V. CONCLUSIONS

Based upon our investigation, it is our opinion that, with regard to geotechnical considerations, construction of the proposed project is feasible at the site.

VI. RECOMMENDATIONS

A. Grading

For the purposes of this report the building pad areas are considered to encompass the footprints of the structures plus a distance of 5 feet laterally beyond the structures in each direction. Prior to grading operations being performed, all vegetation and debris should be removed from the pad areas.

Loose and loose to medium dense soils, and uncontrolled fill (if any is present) should be removed from the pad areas. Based upon the boring logs it is anticipated that removals on the order to 4 to 6 feet is likely necessary. However, based upon the grade changes at the site, it is also anticipated that much of the loose soil will be removed in establishing the site grades. Therefore, the amount of overexcavation required within the pad areas may be minimal.

Prior to any filling and after the necessary excavations are made, the soils within the pad areas are to be scarified to a depth of 12 inches, moisture conditioned, then recompacted. Next, any removed and stockpiled excavated soils should be moisture conditioned then recompacted in the excavations. Oversized material (particles larger than 6 inches in maximum diameter) should be removed from any stockpiled soils prior to fill placement.

The thickness of any lift of soil should not exceed 8 inches in loose thickness. All fill and scarified soil should be moisture conditioned and compacted in accordance with the specifications of Table 1. ASTM Test Method D 1557 should be used for determining the laboratory maximum dry density.

TABLE 1

Soil Type	**Moisture Content**	**Relative Compaction**
Coarse-grained	Over optimum	Minimum 95 percent

If imported soils are necessary to reach the site grade, they should comply with the following specifications contained in Table 2. If possible, the imported materials should be tested for compliance prior to hauling the material to the site.

TABLE 2

Sieve Screen	**Percent Passing (%)**
3 inch	100
¾ inch	50 - 100
No. 4	25 - 75
No. 200	5 - 25

Liquid Limit < 20	Soluble Sulfate < 0.10%
Plasticity Index < 6	Sodium Sulfate < 0.10%
Expansion Potential < 2% (oven-dried, 60 psf surcharge)	Total Solubility < 0.5 %

We anticipate that excavation and recompaction of the on-site soils will result in shrinkage losses of about 10 percent. Therefore, it should require about 1.1 cubic yards of excavated soil to generate 1.0 cubic yards of properly compacted native soil fill. Subsidence of the native soils which are scarified and recompacted will be on the order of 0.1 foot.

The scarification and filling operations should be monitored and density testing performed by a representative of the geotechnical engineer in order to ensure compliance with the recommendations of this report. The frequency of the testing shall be determined by the engineer performing the testing. However as a minimum one density test should be performed for each one foot of fill placed in each pad area. Based on the composition of the on-site soils, the site is considered to be Special Inspection Category G-B. Therefore, continuous inspection of grading operations is required.

B. Excavation Difficulties

The soils encountered at the site may be excavated with conventional earth-moving equipment. No special equipment such as a hoe-ram or large bulldozer is anticipated to be necessary.

C. Foundations

The proposed structures may be supported on shallow conventional foundations consisting of continuous and spread footings. All footings should bear upon on-site soils and/or compacted fill. Prior to concrete placement all footings should be cleaned of loose soils and any trash or debris. If the soils at the bottom of the footing excavations should become disturbed, they should be recompacted in accordance with the recommendations of the Grading section of this report.

Continuous footings should have a minimum width of 12 inches. Spread footings should be a minimum of 2 feet square. For single-story construction the exterior footings should be embedded a minimum of 12 inches below the lowest compacted grade. Interior footings should be embedded a minimum of 12 inches below the top of the slab. For two-story construction the minimum footing embedment should be increased to 15 inches. Allowable bearing pressures of 2000 psf and 2500 psf may be used for both continuous and spread footings for single and two-story construction, respectively. A one-third increase may be used for either wind or seismic loading.

Lateral loading on the structure may be resisted by both friction acting on the base of the foundations as well as passive earth pressure acting along the sides of the foundations. When determining the frictional component of the lateral resistance a coefficient of friction of 0.35 may be used. For the passive pressure component, the resisting pressure may be determined by considering the compacted soil as an equivalent fluid weighing 300 pcf. Passive resistance should not be considered for the upper 4 inches of soil. Furthermore, if any backfilling will be performed against foundations where the passive pressure resistance will be utilized, the backfilling should be monitored and tested by a representative of the geotechnical engineer. If an active pressure will be considered, an equivalent fluid weighing 38 pcf should be used.

Anticipated total settlement of the foundations and slab would be less than 1 inch. Differential settlement is anticipated to be half the magnitude of the total settlement.

If retaining walls will be present on the project, the recommendations given above regarding subgrade preparation, allowable bearing capacity, the coefficient of friction, and the passive pressure may be used in conjunction with an active pressure equivalent to that produced by a 38-pcf fluid. These design parameters assume that water will not collect behind the retaining walls. Suitable drains or weep holes should be provided to eliminate this possibility. If the tops of the walls are restrained, then an at-rest pressure equivalent to a 60-pcf fluid should be used instead of the active pressure.

Under seismic conditions, the active and at-rest equivalent fluid unit weight should be increased to 54 and 120 pcf, respectively. The resultant of the seismic component of earth pressure should be applied at 0.6 of the height where lateral soil pressure is acting. If seismic lateral pressure distribution is needed, it may be taken as an inverted triangle (see attached Earth Pressure Distribution Figure).

If the backfill behind the wall is not horizontal or if surcharge loads exist, these earth pressure design parameters should be reviewed.

D. Slabs

Concrete floor slabs should be supported on a layer of granular material that consists of a combination of Type II Aggregate Base, a visqueen membrane, and sand. The layer acts as a buffer between larger particles in the subgrade and the concrete, a moisture break, and assists the curing of the concrete. After constructing and compacting the pad to subgrade elevation, 4 inches of Type II should be placed. The Type II should be compacted above the optimum moisture content to a minimum of 95 percent of the maximum dry density (ASTM D 1557). The Type II should be overlain by a 10-mil visqueen membrane, then 2 inches of clean sand.

If steel reinforcement will be placed within the slab, it should be placed at mid-height of the slab unless otherwise specified by the structural engineer. Concrete should be placed and cured within the guidelines established by the American Concrete Institute Guide for Floor and Slab Construction, ACI 302.1, latest edition. Since all concrete shrinks upon curing, control joints should be grooved into the concrete at the time of placement, or saw-cut shortly thereafter, in order to control the locations of any shrinkage cracks that might develop.

E. Concrete Durability

Concrete durability is affected by soluble sulfates that are sometimes present in the soil. High concentrations of these sulfates can cause spalling and deterioration of the concrete. A sample of the on-site soils was tested by Silver State Analytical Laboratories, Inc. for soluble sulfate content. Based upon the results of the test, the concentration of soluble sulfates at the site may be considered negligible (S0) for the purpose of concrete design. All concrete that will be in contact with on-site or imported soils should be in accordance with the following table which is adapted from ACI-318.

SEVERITY	EXPOSURE CLASS	WATER-SOLUBLE SULFATE (SO_4) IN SOIL, percentage by weight	SULFATE (SO_4) IN WATER, ppm	CEMENT TYPE ASTM C150	$CaCl_2$ ADMIX	MAXIMUM WATER TO CEMENT RATIO	MINIMUM f'_c NORMAL WEIGHT AND LIGHTWEIGHT AGGREGATE CONCRETE, psi
Negligible	S0	0.00 - 0.10	0 - 150	NR	NR	NA	2500
Moderate	S1	0.10 - 0.20	150 - 1500	II	NR	0.50	4000
Severe	S2	0.20 - 2.00	1500 - 10000	V	NP	0.45	4500
Very severe	S3	Over 2.00	Over 10000	V+ Pozz. or Slag	NP	0.45	4500

NA = Not Applicable NR = No Restriction NP= Not Permitted

F. Site Drainage

Most soil related problems that can lead to structural distress are caused by moisture variations within the soils below the structures. Typically, it is an increase in the moisture content that creates problems. Therefore, the most important thing that one can do to prevent problems is to inhibit additional moisture from entering the soils below the structures. If this occurs, the soils below the building pads may experience volume changes resulting in structural distress due to excessive soil movement.

Positive site drainage should always be established away from the foundations and exterior of any structure. A minimum downward gradient of 5 percent should be maintained away from the structure for a distance of at least 10 feet, if possible.

Planter areas between the flatwork and a structure should not trap water. Care should be taken to not allow roof drainage to spill onto the ground adjacent to the foundations. If necessary, rain gutters should be utilized.

The utility trenches within a structure should be backfilled and compacted above the optimum moisture content to a minimum of 90 percent of the maximum dry density (ASTM D 1557) with the on-site soils or approved imported soils. The compaction of the utility lines should include where the utilities exit or approach the structure for a minimum distance of 5 feet beyond the limit of the structure. Care should be taken to prevent damage to the utility lines during the compaction process.

G. Landscaping

As mentioned in the previous section, increases in the moisture content of the subsurface soils below the structure are typically the root cause of structural distress. Therefore, landscaping restrictions are warranted.

All trees should be placed at least 10 feet from the exterior of each structure and its foundations. No other landscaping or irrigation lines should be placed within 5 feet of a structure.

Some landscaping rocks imported to the area contain high concentrations of soluble sulfates. These rocks can sometimes break down due to chemical weathering and release the sulfates into the soil where either none or only low concentrations had been present before. In order to reduce risk to the concrete, we recommend that landscaping rock be placed at least 5 feet from any structure or any concrete flatwork. If it is desired to place the rock closer to any concrete, the rock should first be tested for soluble sulfate concentrations.

VII. LIMITATIONS AND CLOSURE

The opinions and recommendations contained in this report were based upon the soil conditions observed at the site and our engineering experience in the area. Although the exploration borings provided us with a subsurface profile, it is possible that the soil conditions across the site vary from those described in the boring logs. Should the subsurface conditions be found to vary from those described, our office should be consulted to evaluate the present recommendations and make any necessary modifications.

All recommendations in this report are valid predicated upon verification by a representative of the geotechnical engineer, at the time of construction, that the recommendations have been complied with. Therefore, it is the responsibility of the addressee and/or developer to ensure that all parties involved in the project, that these recommendations may affect, be provided or made aware of this report, so that they may properly satisfy the recommendations contained within.

The developer or owner must understand that because the structure rests upon the ground, there are inherent risks to property due to earth movement. Although we provide recommendations to minimize risk, there are no guarantees that some structural distress could not occur in the future. Therefore, no warranties, either express or implied, are intended or made. We recommend that this report become a mandatory part of the escrow documents for the site so that the future owners of the property will be aware of the report's contents and may take the appropriate steps to follow or maintain its recommendations.

Our professional services have been performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. If you have any questions, concerns, or comments regarding the contents of this report, please give us a call.

Respectfully submitted,
DuPont Engineering, Inc.

David R. DuPont, P.E.
President

DRD