DUPONT
ENGINEERING, INC.

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CITY OF LAS VEGAS

GEOTECHNICAL INVESTIGATION REPORT

PREPARED FOR:

WORLD WELLNESS GROUP LLC
10220 WEST CHARLESTON BOULEVARD
SUITE 3
LAS VEGAS, NEVADA 89135

COMMERCIAL BUILDING ADDITION
3100 SOUTH VALLEY VIEW BOULEVARD
LAS VEGAS, NEVADA

DEI NO.: 07-0341

MAY 17, 2007

4420 Arville St., Suite 1 Las Vegas, NV 89103 **Tel** 702.364.5099 **Fax** 702.364.1129

DUPONT
ENGINEERING, INC.

World Wellness Group LLC
10220 West Charleston Boulevard
Suite 3
Las Vegas, Nevada 89135

May 17, 2007

Attention: Mr. Ryan Grauberger

Subject: Geotechnical Investigation Report
Commercial Building Addition
3100 South Valley View Boulevard
Las Vegas, Nevada

DEI No.: 07-0341

Gentlemen:

DuPont Engineering, Inc. is pleased to present this geotechnical investigation report for the proposed commercial building addition. This report discusses the investigation we performed and provides recommendations for items such as grading, foundations, and drainage. We have concluded that the structure may be supported on a shallow foundation system consisting of continuous and spread footings bearing on recompacted on-site soils 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.

PROFESSIONAL ENGINEER STATE OF NEVADA
DAVID R. DUPONT
Exp: 12-31-08
CIVIL
No. 14261

5/17/07

Todd C. Hayne, P.E.
Principal Engineer

David R. DuPont, P.E.
President

TCH/DRD/pjb
Dist: 3/Addressee

GEOTECHNICAL INVESTIGATION REPORT
Commercial Building Addition
3100 South Valley View Boulevard
Las Vegas, Nevada

Table of Contents

I. INTRODUCTION

This report describes the results of DuPont Engineering, Inc.'s geotechnical investigation for the proposed commercial building addition.

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 addition 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 an addition to the south side of the existing structure located at 3100 South Valley View Boulevard in Las Vegas, Nevada. The addition will be three stories in height. We anticipate metal-frame construction and a slab-on-grade lower floor. 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 an existing commercial structure and was paved. Drainage by sheet flow was to the east at a gradient of about 1½ percent. No trash or debris was noted.

B. Drilling and Sampling Methods

A subsurface exploration of the site was conducted by drilling two 20-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 materials encountered at the site consisted of pavement and base course over firm clayey silt. The clayey silt was underlain at 7 and 6 feet in Borings B-1 and B-2, respectively, by medium dense silty sand. Firm clayey silt was present at depths of 13 and 12 feet and was underlain at 15 and 14 feet by hard caliche. In Boring B-2 the caliche was underlain at 17 feet by medium dense clayey sand. Groundwater was encountered at a depth of 18 feet in Boring B-2 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 10½ miles northeast of the site. The nearest mapped Quaternary fault is located approximately ½ mile to the east 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.

The latitude and longitude of the site are 36.1346 and -115.1895, respectively. The Seismic Design Category (International Building Code, 2006) is D based on the following data:

SITE CLASS	ASSUMED SEISMIC USE GROUP	S_S	S_1	S_{DS}	S_{D1}	SEISMIC DESIGN CATEGORY
D	I	0.63	0.19	0.54	0.26	D

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 hydrocollapse.

(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.

The subject site is located in the non-hazard area.

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 described as having expansion potentials considered none to low.

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. Dry density and moisture content tests were made on selected samples to characterize the soil zones encountered. The dry density and moisture content test data are provided on the boring logs.

A collapse test was made to determine the collapse of the on-site soils under different loading conditions. The test was performed by loading an undisturbed sample in a one-dimensional consolidation device with a surcharge pressure, inundating the sample, and measuring the resulting settlement. These test results were as follows:

SAMPLE LOCATION	MOISTURE CONTENT, %		DRY DENSITY, pcf		COLLAPSE AT DESIGNATED PRESSURE (psf), %		
	PRE-TEST	POST-TEST	PRE-TEST	POST-TEST	60	560	1560
B-1 @ 5'	15.5	28.1	96.2	110.4	3.0	4.8	6.9

A total solubility test was performed using the local testing practice. The test results are shown in the following table:

SAMPLE LOCATION	TOTAL SOLUBILITY, %
B-1 @ 5'	0.5

Chemical tests made by Silver State Analytical Laboratory to determine concrete durability requirements had the following results:

SAMPLE LOCATION	SODIUM, %	WATER SOLUBLE SULFATE, %	TOTAL AVAILABLE SODIUM SULFATE, %
B-1 @ 1' - 2'	0.00	0.21	0.00

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. The adverse soil condition noted at the site is that some of the in-place soils are hydro-collapsible in their present state. Although these soils can adequately support a structure, they will consolidate upon the addition of moisture.

In order to minimize the risk of distress due to settlement from the consolidation of the hydro-collapsible soils, an overexcavation is necessary. Since the thickness of the hydro-collapsible soils can often reach great depths, overexcavating all of these soils is not always practical. The cost of doing so could prevent many owners from being able to build their structures. Therefore, what is generally recommended is to perform an overexcavation to remove these soils which are within the influence of the loads produced by the foundations, and to take precautionary measures to avoid moisture infiltration of the hydro-collapsible soils that remain in place. Additionally, a reinforcement schedule is provided for the foundations in order to resist some deflections that may occur.

This technique involving partial overexcavation of the hydro-collapsible soils is the basis by which the recommendations in the following sections are made. It must be understood that because some hydro-collapsible soils remain in-place there is always risk of distress in the future should these soils become excessively wet. A more detailed explanation regarding the problems these soils can create, the engineering methods of dealing with these soils, and the preventative measures that can be used to minimize risk is contained in the Appendix and is considered part of this report. If the risk of distress is unacceptable, then we should be contacted for other recommendations. It must be understood, however, that recommendations that lower the risk generally involve higher construction costs.

VI. RECOMMENDATIONS

A. Grading

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

Soils within the pad area should be overexcavated to a depth of 5 feet below the existing grade or 5 feet below the bottoms of foundations, whichever is lower. When excavating adjacent to existing foundations, no more than 8 feet of footing line should be exposed at any one time. A minimum lateral cover of 3 feet should be maintained where the footing is not exposed.

The removed soils may be stockpiled for later use as fill. Prior to any filling and after the necessary excavations are made, the soils within the pad area are to be scarified to a depth of 9 inches, moisture conditioned, then recompacted. Next, the stockpiled excavated soils should be moisture conditioned then recompacted in the excavation. Any pavement fragments and 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
Fine-grained	Min. 2 percent over optimum	Minimum 90 percent
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 20 percent. Therefore, it should require about 1.3 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. Per the building code, full-time inspection during grading operations is required for hydro-collapsible soils.

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 structure may be supported on shallow conventional foundations consisting of continuous and spread footings. Footings for new construction which will abut existing footings should bear at the same elevation as the existing footings. All footings should bear upon 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 for the soil type exposed.

Continuous footings should have a minimum width of 16 inches. Spread footings should be a minimum of 2½ feet square. The exterior footings should be embedded a minimum of 18 inches below the lowest compacted grade. Interior footings should be embedded a minimum of 18 inches below the top of the slab. An allowable bearing pressure of 2250 psf may be used for both continuous and spread footings. A one-third increase may be used for either wind or seismic loading. As a minimum, the continuous footings should be reinforced with two No. 5 bars at the top of the footings and two No. 5 bars at the bottom of the footings. If desired, three No. 4 bars may be substituted for two No. 5 bars.

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 0.30 may be used. For the passive pressure component, the resisting pressure may be determined by considering the compacted soil as an equivalent fluid weighing 275 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 40 pcf should be used.

The soil underlying the structural fill and scarified material is hydro-collapsible. In its present state, this soil will adequately support the structural fill and the structure. 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, however, the underlying soil becomes excessively wet or saturated, it will undergo collapse. Settlement and differential settlement could increase dramatically resulting in distress to the structure. The amount of settlement would depend upon the degree of saturation and the extent to which water migrated. Differential settlement would be equal to total settlement which could be several inches. Precautions, therefore, should be taken to minimize the potential for water to migrate into the underlying soils. Adequate reinforcement should be used to resist against slab and foundation deflections due to settlement. The above specified reinforcement schedule is generally adequate to tolerate some deflection, however, it is not a guarantee against cracking should the subsurface soils become excessively wet and substantial settlement occur.

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 40-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 56 and 126 pcf, respectively. The resultant of the seismic component of earth pressure (i.e., the difference between seismic and static conditions) 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 trapezoid (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, 6 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.

As a minimum, the slab should be reinforced with No. 3 bars at 18 inches on centers, each way. The steel reinforcement 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 severe 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 the International Building Code.

SULFATE EXPOSURE	WATER-SOLUBLE SULFATE (SO_4) IN SOIL, percentage by weight	SULFATE (SO_4) IN WATER, ppm	CEMENT TYPE	MAXIMUM WATER TO CEMENT RATIO	MINIMUM f'_c NORMAL WEIGHT AND LIGHTWEIGHT AGGREGATE CONCRETE, psi
Negligible	0.00 - 0.10	0 - 150	---	---	---
Moderate	0.10 - 0.20	150 - 1500	II, IP (MS), IS (MS)	0.50	4000
Severe	0.20 - 2.00	1500 - 10000	V	0.45	4500
Very severe	Over 2.00	Over 10000	V plus pozzolan	0.45	4500

F. Site Drainage

Most soil related problems that can lead to structural distress are caused by moisture variations within the soils below the structure. 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 structure. If this occurs, the soils below the building pad 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. Desert landscaping is recommended, in order to minimize the amount of water entering the ground. Only the amount of water necessary to maintain the life of the vegetation should be used. Excessive watering may lead to structural problems.

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. The Appendix section of this report contains an explanation of the problems associated with the soils at the site, and also describes the methods by which the property owners can reduce their risk of creating their own structural distress. 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 this contents of this report, please give us a call.

Respectfully submitted
DuPont Engineering, Inc.

PROFESSIONAL ENGINEER-STATE OF NEVADA
DAVID R.
DuPONT
Exp: 12/31/08
CIVIL
No. 14261

5/17/07

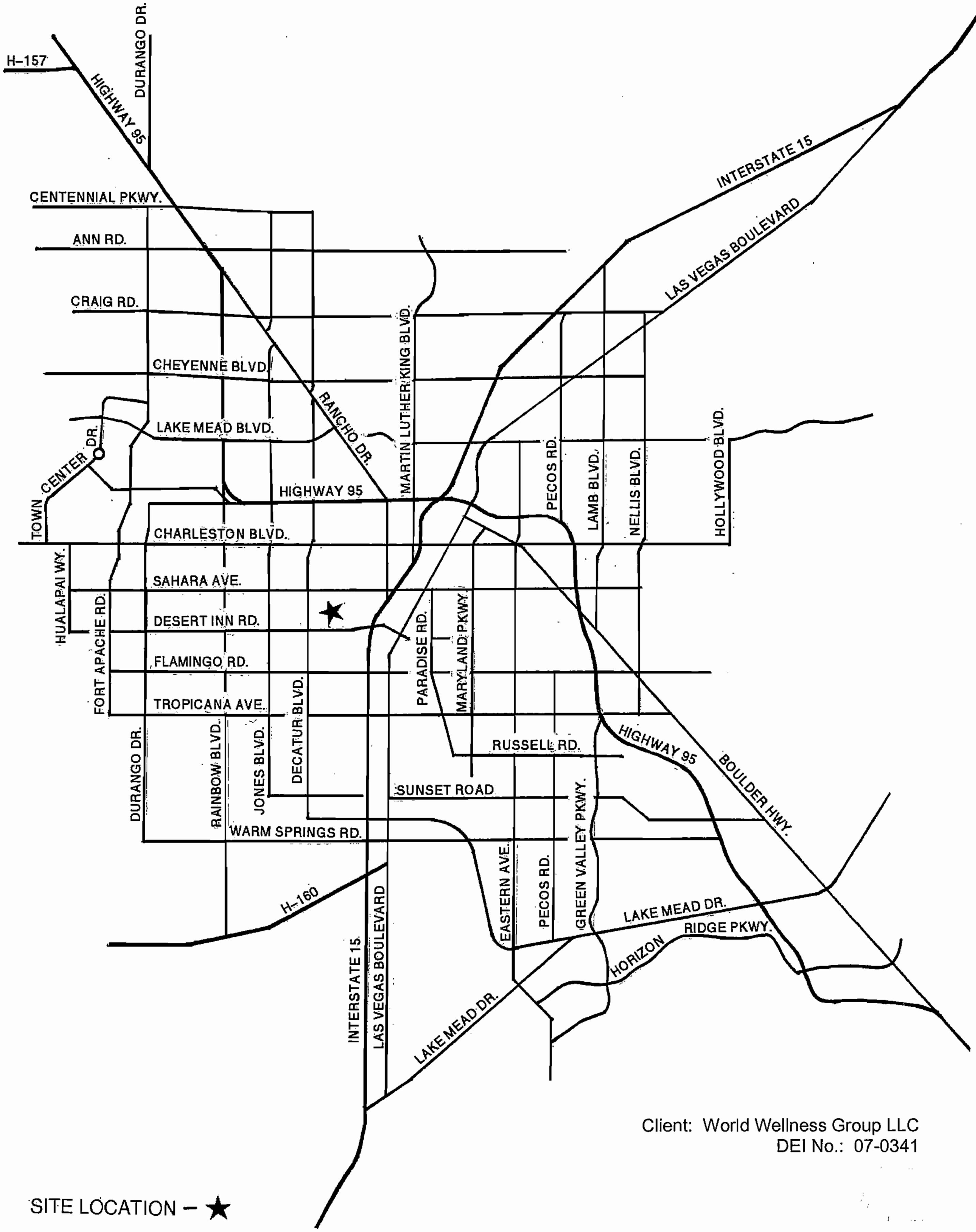
Todd C. Hayne, P.E.
Principal Engineer

David R. DuPont, P.E.
President

TCH/DRD/pjb

APPENDIX

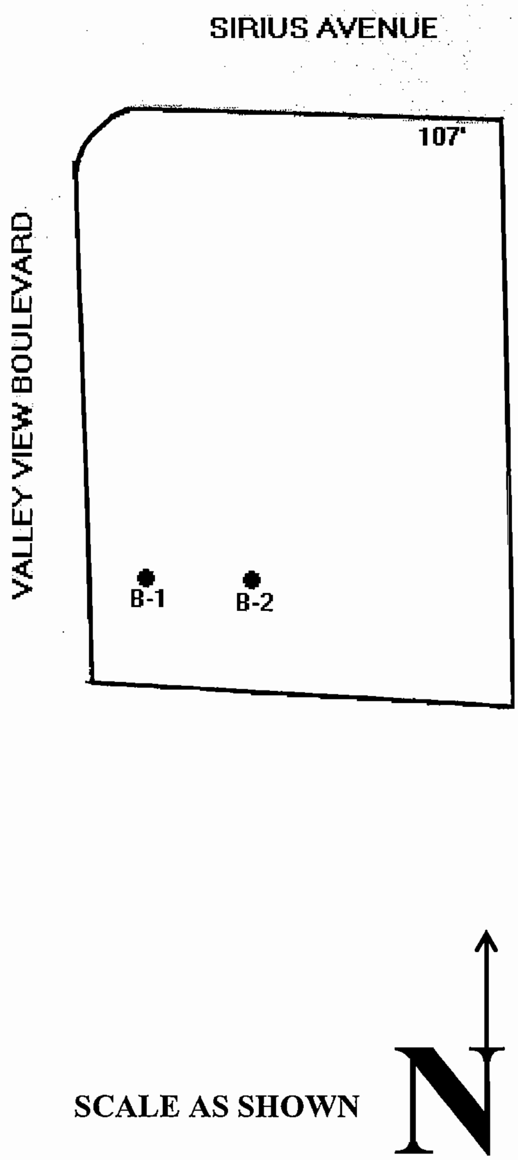
VICINITY MAP



Client: World Wellness Group LLC
DEI No.: 07-0341

SITE LOCATION – ★

SITE PLAN
COMMERCIAL BUILDING ADDITION
3100 SOUTH VALLEY VIEW BOULEVARD
LAS VEGAS, NEVADA



CLIENT: WORLD WELLNESS GROUP LLC
DEI NO.: 07-0341

Client: World Wellness Group LLC
Project: Commercial Building Addition
Boring #: B-1

Location: 3100 South Valley View Boulevard
Las Vegas, Nevada

DEI No.: 07-0341 **Date Drilled:** 4/19/2007
Elevation: -- **Datum:** Ground Surface
Boring Type: 6" Rotary Air **Type Rig:** Mayhew

Groundwater Conditions: None ATE

Depth, feet	Sample type	Resistance, blows/6 inches	Cohesion, ksf	Moisture Content, %	Dry Density, pcf	Unified Soil Classification	SOIL DESCRIPTION
							Asphalt concrete and base course
						ML-CL	Light tan Clayey SILT, little fine sand, trace gypsum, slightly porous, firm, slightly moist
2--	B			6.8			
4--							
	R	4/16		8.8	100		
6--							
						SM	Red-brown Silty SAND, trace clay, trace fine gravel, medium dense, moist, some water-staining
8--							
	R	5/9		11.7	109		
10--							
12--							
						ML-CL	Tan Clayey SILT, little fine to coarse sand, little caliche gravel, firm, very moist
14--							
	R	25 for 1" REF NR					CALICHE, hard
16--							
18--							

BOE @ 20'

Client: World Wellness Group LLC
Project: Commercial Building Addition
Boring #: B-2

Location: 3100 South Valley View Boulevard
Las Vegas, Nevada

DEI No.: 07-0341 **Date Drilled:** 4/19/2007
Elevation: -- **Datum:** Ground Surface
Boring Type: 6" Rotary Air **Type Rig:** Mayhew

Groundwater Conditions: 18' ATE

Depth, feet	Sample type	Resistance, blows/6 inches	Cohesion, ksf	Moisture Content, %	Dry Density, pcf	Unified Soil Classification	SOIL DESCRIPTION
							Asphalt concrete and base course
						ML-CL	Light tan Clayey SILT, little fine sand, trace gypsum, slightly porous, firm, slightly moist
2--							
4--							
	R	7/12		12.5			
6--						SM	Red-brown Silty SAND, trace clay, trace fine gravel, medium dense, moist, some water-staining
8--							
	R	7/11		11.5	114		
10--							
12--						ML-CL	Tan Clayey SILT, little fine to coarse sand, little caliche gravel, firm, very moist
14--							CALICHE, hard
16--							
						SC	Tan Clayey SAND, some fine to coarse gravel, medium dense, very moist to saturated
18--							∇
	R	5/6		13.8			

BOE @ 20'

SilverState
Analytical Laboratories

LABORATORY REPORT

DATE: April 23, 2007

REPORT NUMBER: 07-1166

CLIENT: DuPont Engineering, Inc.
4420 South Arville, Suite 1
Las Vegas, NV 89103

PAGE: 1 of 1

CLIENT PROJECT:

Sampled By: Client
Date Sampled: --
Time Sampled: --

Date Received: 04/20/07
Time Received: 1706

Sample ID: World Wellness B-1 @ 1-2'

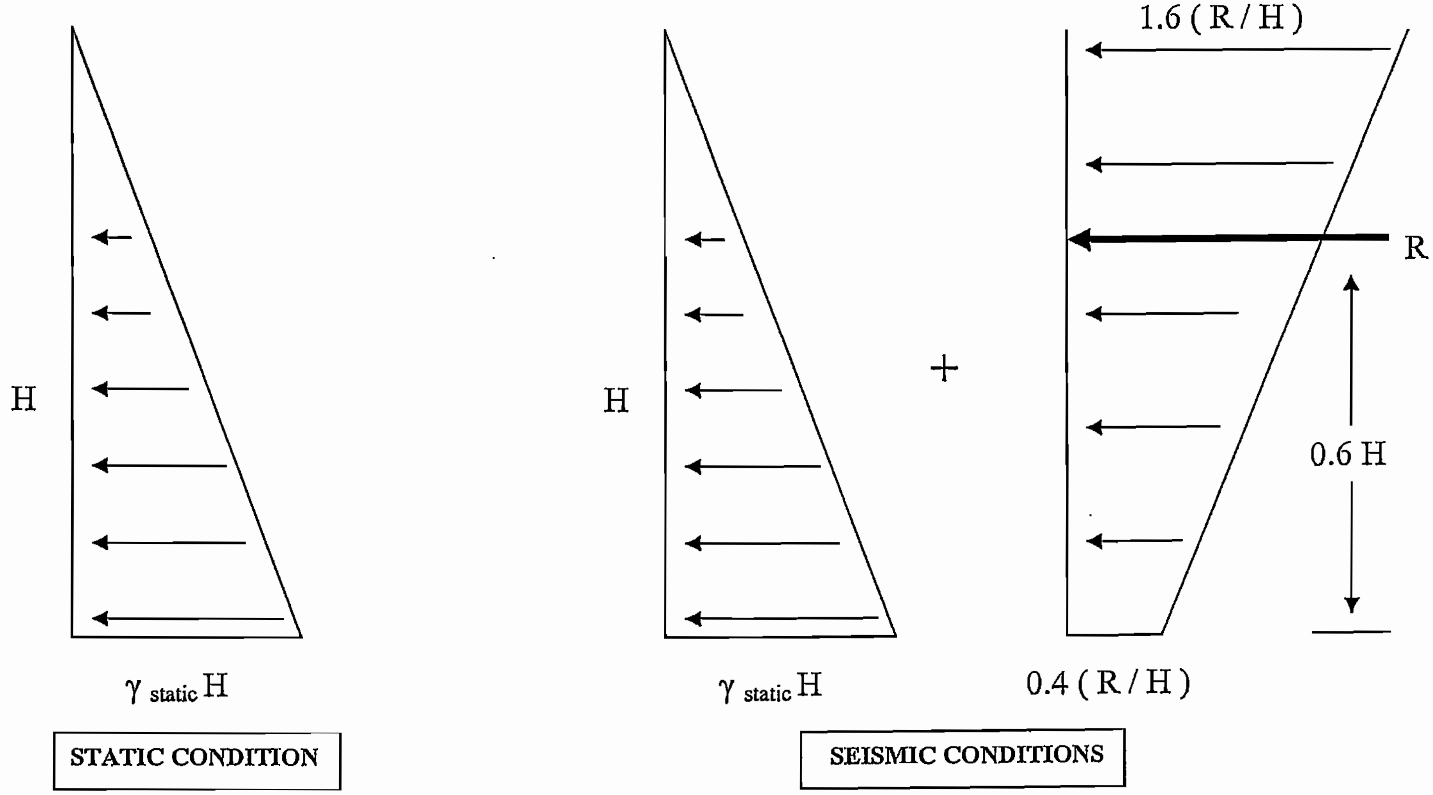
Test	Result	Unit	Method
Sodium	0.00	%	ASTM D2791
Sulfate	0.21	%	SM 4500 E
Sodium Sulfate	0.00	%	Calculation

NOTES: The results for each constituent denote the percentage (%) for that particular element which is soluble in a 1:5 (soil to water) extraction ratio and corrected for dilution. To calculate from a % to a concentration, multiply the % by 10,000 to obtain ppm. This conversion is only a rough number due to atomic weights.

REVIEWED BY:

Ronald W. Winter
Laboratory Director

Earth Pressure Distribution



R = Resultant of seismic component of earth pressure per unit length of the wall = $\frac{1}{2} (\gamma_{seismic} - \gamma_{static}) H^2$.
H = Height of the wall
γ = Equivalent fluid density, either for active or at rest conditions, as applicable.
Note: Surcharge pressure is not included here and should be added, if applicable.

DUPONT ENGINEERING, INC.

KEY TO SYMBOLS AND TERMS USED ON BORING AND TEST PIT LOGS

METHOD OF SOIL CLASSIFICATION (ASTM D 2487)

COARSE-GRAINED SOILS
LESS THAN 50% FINES

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES	
GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, MORE THAN 12% FINES	
GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, MORE THAN 12% FINES	
SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES	SANDS One half or more of coarse fraction is smaller than No. 4 sieve size
SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES	
SM	SILTY SANDS, SAND-SILT MIXTURES, MORE THAN 12% FINES	
SC	CLAYEY SANDS, SAND-CLAY MIXTURES, MORE THAN 12% FINES	

FINE-GRAINED SOILS
MORE THAN 50% FINES

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS	SILTS AND CLAYS Liquid limit less than 50
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
OL	ORGANIC SILTS OR ORGANIC SILTY-CLAYS OF LOW PLASTICITY	
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS	SILTS AND CLAYS Liquid limit 50 or more
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY	
PT	PEAT, MUCK, AND OTHER HIGHLY ORGANIC SOILS	HIGHLY ORGANIC SOILS

Notes: Only sizes smaller than 3 inches are used to classify soils.
Dual classification used for borderline soil groups.

LOG ABBREVIATIONS

Miscellaneous:
- ATE - at time of exploration
- AE - after exploration
- BOE - bottom of exploration
- ∇ 0 - Groundwater level at time of exploration
- ∇ 72 - Groundwater level 72 hours after exploration

Sample Type:
- R - ring sample
- S - Standard Penetration Test
- D - disturbed sample
- C - core sample
- U - undisturbed sample
- B - bulk sample
- NR - no recovery
- REF - sampler refusal

Note: Ring sampler is a split-barrel sampler with 1" rings. Inside diameter is 2½" and outside diameter is about 3.1". Blow counts shown represent the number of blows for 6" penetration for a 320-pound hammer falling freely through a 30" vertical distance. Energy ratio correlations indicate a close correspondence with Standard Penetration Test values with a 140-pound hammer falling 30" with a standard split spoon sampler.

Understanding Soil Related Problems, the Engineering Solutions to These Problems, and How to Try to Prevent Them (subsidence)

Introduction

This section of the report is written in order to provide owners, builders, contractors, or others who may be involved in the project an understanding of some of the soil-related issues that have led to the distress of many homes within Southern Nevada. If any statements are not understood, we recommend that you call our office so that we can provide clarification. Distress-free structures are not simply the responsibility of the developer. They result from a team effort among the engineers, contractors, and occupants. We believe that through a better understanding of the causes and effects of soil related problems, all parties involved can do their part in minimizing risk.

Most of the distresses that we have witnessed in Southern Nevada have occurred due to either regional subsidence or localized subsidence. Most localized subsidence problems can be controlled and/or prevented. This is not true for regional subsidence, at least not on a small lot-size scale. Each of these two types of subsidence will be addressed separately below.

After discussing the causes of subsidence, we provide a section on the typical engineering solutions to building in areas that are experiencing, or may experience, subsidence. A final section regarding preventative measures is provided to inform owners of how to maintain their property in order to minimize their risk of distress.

Regional Subsidence

Regional subsidence refers to a lowering of the ground elevation on a large scale. In this case a large scale means valley wide, or at least most of the valley. This occurs due to the removal of groundwater from the subsurface aquifers. An aquifer is a layer of soil below the ground which holds great quantities of water and through which the water can permeate easily. Since water flows through aquifers, it is the aquifers that are tapped for water wells. As the water is removed the soil particles within the aquifers begin to consolidate and the thickness of the aquifers decrease. The elevation at the ground surface lowers in relation to the decreases in the thicknesses of the aquifers. Since the water in the aquifers is being depleted in many places across the valley, the majority of the valley is lowering in elevation. This is regional subsidence.

As development in the valley continues to increase the quantity of groundwater pumped from the aquifers will also increase. This means that regional subsidence will continue. In some localized areas subsidence may only be a few inches. In others it may be several feet. To appreciate the magnitude of how bad this phenomenon can be, in the Las Vegas Valley there are well heads that were set in place many years ago that now protrude nearly six feet out of the ground. The problem would be much worse if it were not for the establishment of a program that manually recharges some of the aquifers that exist there. An extreme case involving regional ground subsidence took place in the San Joaquin Valley in California. Due to heavy groundwater pumping to supply the agricultural fields, the area subsided approximately 27 feet between the years 1955 and 1980.

The positive thing about regional subsidence is that because it is occurring over a wide area it does not usually cause structural distress. The reason for this is that if the ground has subsided 3 feet on the east side of the property it has likely subsided 3 feet on the west side of the property as well. Since the subsidence is the same on both ends of the property, the structure has settled uniformly and no cracking results. In reality, unless you were surveying the site, you would not know that the ground has subsided at all.

Unfortunately, there is another phenomenon known as fissuring that sometimes results in conjunction with regional subsidence. Fissures appear to be related to groundwater pumping and often occur in the vicinity of faults.

The Las Vegas Valley has a tectonic fault system that cuts through the valley in predominantly a south to north direction. Some of these faults are known to be tectonically active and others were active in the not too distant past. A fault is simply the contact plane between two large blocks of the earth's crust that are sliding past each other in order to relieve stress build-up. When there is movement along a fault, stresses and strains occur within some variable distance beyond the fault plane itself. Although no visible break may be evident at the surface, weak planes or tension fractures may have developed well below the surface. When wells begin to draw water from the aquifers these tension fractures can start to open deep within the ground. Eventually they may propagate all the way to the ground surface where they are known as fissures. A fissure may only be a few inches wide deep within the ground. However, as it nears the surface water from sources such as irrigation or rainfall can create erosion within the fissure and widen it to several feet. In time the fissure may fill-in by collapsing within itself at which time it becomes no longer visible at the surface.

The Las Vegas Valley is known to have many fissures. Many more likely exist that have not yet reached the surface or have filled themselves in and are no longer visible. These present tremendous hazards to structures. The filled-in fissure is a weak soil zone that can easily be reactivated by continued groundwater pumping. If the fissure is beneath a structure, the structure will likely undergo tremendous distress. Subsurface exploration for locating fissures is costly and is not generally provided in the scope of services for the geotechnical investigation. If requested, the study can be performed as a supplemental investigation.

Localized Subsidence

Localized subsidence is subsidence that occurs on a small scale such as an area of a particular lot. Most of this is caused by improper design or management of moisture-related items on the property. Other times it is merely misfortune due to breaks in subsurface water lines.

Many of the soils in the Las Vegas Valley are known as hydro-collapsible soils and consolidate upon the addition of moisture. These soils were deposited in a loose condition out of a slow moving body of water such as a lake. The small point of contact between individual soil particles provides enough friction to prevent the soil particles from moving relative to one another. However, when enough moisture is added the friction is reduced. The particles then begin to move into a more dense condition. When billions of particles move or consolidate in this manner, the result is ground subsidence.

The only mechanism necessary for this to occur is the addition of moisture. This can come from irrigation, septic systems, rainfall, or leaks in plumbing lines. Since the source of moisture is localized the subsidence is also localized. The amount of subsidence that can occur depends upon a number of factors such as the thickness of the loose soil deposit, how loose the soil is, the amount of moisture added, and the load upon the consolidating soil.

Engineering Solutions to Building Upon Collapsible Soils

When the hydro-collapsible soils are deposited near the ground surface and the thickness of these soils is not too great, the easiest way to mitigate them is to excavate them out from under the structure area. These soils can be recompacted in the excavation to a dense condition where they will no longer consolidate upon the addition of moisture. Unfortunately, nature has deposited many of these hydro-collapsible soils in very thick layers. The thickness of these layers generally makes it uneconomical to remove all of them. Therefore, what is normally recommended is to remove a portion of them, provide additional reinforcement in the foundations, and to recommend procedures that will minimize the ability for those hydro-collapsible soils that remain in-place to become wetter.

By removing and recompacting the near surface hydro-collapsible soils, a dense structural building pad is created. This pad would be more than suitable to support a structure with minimal settlement provided the hydro-collapsible soils below the pad do not experience a moisture increase. However, since there is still a risk of these soils becoming wet, additional steel reinforcement is added to the foundations in order to provide the foundations some added ability to resist deflection should some settlement occur. If the foundations were to deflect, cracking can result.

Hardened concrete is extremely strong in compression, but very weak in tension. Steel, on the other hand, is strong in tension. The building code has historically required that a single No. 4 steel-reinforcing bar be placed near the lower portion of the concrete footings. The intent of this bar is to increase the footing's resistance to bending by providing tensile strength at the lower portion of the footing where there would otherwise be very little. The bending that the building code tries to resist comes from the load imposed upon the footing by the structure's walls and roof. Unfortunately, what the code does not account for are the deflections of the footing that might occur due to the loss of soil support below the footing as the hydro-collapsible soils consolidate. The loss of soil support can create bending of the footing in either direction. Therefore, it is necessary to reinforce the footing by placing steel reinforcement near the top and the bottom of the footing. Furthermore, a single No. 4 bar is not enough. After seeing many distressed structures, we concluded that additional, and larger reinforcement was required to reduce the risk of distress. Typically, two No. 5 steel-reinforcing bars are now recommended near the top of the footing, and two additional bars near the bottom. This reinforcement schedule has to date been more successful in minimizing problems. However, it should not be considered a guarantee of a problem-free structure. It is a cost-effective solution that greatly reduces the number and severity of problems.

Removing and recompacting some of the soils and adding reinforcement are only two of the three factors in minimizing distress. The third is absolutely the most important. It is simply keeping moisture from getting to the hydro-collapsible soils. Even if the first two factors are not even performed, the soils will still adequately support a structure. Barring heavy ground shaking such as from an earthquake, the collapse of these soils and the subsequent settlement and distress of the structure should only occur if the hydro-collapsible soils become wet.

Another technique used in hydro-collapsible soils is to support the structure on a deep foundation. A deep foundation can provide support by bearing the tips of the foundation upon a dense stratum. Another method is to provide support through the friction that develops between the sides of the deep foundation and the adjacent soils.

Deep foundations installed in the southern Nevada area are typically cast-in-place concrete drilled piers. To construct these piers, deep holes are drilled into the ground and the soil is removed. Steel reinforcement is set inside and concrete is placed into the excavations. For the end-bearing method, another type of pier known as a helical pier is sometimes used. These are made of steel and are screwed into the ground.

With the end-bearing method the loads from the structure are transferred to the concrete grade beam connecting the individual piers and then onto the piers themselves. The loads then transfer down the length of the piers to the pier tips and finally onto the soil. Since each pier is spread apart from the others, the load that each pier carries is very large. Therefore, the soil below the pier tips must be dense or settlement will occur. Unfortunately, many places in the Las Vegas Valley where deep hydro-collapsible soils occur do not provide a dense soil stratum at a depth where end-bearing piers would be cost effective.

The other type of pier is one that is supported through sidewall friction. In this case, pressure exerted on the sides of the pier from the adjacent soil creates friction that holds the pier in place. The larger the pressure exerted from the soil, the greater is the friction acting upon the pier. The soil pressure comes from the weight of the soil lying upon other soil below. Therefore, there is much greater pressure at depth than there is near the surface. Subsequently, there is much greater friction generated at depth. In fact, near the surface the weight of the concrete in the pier is greater than the friction that is developed there. This means that friction piers have to be installed to substantial depths in order for the soil to develop enough frictional carrying capacity to support the loads. The deeper the pier is the more costly it is to construct. Therefore, installing friction piers is often not economical.

Preventing Moisture from Reaching In-place Hydro-collapsible Soils

A. Drainage

The first step in trying to prevent moisture from entering the hydro-collapsible soils that remain under the structural pad is to ensure that proper drainage is provided around the structure. Proper drainage is intended to allow surface water to flow over the ground away from the structure to some safe distance where it will not impact the foundation elements or subsurface hydro-collapsible soils. How far should this distance be? As far as practical. The actual distance will depend upon how much water will be accumulating in any given area. It will also depend upon where the water flows. The grade should be established so that the water maximizes the potential for evaporation and minimizes the potential for seepage into the ground. Areas that will act as ponds should not be allowed.

The minimum gradient to allow for drainage should be established at no less than 5 percent. Steeper gradients are encouraged as long as the gradients do not become so steep that they are a physical hazard or that they lead to erosion.

The architectural design of most roofs have a series of ridges and valleys. During heavy rainfall some of these roofs can collect large quantities of water which are concentrated in the valleys prior to spilling off of the roof. When the water pours onto the ground it lands near the foundations and often becomes trapped in small planter areas. The water then percolates into the ground past the foundations and into the hydro-collapsible soils. Therefore, wherever roofs will deposit water in this manner, flatwork should be strategically placed to intercept the water and divert it away from the structure. As an alternative, roof drains may be installed to collect and divert the water. In either case the water should be transported to an area far away from the structure where it will no longer be allowed to percolate near the soils below the structural pad. If roof drains will be used they should be periodically checked to ensure that they are free of debris that may otherwise impede the flow of water.

B. Landscaping

Landscaping water produces a serious risk of moisture infiltrating into the hydro-collapsible soils. Water supplied to plants can either be taken up by the plants, evaporated into the atmosphere, or infiltrated into the ground. It is the ground infiltration that is of great concern. The excess water infiltrating the ground will migrate downward and outward. Eventually this water may reach the hydro-collapsible soils under the structural building pad causing settlement of the structure. Therefore, over-watering must not be allowed. When plants are watered, only that amount of water necessary to sustain the life of the vegetation should be used.

It is best to utilize a desert landscape because the plants used are sparsely spaced and are fed through a drip irrigation system which places only small quantities of water into the ground. Planting grass generally invites trouble because, in an effort to keep grass looking lush and green all year, many people over-water their lawns. Not only does this produce excess water, it produces it in large quantities. All the additional water put into the ground leads to higher risk of structural distress.

The distance between the structure and the vegetation is also important. If plants are planted adjacent to the structure they should be contained in raised planter boxes to avoid any excess water supplied to these plants from entering the ground near the foundations. For other plants that will be placed in the ground, they should not be placed close to the structure. The farther away the plants are placed, the farther any excess water has to travel to reach the hydro-collapsible soils.

The irrigation lines for the plants should not be placed near the structure. Irrigation lines commonly leak. The farther these lines are away, the farther that any leaking water has to travel to cause problems. The areas where these lines are in place should be periodically monitored to see if wet areas exist where no wet areas should be. A wet area could be a sign of a leaking line. The sooner a leak can be discovered and repaired, the less damage the leak will cause.

Visqueen that is placed as ground cover to prevent the growth of weeds should not be used near the structure. The visqueen acts a barrier preventing evaporation. Without evaporation, there is more infiltration and more risk of distress.

C. Septic Systems

Like landscaping water, septic systems produce a great risk of structural distress and therefore, understanding septic systems is of great importance. In order to install a septic system, a soil percolation rate must be determined. The percolation rate is a measure of how fast the water flows through the soil, and is determined by a percolation test. A faster percolation rate requires a smaller leach field.

The septic system consists of a septic tank and a leach field. Wastewater enters the tank from the structure and eventually discharges the effluent into perforated plastic leach lines that lay in the ground within a bed of gravel known as leach rock. Some of the moisture of the effluent will evaporate away while the remainder will percolate into the ground. If the septic system is too close to the structure, the moisture does not have to travel far to create a problem. Therefore, the septic system should always be placed far from the structure. Problems can occur even with a properly designed and installed septic system. The problems are magnified when the system is undersized or is not functioning as it should.

For a leach field to operate properly, the leach lines must lay at a particular gradient in order to ensure that the effluent disperses evenly rather than concentrating most of the effluent to one area. When these systems are installed into hydro-collapsible soils, the effluent that infiltrates the ground begins to cause subsidence under the leach field. If only one leach line is installed, the effect will be that the leach line will settle more at the end closest to the septic tank producing an upward gradient in the line. Since the effluent does not want to travel uphill within the line, it will concentrate its discharge near the entrance to the line as opposed to along the entire length of line. The effective functional length of the leach line is, therefore, greatly reduced. This leads to further infiltration and less evaporation. The problem is similar when more than one line is installed. In this case, if one line settles more than the other the higher line may no longer receive any effluent. Therefore, only one line becomes functional and the effective size of the leach field is reduced.

The rate and amount of settlement under the leach field increases when the field is not operating properly. In deep hydro-collapsible soils the problem can become so severe that tension cracks develop and begin to encircle the leach field. These cracks can radiate far from the leach field and cause severe problems to a structure.

Leach fields should be monitored periodically to ensure that the proper gradients of the leach lines are maintained.

D. Pools

Pools obviously are designed to hold water. Unfortunately, the plumbing lines of many pools are prone to leaking. When this occurs they supply water into the ground which will then cause collapse of the soils supporting the pool. This can lead to cracking of the pool shell and further leaking. The water that infiltrates the ground may eventually make its way toward the structure. If this occurs distress is likely to follow. Therefore, pools always create a potential for distress in hydro-collapsible soils. If the owner chooses to install one it should be located far from the structure.