Gordon Spilker Huber
Geotechnical Consultants, Inc.

REPORT
GEOTECHNICAL STUDY
PROPOSED PARK CITY HEIGHTS DEVELOPMENT
SOUTHWEST OF THE INTERSECTION OF US HIGHWAY 40 AND HIGHWAY 248 (KEETLEY JUNCTION)
PARK CITY, SUMMIT COUNTY, UTAH

Submitted To:

Plumb and Dalton
809 Edgehill Road
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Submitted By:

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June 9, 2006

Job No. 0013-007-05

GSH
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Plumb and Dalton
809 Edgehill Road
Salt Lake City, Utah 84103

Attention: Mr. Walter Plumb

Gentlemen:

Re: Report
Geotechnical Study
Proposed Park City Heights Development
Southwest of the Intersection of US Highway 40
and Highway 248 (Keetley Junction)
Park City, Summit County, Utah

1. INTRODUCTION

1.1 GENERAL

This report presents the results of our geotechnical study performed at the site of the proposed Park City Heights Development, which is located southwest of the intersection of US Highway 40 and Highway 248 (Keetley Junction) in Park City, Summit County, Utah. The general location of the site with respect to major topographic features and existing facilities, as of 1999, is presented on Figure 1, Vicinity Map. A more detailed layout of the site showing the proposed locations of lots and roadways, site-specific topography, and existing facilities is presented on Figure 2, Site Plan. The locations of the test pits excavated in conjunction with this study are also presented on Figure 2.

1.2 OBJECTIVES AND SCOPE

The objectives and scope of our study were planned in discussions between Mr. Walter Plumb of Plumb and Dalton, and Mr. Bill Gordon of Gordon Spilker Huber Geotechnical Consultants, Inc. (GSH).

In general, the objectives of this study were to:

1. Define and evaluate the subsurface soil, bedrock, and groundwater conditions across the site.

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2. Provide appropriate foundation, earthwork, and pavement recommendations to be utilized in the design and construction of the proposed development.

In accomplishing these objectives, our scope has included the following:

1. A field program consisting of the excavation, logging, and sampling of 20 test pits.

2. A laboratory testing program.

3. An office program consisting of the correlation of available data, engineering analyses, and the preparation of this summary report.

1.3 AUTHORIZATION

Authorization was provided by returning a signed copy of our Professional Services Agreement No. 05-1118 dated November 22, 2005.

1.4 PROFESSIONAL STATEMENTS

Supporting data upon which our recommendations are based are presented in subsequent sections of this report. Recommendations presented herein are governed by the physical properties of the soils encountered in the exploration test pits, projected groundwater conditions, and the layout and design data discussed in Section 2., Proposed Construction, of this report. If subsurface conditions other than those described in this report are encountered and/or if design and layout changes are implemented, GSH must be informed so that our recommendations can be reviewed and amended, if necessary.

Our professional services have been performed, our findings developed, and our recommendations prepared in accordance with generally accepted engineering principles and practices in this area at this time.

2. PROPOSED CONSTRUCTION

The proposed development will consist of a total of 150 single-family residential structures. The structures will be one- to two-stories above grade. Full-depth and partial-depth basements are desired. Above grade, the structures will be of wood-frame construction with wood, brick, stucco, or stone wall veneer. The estimated vertical structural loads are 1 to 3 kips per lineal foot for continuous wall foundations and 15 to 25 kips for isolated column footings.

Overall site development for roadways and general grading will require a moderate amount of earthwork in the form of cuts and fills. We estimate that maximum site grading cuts and fills will generally be on the order of five to six feet.

Long-term traffic over the roadways will consist of a moderate volume of automobiles and light trucks, and a light volume of medium- and heavy-weight trucks (typical subdivision traffic). Truck traffic will be high during "build out."

3. SITE INVESTIGATIONS

3.1 FIELD PROGRAM

In order to define and evaluate the subsurface soil, bedrock, and groundwater conditions, a total of 20 test pits were explored to depths ranging from 4 to 15 feet below existing grade. The test pits were excavated using a rubber-tired backhoe. Approximate locations of the test pits are presented on Figure 2.

The field portion of our study was under the direct control and continual supervision of an experienced member of our geotechnical staff. During the course of the excavation operations, a continuous log of the subsurface conditions encountered was maintained. In addition, relatively undisturbed and small disturbed samples of the typical soils encountered were obtained for subsequent laboratory testing and examination. The soils were classified in the field based upon visual and textural examination. These classifications have been supplemented by subsequent inspection and testing in our laboratory. Detailed graphical representation of the subsurface conditions encountered is presented on Figures 3A through 3T, Log of Test Pits. Soils were classified in accordance with the nomenclature described on Figure 4, Unified Soil Classification System. Bedrock was classified in accordance with the nomenclature described on Figure 5, Rock Description Terminology.

Following completion of excavating and logging, each test pit was backfilled. Although an effort was made to compact the backfill with the backhoe, backfill was not placed in uniform lifts and compacted to a specific density. Consequently, settlement of the backfill with time is likely to occur. It is recommended that test pit backfill be removed and recompacted to the requirements of structural fill prior to the construction of improvements over these areas.

Following completion of drilling operations, one and one-quarter-inch diameter slotted PVC pipe was installed in Test Pit TP-1 to a depth of nine and one-half feet in order to provide a means of monitoring the groundwater fluctuations.

3.2 LABORATORY TESTING

3.2.1 General

In order to provide data necessary for our engineering analyses, a laboratory testing program was initiated. The program included moisture and density, Atterberg limits, collapse/swell-consolidation, gradation, and chemical tests. The following paragraphs describe the tests and summarize the test data.

3.2.2 Moisture and Density Tests

To aid in classifying the soils and to help correlate other test data, moisture and density tests were performed on selected undisturbed samples. The results of these tests are presented to the right on the test pit logs, Figures 3A through 3T.

3.2.3 Atterberg Limits Test

To further aid in classifying and in determining the expansive potential of the site soils, an Atterberg limits test was performed on a selected sample. Results of the tests are as follows:

Test Pit No.	Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	Soil Type
TP-1	5.5*	43	24	19	CL
TP-2	2.0	51	21	30	CH
TP-9	1.5	80	13	68	CH
TP-12	1.5	57	33	24	MH
TP-12	10.5*	46	19	27	CL
TP-19	10.0*	45	19	26	CL

* Test performed on portion of the GC sample passing the No. 40 sieve.

3.2.4 Collapse/Swell-Consolidation Tests

To provide data necessary for our settlement analyses, collapse/swell-consolidation tests were performed on a representative sample of the fine-grained soils encountered at the site. The collapse/swell portion of the overall test was performed in accordance with the following procedure:

1. The sample is loaded to a specified axial pressure at in-situ moisture content.
2. The resulting axial deflection is measured and recorded.
3. The sample is saturated.
4. The resulting collapse/swell is measured and recorded.

A tabulation of the results of the collapse/swell portion of the tests are presented below:

Test Pit No.	Depth (feet)	Soil Type	Axial Pressure At time of Saturation (psf)	Swell (+) (percent)	Swell Pressure (psf)
TP-13	1.5	CL	1,600	4.68	3,500

The test results indicate that the soils are generally highly over-consolidated and will exhibit expansive characteristics under the anticipated loading conditions. Detailed results of the tests are maintained within our files and can be transmitted to you, at your request.

3.2.5 Gradation Test

Gradation tests were performed to aid in classifying soils. The tests results are tabulated below:

Sieve Size	Percent Passing		
	TP-1 @ 5.5'	TP-12 @ 10.5'	TP-19 @ 10.0'
2"	-	-	-
1-½"	100	100	100
1"	-	-	-
¾"	53.7	44.8	78.3
½"	-	-	-
⅜"	36.7	41.0	59.4
No. 4	28.1	38.0	47.7
No. 10	24.2	34.9	39.3
No. 20	-	-	-
No. 40	21.3	30.8	32.1
No. 100	18.2	26.2	27.3
No. 200	15.8	22.6	24.5
Soils Classification	GC	GC	GC

3.2.6 Chemical Tests

To determine if the site soils will react detrimentally with concrete, chemical tests were performed on a representative sample of the silty clay soils encountered in Test Pit TP-6 at a depth of two feet below existing grade. The results of the chemical tests are presented below:

Test Pit No.	Depth (feet)	pH	Total Water Soluble Sulfate (ppm)
TP-19	2.0	7.18	36

4. SITE CONDITIONS

4.1 SURFACE

The Park City Heights Development is located southwest of the intersection of US Highway 40 and Highway 248 (Keetley Junction) in Park City, Summit County, Utah. The development is irregularly shaped and consists of two parcels. The northern parcel shown on Figure 2 contains 23.6 acres will be used for open space, and was not included in this study. The south parcel contains 98.4 acres that will be used for open space, 150 single-family residential lots, and access roadways.

The planned residential parcel is approximately 2,000-feet by 3,000-feet. The site consists of open areas with grasses and brush covering the majority of the site. Some scattered trees are located across the site. The site is bounded by roadways to the north and east. Undeveloped land and residential structures primarily bounds the site to the south and west.

The site slopes downward to the northeast with an overall elevation change of approximately 200 feet. The slope is approximately 12 horizontal to 1 vertical. Some drainage swales bisect the site. No signs of past or eminent slope instability were noted.

4.2 SUBSURFACE SOIL AND GROUNDWATER

The subsurface sequence generally consists of surficial clays underlain by clayey gravels with some sands and generally occasional cobbles. The clays generally extend to depths ranging from 2.5 to 9.5 feet. These soils contain trace to some sands and gravels and are moderately to highly plastic. These soils exhibit high expansive characteristics. The upper as much as 6 to 12 inches of the soils contain major roots which have been classified as topsoil. The upper 9 to 12 inches are generally loose as the result of normal weathering. Clays below the loose surface zone will exhibit moderate strength and compressibility characteristics.

The surface clays are, in turn, generally underlain by a fairly thick sequence of medium dense to dense clayey gravels with some sand containing occasional cobbles and in areas occasional boulders. These soils generally extend to the maximum depths penetrated and will exhibit high strength and low compressibility characteristics. Tests run on the fine-grained portion of the sands and gravels indicate that these soils are non-expansive.

Bedrock was encountered at some of the test pit locations. The bedrock appears to consist of quartzite and will exhibit relatively high strength and low compressibility characteristics. Excavation refusal was encountered at some of the test pit locations. The test pits were excavated with a rubber-tired backhoe (JCB 214 S) with a 30-inch bucket. Depth to granular soils, and depth to refusal (or near refusal) in bedrock are presented. Depth of refusal or near-refusal is defined as the depth at which the 30-inch bucket was either scrapping on fractured bedrock or progress was significantly slowed due to the presence of large particles of bedrock. The following table summarizes the soil/bedrock conditions encountered:

Test Pit No.	Depth to Dense Granular Soils (feet)	Depth to Refusal or Near Refusal (feet)
TP-1	3.0	-
TP-2	4.5	-
TP-3	4.0	-
TP-4	3.0	4.5
TP-5	-	9.0
TP-6	-	6.0
TP-7	-	5.0
TP-8	-	4.0
TP-9	5.0	5.5
TP-10	4.5	12.0
TP-11	2.5	-
TP-12	3.5	-
TP-13	9.5	-
TP-14	6.0	-

Test Pit No.	Depth to Dense Granular Soils (feet)	Depth to Refusal or Near Refusal (feet)
TP-15	9.0	-
TP-16	3.0	6.0
TP-17	4.0	-
TP-18	5.0	6.5
TP-19	7.5	-
TP-20	4.0	-

During and immediately following excavation, groundwater was not encountered to the depths penetrated.

5. DISCUSSIONS AND RECOMMENDATIONS

5.1 SUMMARY OF FINDINGS

The most significant geotechnical aspects of the site are:

1. The expansive clays encountered in the upper two and one-half to nine and one-half feet encountered across the site.

2. Shallow bedrock within portions of the site.

Under no circumstances should the foundations and primary at-grade slabs be established directly upon the expansive soils. In most cases basements will penetrate through these deposits. Where the clays are deeper, they must be over-excavated and replaced with non-expansive structural fills. In garage areas as a minimum the clays beneath the slabs must be removed to a depth of at least three feet and replaced with non-expansive structural fill. Similar procedures are recommended for outside slabs (sidewalks, stairways, driveways, etc.). With partial replacement, some vertical movements could still occur if the clays experience significant moisture variations. We recommend that the unsuitable soils be removed and used as non-structural fill.

The proposed structures may then be supported on conventional spread and continuous wall foundations established on suitable natural soils or bedrock and/or structural fill extending to suitable natural soils or bedrock.

Some excavations for the below grade portions of the structures will be difficult due to the presence of bedrock.

horizontal to one vertical. If clean granular soils are encountered, or if excessive sloughing occurs, the sideslopes must be flattened to one horizontal to one vertical.

To reduce disturbance, we recommend that excavations for footings be accomplished utilizing a backhoe with a smooth-lip bucket.

In bedrock cuts up to 12 feet can be established with slopes no steeper than one-quarter horizontal to one vertical. Loose and raveling bedrock and soils are anticipated from deep steep cuts. Therefore, the face of the deeper-steeper slopes must be protected by anchoring chain-link fencing from the crest to the toe. Field data and past experience in the area indicates excavations into bedrock, 5 to 10 feet, can generally be accomplished using heavy construction equipment or a "stringer." Drilling and blasting could possibly be required for localized deeper areas.

All excavations must be inspected periodically by qualified personnel. If any signs of instability or excessive sloughing are noted, immediate remedial action must be initiated.

5.2.3 Structural Fill

Structural fill is defined as all fill that will ultimately be subjected to structural loadings, such as those imposed by footings, floor slabs, pavements, etc. Structural fill will be required as backfill over foundations and utilities, and as replacement fill beneath some footings and floor slabs. All structural fill must be free of sod, rubbish, topsoil, frozen soil, and other deleterious materials. Structural site grading fill is defined as fill placed over relatively large open areas to raise the overall grade. For structural site grading fill, the maximum particle size should generally not exceed four inches; however, occasional larger particles, not exceeding eight inches in diameter, may be incorporated if placed randomly in a manner such that "honeycombing" does not occur and the desired degree of compaction can be achieved. The maximum particle size within structural fill placed within confined areas should generally be restricted to two inches.

Because of the expansive nature of the fine-grained soils, these soils will need to be removed and stockpiled for subsequent landscaping purposes. Fine-grained soils with a Plasticity Index (PI) less than 18 percent may be used as structural site grading fill. It should be noted that unless moisture control is maintained, utilization of soils with a relatively high fines content as structural fill will be difficult, if not impossible, during wet and cold periods of the year. Only granular soils are recommended as structural fill in confined areas, such as around foundations and within utility trenches. We recommend that all granular structural fill including the on-site soils consist of a well-graded mixture of sands and gravels with at least 20 percent fines (material passing the No. 200 sieve). The purpose of the minimum fines percentage is to provide a fill when compacted that will exhibit low permeability characteristics.

Non-structural site grading fill is defined as all fill material not designated as structural fill or unsuitable material, and may consist of any cohesive or granular soils not containing excessive amounts of degradable material.

5.2.4 Fill Placement and Compaction

All structural fill should be placed in lifts not exceeding eight inches in loose thickness. Beneath buildings, the fill should be compacted to at least 95 percent of the maximum dry density as determined by the AASHTO[1] T-180 (ASTM[2] D-1557) compaction criteria. If the fill is more than 7 feet thick, the fill must be compacted to at least 97 percent of the above-defined criteria. Structural fills less than 5 feet thick, outside the building areas, and beneath outside flatwork and pavements, should be compacted to at least 90 percent of the above-defined criteria. If over 5 feet thick, they should be compacted to at least 92 percent.

Prior to the placement of structural site grading fill, pavements, floor slabs, or footings, the exposed subgrade should be prepared as discussed in Section 5.2.1, Site Preparation, of this report. In confined areas, subgrade preparation should consist of the removal of all loose or disturbed soils.

Non-structural fill may be placed in lifts not exceeding 12 inches in loose thickness and compacted by passing construction, spreading, or hauling equipment over the surface at least twice.

5.2.5 Utility Trenches

All utility trench backfill material below structurally loaded facilities (flatwork, floor slabs, roads, etc.) should be placed at the same density requirements established for structural fill. If the surface of the backfill becomes disturbed during the course of construction, the backfill should be proofrolled and/or properly compacted prior to the construction of any exterior flatwork over a backfilled trench. Proofrolling may be performed by passing moderately loaded rubber tire-mounted construction equipment uniformly over the surface at least twice. If excessively loose or soft areas are encountered during proofrolling, they should be removed to a maximum depth of two feet below design finish grade and replaced with structural fill.

Most utility companies and City-County governments are now requiring that Type A-1 or A-1a (AASHTO Designation – basically granular soils with limited fines) soils be used as backfill over utilities. ***Because of the expansive soils, we strongly recommend that the fills used as backfill meet the requirements stated in Section 5.2.3 of this report***. These organizations are also requiring that in public roadways the backfill over major utilities be compacted over the full depth of fill to at least 96 percent of the maximum dry density as determined by the AASHTO T-180 (ASTM D-1557) method of compaction. We recommend that as the major utilities continue onto the site that these compaction specifications are followed.

1 American Association of State Highway and Transportation Officials
2 American Society for Testing and Materials

5.3 SPREAD AND CONTINUOUS WALL FOUNDATIONS

5.3.1 Design Data

The proposed structure may be supported upon conventional spread and continuous wall foundations established on suitable non-expansive natural soils and/or structural fill extending to suitable natural soils. For design, the following parameters are provided:

Minimum Recommended Depth of Embedment for Frost Protection	- 36 inches
Minimum Recommended Depth of Embedment for Non-frost Conditions	- 15 inches
Recommended Minimum Width for Continuous Wall Footings	- 18 inches
Minimum Recommended Width for Isolated Spread Footings	- 24 inches
Recommended Net Bearing Pressure for Real Load Conditions	
Suitable Natural Soils and/or Structural Fill Extending to These Soils	- 3,000 pounds per square foot
Bedrock	- 5,000 pounds per square foot
Bearing Pressure Increase for Seismic Loading	
Soils	- 50 percent
Bedrock	- 100 percent

No specific minimum depth of embedment will be required for footings established upon massive bedrock; although, the footings must be appropriately "tied" to the underlying bedrock to provide appropriate lateral resistance.

The term "net bearing pressure" refers to the pressure imposed by the portion of the structure located above lowest adjacent final grade. Therefore, the weight of the footing and backfill to lowest adjacent final grade need not be considered. Real loads are defined as the total of all dead plus frequently applied live loads. Total load includes all dead and live loads, including seismic and wind.

5.3.2 Installation

Under no circumstances should the footings be founded on expansive soil, non-engineered fill, loose or disturbed soils, sod, rubbish, construction debris, other deleterious materials, frozen soils, or within ponded water. If unsuitable soils are encountered, they must be completely removed and replaced with structural fill. If the natural soils upon which the footings are to be established become loose or disturbed, they must be removed and replaced with structural fill. If the structural fill upon which the footings are to be established becomes disturbed, it should be recompacted to the requirements of structural fill or be removed and replaced with structural fill.

The width of structural replacement fill, as required below footings, should be extended laterally at least six inches beyond the edges of the footings in all directions for each foot of fill thickness beneath the footings. For example, if the width of the footing is two feet and the thickness of the structural fill beneath the footing is one foot, the width of the structural fill at the base of the footing excavation would be a total of three feet.

5.3.3 Settlements

Maximum settlements of foundations designed and installed in accordance with recommendations presented herein and supporting maximum anticipated loads as discussed in Section 2., Proposed Construction, are anticipated to be on the order of one-quarter to one-half of an inch.

Approximately 60 percent of the quoted settlement should occur during construction.

5.4 ALTERNATE FOUNDATIONS

Under most structures the basement excavations will penetrate through the expansive clays. In some areas, over-excavation beyond the base of the basement excavation will be required to remove the unsuitable clay. If the over-excavation exceeds six to eight feet, alternate systems may be considered. One alternate would consist of helical piers extending to the underlying non-expansive soils. A two-inch void is recommended between the base of the pier cap and grade beams and the underlying expansive soils. Helical piers can also be utilized to support outside columns.

Footings in garage areas must be installed as per the recommendation for the primary building.

5.5 SUBDRAINS

A permanent foundation/chimney subdrain system will be required around the outside of subgrade wall.

The perimeter subdrain pipe should consist of a minimum of four-inch diameter, slotted or perforated pipe with the invert established at least 18 inches below the top of the lowest adjacent slab. The pipe should be encased in a one-half to one-inch minus clean gap-graded crushed gravel extending two inches below, laterally, and up continuously at least 12 inches above the top of the lowest adjacent slab. The same granular material could be utilized as the chimney drain against the subgrade walls. In all cases, the gravels must be separated from the natural soils or finer-grained backfill with a geotextile, such as Mirafi 140N or equivalent. As an alternate a synthetic drain board, such as Miradrian or equivalent, can be used for the chimney subdrain. The slope of the pipe should be at least 0.25 percent to a suitable point of gravity discharge; such as a sump within or outside the perimeter of the below-grade portion of the structure or by gravity down-gradient. Prior to the installing the gravels, we recommend that the outside walls adjacent to habitable areas be appropriately waterproofed. If the areas are mechanical areas or parking dampproofing should be adequate.

5.6 LATERAL RESISTANCE

Lateral loads imposed upon foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footings and the supporting soils. In determining frictional resistance, a coefficient of 0.45 should be utilized. Passive resistance provided by properly placed and compacted granular structural fill above the water table may be considered equivalent to a fluid with a density of 300 pounds per cubic foot. Below the water table, this granular soil should be considered equivalent to a fluid with a density of 150 pounds per cubic foot.

A combination of passive earth resistance and friction may be utilized provided that the friction component of the total is divided by 1.5.

5.7 LATERAL PRESSURES

The lateral pressure parameters as presented within this section assume that the backfill will consist of granular soil placed and compacted in accordance with the recommendations presented herein. The lateral pressures imposed upon subgrade facilities will therefore be basically dependent upon the relative rigidity and movement of the backfilled structure. For active walls, such as retaining walls which can move outward (away from the backfill), granular backfill may be considered equivalent to a fluid with a density of 45 pounds per cubic foot in computing lateral pressures. For more rigid basement walls that are not more than 10 inches thick and 12 feet or less in height, granular backfill may be considered equivalent to a fluid with a density of 55 pounds per cubic foot. For very rigid non-yielding walls, granular backfill should be considered equivalent to a fluid with a density with at least 75 pounds per cubic foot. The above values assume that the surface of the soils slope behind the wall is horizontal and that the granular fill has been placed and lightly compacted, not as a structural fill. If the fill is placed as a structural fill, the values should be increased to 60 pounds per cubic foot, 75 pounds per cubic foot, and 130 pounds per cubic foot, respectively. If the slope behind the wall is two horizontal to one vertical, the values for purely active walls and basement walls should increase to 57 pounds per cubic foot and 67 pounds per cubic foot, respectively.

The above equivalent fluid pressures are for static loading conditions. For seismic loading, a uniform pressure of 100 pounds per square foot should be added. It should be noted that the lateral pressures as quoted assume that the backfill materials will not become saturated. If the backfill becomes saturated, the above values may be decreased by one-half; however, full hydrostatic water pressures will have to be included.

5.8 FLOOR SLABS

Floor slabs are recommended to be established upon properly prepared existing suitable non-expansive soils and/or upon structural fill extending to suitable natural soils. Topsoil, expansive clay, and non-engineered fills are considered suitable. If proper moisture control can be provided and some movement tolerated, at-grade slabs in garage areas can be supported upon three feet of suitable structural fill extending to expansive soils. Settlements of lightly loaded floor slabs not affected by expansive soils should be negligible. Upward movements of possibly one inch could occur if underlying expansive soils were to experience a significant increase in moisture content.

Floor slabs within habitable areas and not underlain by expansive soils should be immediately underlain by a minimum four-inch layer of "free-draining" clean gap-graded gravel. When at-grade slabs are underlain by some deeper expansive soils a four-inch layer of aggregate base should be used.

5.9 PAVEMENTS

The natural fine-grained soils will exhibit poor pavement support characteristics when saturated or nearly saturated. Considering the clay as the design subgrade soils and the projected traffic conditions, the following pavement sections are recommended:

<u>Primary Roadway Areas</u>

(Moderate Volume of Automobiles and Light Trucks
with Occasional Medium and Heavy Trucks)
[50 equivalent 18-kip axle loads per day]*

4.5 inches	Asphalt concrete
6.0 inches	Aggregate base
10.0 inches	Granular subbase**
Over	natural clay subgrade

* This is based upon the "build-out" traffic over a period of three to five years.

** Natural granular soil and/or granular structural fill will satisfy this requirement.

For short cul-de-sac roadways, a thinner section may be applicable.

Asphalt concrete and base course components should meet the requirements and be placed in accordance with the Summit County specifications.

5.10 GEOSEISMIC SETTING

5.10.1 General

Utah municipalities have adopted the International Building Code (IBC) 2003 and International Residential Code for One- to Two-Family Dwellings 2003. The IBC 2003 determines the seismic hazard for a site based upon regional mapping of bedrock accelerations prepared by the United States Geologic Survey (USGS) and the soil site class (formerly soil profile type). The USGS values are presented on maps incorporated into the IBC and are also available based on latitude and longitude coordinates (grid points). In comparison, the former UBC (Uniform Building Code) generally placed the entire Wasatch front into a single seismic zone (Seismic Zone 3).

The structures must be designed in accordance with the procedure presented in Chapter 16 of the IBC 2003 edition.

5.10.2 Faulting

Based on our review of available literature, no active faults pass through or immediately adjacent to the site. The nearest mapped active fault is the Wasatch fault approximately 18 miles to the west (Hacker, 1993).

5.10.3 Liquefaction

Liquefaction is defined as the condition when loose, saturated granular soils lose their support capabilities due to excess pore water pressure buildup that develops during a seismic event. The on-site cohesive soils and granular soils which are not saturated will not liquefy.

5.10.4 Soil Class

For dynamic structural analysis, the Site Class "D" as defined in Table 1615.1.1, Site Class Definition of the IBC 2003, can be utilized.

5.10.5 Ground Motions

The IBC 2003 code is based on 1997 USGS (United State Geologic Survey) mapping, which provides values of short and long period accelerations for the Site Class "B"-"C" boundary for the 2 percent in 50 year event (2,475 year return period). This Site Class "B"-"C" boundary represents a hypothetical bedrock surface and must be corrected for local soil conditions. The following table summarizes the peak horizontal and short and long period accelerations for a 2 percent in 50-year event and incorporates a soil amplification factor for a Site Class "D" soil profile. Based on the site latitude and longitude (40.6683 north and 11.4638 degrees west, respectively), the values for this site are tabulated below:

Spectral Acceleration Value, T Seconds	**MCE 2% in 50 Yr event (2,475 yr return), % g**
Peak Horizontal Ground Acceleration	40.1
0.2 Seconds, (Short Period Acceleration, S_S)	100.3
1.0 Seconds (Long Period Acceleration, S_L)	52.9

MCE – Maximum considered earthquake

The IBC 2003 site accelerations are based on taking the above short and long period accelerations for the Maximum Considered Earthquake Event, and multiplying by two-thirds (⅔).

5.11 CEMENT TYPES

Laboratory tests indicate that the site soils contain negligible amounts of water soluble sulfates. Therefore, all concrete which will be in contact with the site soils may be prepared using Type I or IA cement.

5.12 WATER CONTROL

To reduce the possibility of water infiltration into expansive soils, as a minimum the following are recommended:

1. There should be a minimum 2 percent downward slope maintained away from the building for a distance at least 15 feet.

2. Water discharged from downspouts must be collected and discharged downslope at least 15 feet from the proposed structures.

3. All water-conveying utilities should be checked for leakage prior to backfilling and installed with flexible connections and joints.

4. Sprinkler heads should not be installed around the immediate perimeter of the structures.

5.13 REVIEW

Because of the expansive soils and their potential detrimental affects of proposed structures, pavements, etc., we request the opportunity of reviewing the site grading places and earthwork and pavement specifications prior to initiation of construction. In addition, periodic site observations during initial earthwork are recommended.

We appreciate the opportunity of providing this service for you. If you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

GSH Geotechnical Consultants, Inc.

Joshua M Whitney, EIT
Staff Engineer

Reviewed by:

William Gordon, State of Utah No. 146417
Professional Engineer

JMW/WJG:sn

Encl.	Figure	1,	Vicinity Map
	Figure	2,	Site Plan
	Figures	3A	through 3T, Log of Test Pits
	Figure	4,	Unified Soil Classification System
	Figure	5,	Rock Description Terminology

Addressee (6)