APPENDIX A-5

SOILS STUDIES & OPINION

Rollins, Brown and Gunnell	June 8, 1977
William Lund	May 1979
SHB Agra	April 22, 1994
AGEC	Oct 7, 2003

ROLLINS, BROWN AND GUNNELL

June 8 ,1997

SOILS INVESTIGATION

NORFOLK AVENUE
PARK CITY, UTAH

JUNE 1977

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June 8, 1977

J. J. Johnson and Associates
1515 Park Avenue
Park City, UT 84060

Gentlemen:

In accordance with your request, a soils investigation has been completed at the site of the proposed development known as Norfolk Avenue. This investigation was performed for the purpose of defining the subsurface soil and rock conditions throughout the project development so that satisfactory substructures and slope protection could be designed for the proposed facilities in this area.

Access to the hillside above the actual development area was limited because of the disturbing effects which the subsurface investigation would have on the natural vegetative growth on the hillside.

The conclusions and recommendations presented in the report are necessarily based upon the subsurface conditions throughout the development site. The results of the investigation along with pertinent recommendations relative to slope stability, foundation design and lateral earth pressures are discussed in the following sections of this report.

1. Site Geology and the Subsurface Soil Conditions

The characteristics of the subsurface soils throughout the development area were investigated by excavating 11 test pits varying in depth from approximately 6 feet to 15 feet below the existing ground surface. The location of the test pits is presented in Figure No. 1 while the logs are presented in Figures No. 2 through No. 7.

In most of the test pits, bedrock was encountered at a depth of between 7 and 8 feet below ground surface. However, some exceptions to this general pattern are noted. In test boring No. 2, the overburden extended to a depth of nearly 15 feet while in test boring No. 6, bedrock was encountered at a depth of 2 feet below the ground surface.

The subsurface profile throughout the development site can generally be described in terms of 4 zones. Zone No. 1 consists of a black silty topsoil which extends to a depth of between 1.5 and 3 feet below the existing ground surface. The surface zone is underlain by a granular zone varying in depth from 2 to 7 feet. The granular zone is composed of angular fragments in a matrix of silt. The angular fragments vary all the way from sand-size particles through gravels and cobbles. Zone No. 2 is underlain by a medium plastic clay of variable thickness which extends to the bedrock surface.

At all locations encountered during this investigation, the clay was in a medium to stiff condition. It should be noted that the clay zone is absent in test holes No. 1, 3, 4, 6, 8 and 9. However, the clay exists in a sufficient number of test holes throughout the site that its presence in the area cannot be neglected.

The bedrock underlying the overburden material is known as the Weber Quartzite Formation. In general, the Weber Quartzite Formation is a pale gray to tan quartzite and limey sandstone with some interbedded gray and white limestone and dolemite layers. At the proposed development, the Weber Quartzite Formation exposed in the trenches all showed the bedrock to be a light gray quartzite. From a structural standpoint, the development site is located on the northwest limb of the Park City anticline. The Park City anticline plunges to the northeast with the northwest limb having a stratigraphic dip to the northwest. The strike of the Weber Quartzite varies between North 30° West to North 45° West with a dip between 5° and 20° to the northwest. All joint sets observed throughout the development area essentially have high dip angles into the mountain. One joint set, however, was observed which has a low angle dip toward the mountain.

The results of this investigation indicate that there is no apparent joint set which will cause slippage down the slope of the mountain. However, the high angle joints will cause some fallout on any vertical wall cut perpendicular to the face of the mountain. The joint pattern exposed in the adit above Norfolk Avenue is presented in Figure No. 8. The symbols designating the strike and the dip of the joint sets are separated on the diagram for illustration purposes.

During the excavation of the test pits throughout the development area, in-place density tests were performed at three-foot intervals and miniature vane shear tests were performed in the clay materials. The results of the in-place density tests are presented on the boring logs, and it will be observed that the in-place dry density of the granular material varied from 112 pounds per cubic foot to 119 pounds per cubic foot, while the clay material varied from 93 to 95 pounds per cubic foot.

The miniature vane shear tests provide an indication of the undrained shearing strength of the clay materials. The miniature vane shear tests are designated as the torvane value on the test pit logs and are specified in terms of tons per square foot. The results of the miniature vane shear tests indicate that the subsurface clays are in a medium to stiff condition.

Each sample obtained in the field was subsequently classified in the laboratory according to the Unified Soil Classification System. The symbol designating the soil type according to this system is presented on the boring logs. A description of the Unified Soil Classification System is presented in Figure No. 9 and the full meaning of the various soil symbols can be obtained from this figure.

2. Slope Stability Considerations

Based upon the topographic map furnished our organization, the average slope throughout the development area is approximately 2 horizontal to 1 vertical. In order to perform a rigorous stability analysis at this site, it would be necessary to determine the soil profile of the entire hillside above the development area. Since it was not possible to excavate test pits up the hillside due to environmental considerations, the statements made in this section of the report are of necessity based upon the conditions which exist in the development area.

The results of the geological investigation indicate that there is no tendency for any slide to occur within the rock mass along the face of the slope in this area and that any failure that may occur will take place in the overburden material.

In attempting to obtain an estimate of the stability conditions for the overburden material at the site, two cases have been considered. Case No. 1 assumes an infinite slope with a depth of cohesionless soil equal to approximately 8 feet. Soil parameters obtained during the field and laboratory investigation have been used in the stability analysis for this case. A friction angle of 34° and a saturated unit weight of 133 pounds per cubic foot have been used in the analysis. If the entire mass of granular material above the bedrock is assumed to be saturated with seepage occurring parallel to the slope, the results of our analysis indicate that a factor of safety of 0.70 would occur for this site. Failure conditions would obviously occur under the stipulated conditions.

If the overburden material is less than saturated with no seepage occurring parallel to the slope, the results of the stability analysis indicate a factor of safety of 1.40. It is also apparent that the hillside would be stable under these conditions.

Case No. 2 considers the overburden material to consist of clay having the characteristics of the clay material observed in the lower portion of the soil profile at the site. The results of a stability analysis performed for this condition assuming the clay to be near saturated, but with no seepage parallel to the hillside, indicates a factor of safety of greater than 2. It is apparent from the above considerations that the stability of the overburden material at this location is a sensitive function of seepage parallel to the slope. If seepage conditions parallel to the slope can be restricted, the calculations indicate that the overburden material throughout the area would be stable. Our study of the entire area throughout the development site indicates that no slides or slumps exist throughout the overburden material and that the hillside is stable under its existing conditions.

In order to insure stability throughout the development area, we recommend that adequate subsurface and surface drainage be provided throughout the development area and that all disturbance of existing shrubs and overburden material be minimized as much as possible. We recommend that an interceptor drain be constructed uphill from the proposed development area to restrict downhill seepage. Such a facility will not only prevent water from flowing into the development area, but it will provide a means whereby water uphill from the proposed development can be readily intercepted and removed from the site.

In constructing roads and houses throughout the area, care should be taken to minimize the disturbance of the existing vegetative cover. We also recommend that adequate lateral support be provided in all areas where the overburden material is under cut.

If the above precautions are taken, it is our opinion that the slopes at this location will remain stable during the development of this site.

3. Foundation Considerations

In accordance with our recommendations outlined above, to minimize the disturbance of the existing materials throughout the development area, we recommend that the structures erected at this site be stepped up the hillside in such a way that the maximum cut at any location does not exceed 10 feet and that all foundations supporting the structures be located on bedrock. The normal cut for the proposed facility would expose bedrock over a portion of the building area; however, piers extending to bedrock may be required at other locations. Allowable soil bearing pressures of 3 to 4 tons would be very conservative for the rock existing at this location.

It is recognized that there may be some areas in which minor structural foundations would be located on the overburden material above the bedrock. In order to provide basic information in which foundations in these areas can be proportioned, bearing capacity recommendations are provided in Table No. 1. In providing the bearing capacity recommendations, it has been assumed that the foundations would be located on the existing slopes and that the depth below the existing ground surface may vary considerably. It is apparent from Table No. 1 that the allowable soil bearing pressures for footings placed on the slope is a function of the width of the footing and the depth at which the footing is placed below the actual ground surface.

In preparing Table No. 1, consideration has also been given to differential settlement. If the proposed facilities are designed in accordance with Table No. 1,

the maximum settlement of any footing will not exceed one inch and differential settlements will not likely exceed one-half inch which should be tolerable for the proposed facility.

4. Excavation Procedures and Lateral Earth Pressures

It is our understanding that Norfolk Avenue will be constructed by widening an existing trail throughout the proposed development area. We recommend that the depth of the cut into the hillside along the roadway alignment be minimized by filling downhill for a portion of the roadway. The natural angle of repose for the quartzite rock to be excavated along the roadway alignment will be approximately 1.5 horizontal to 1 vertical. If the rock excavation is performed in such a manner that the cut and fill is balanced, the dump rock should provide a stable roadway on the downhill side of the cross-section.

Prior to the placement of any rock along the alignment, we recommend that all of the topsoil existing throughout the area be removed to eliminate the possibility of slippage along this plane of weakness. It is also recommenced in placement of the rock fill that it be densified by rolling the material with at least 4 passes of a D8 cat or with 5 to 6 passes with a vibratory roller having a 10-ton vibratory force.

We also recommend that lateral restraint be provided for the overburden material located above the bedrock surface.

As indicated earlier in the report, the bedrock throughout the site is competent rock and will stand at a near vertical slope. It is our understanding that the retaining facility to be used in providing the lateral restraint for the overburden materials will consist of 4 by 6 woodpiles imbedded into the rock on the innerside of the roadway alignment and that lagging will be placed between the wood piles to provide the necessary lateral support.

In designing the proposed earth retaining facility, we recommend that an earth pressure coefficient of 0.4 be used to determine the lateral earth pressures. We also recommend that the bedrock surface be inclined slightly in order to provide a more aesthetical and efficient design for the pile sections. It may be necessary to anchor the pile sections at the top of the pile in order to resist the applied moment. This could be performed in a relatively simple manner by extending a cable from the pile supports to the bedrock in the hillside.

5. The Results of Field and Laboratory Tests

A number of field and laboratory tests have been performed during this investigation to define the characteristics of the subsurface material throughout the

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area. These tests include: in-place unit weight, natural moisture content, Atterberg limits, mechanical analysis, and unconfined compressive strength. A summary of all test data performed during the investigation is presented in Table No. 2, Summary of Test Data. It will be observed that the unconfined compressive strength of the clay layer underlying the granular material in the soil profile varies from 2738 pounds per square foot to 3435 pounds per square foot.

As indicated earlier in the report, the clay is in a relatively stiff condition and is capable of supporting moderate load intensities.

Atterberg limits performed on the clay material indicate that it classified generally as a CL-2 material according to the Unified Soil Classification System. This means that the material has medium plastic characteristics and may possess some slight swell potential if it is permitted to absorb moisture. This material is sufficiently plastic that it should not be used for any kind of backfilling operations behind retaining facilities.

In areas where the natural material will exist adjacent to earth retaining structures, we recommend that it be excavated and replaced with granular material.

The in-place density of the natural granular material is relatively high, and the strength characteristics of this materialare reasonably good.

The conclusions and recommendations presented in this report are based upon the results of the field and laboratory tests which, in our opinion, define the characteristics of the subsurface material in the development area in a reasonable manner. The characteristics of the overburden material, however, uphill from the development area are unknown.

Please advise us if there are any questions relative to the information contained herein.

Yours truly,

ROLLINS, BROWN AND GUNNELL, INC.

Ralph L Rollins

Ralph L. Rollins

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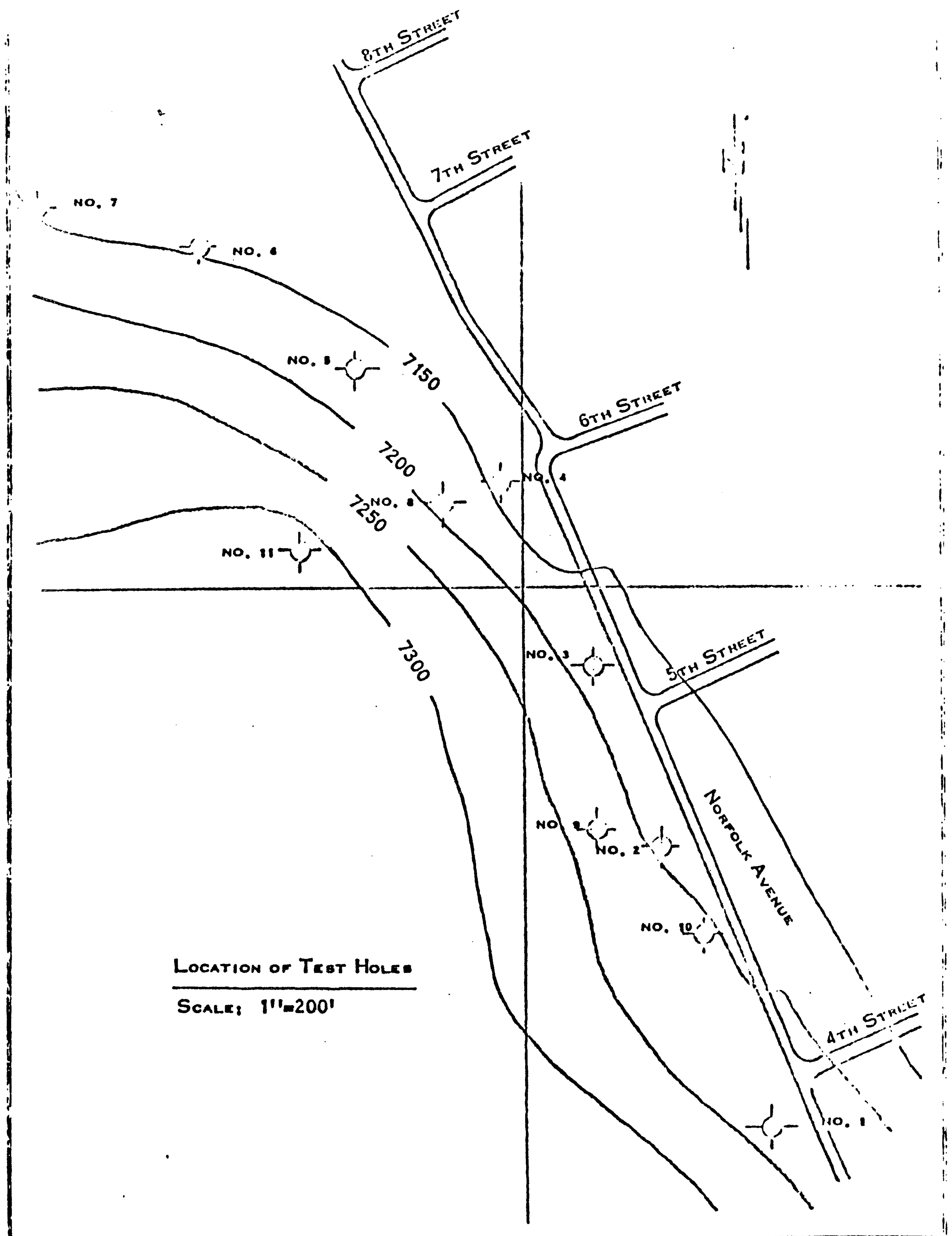
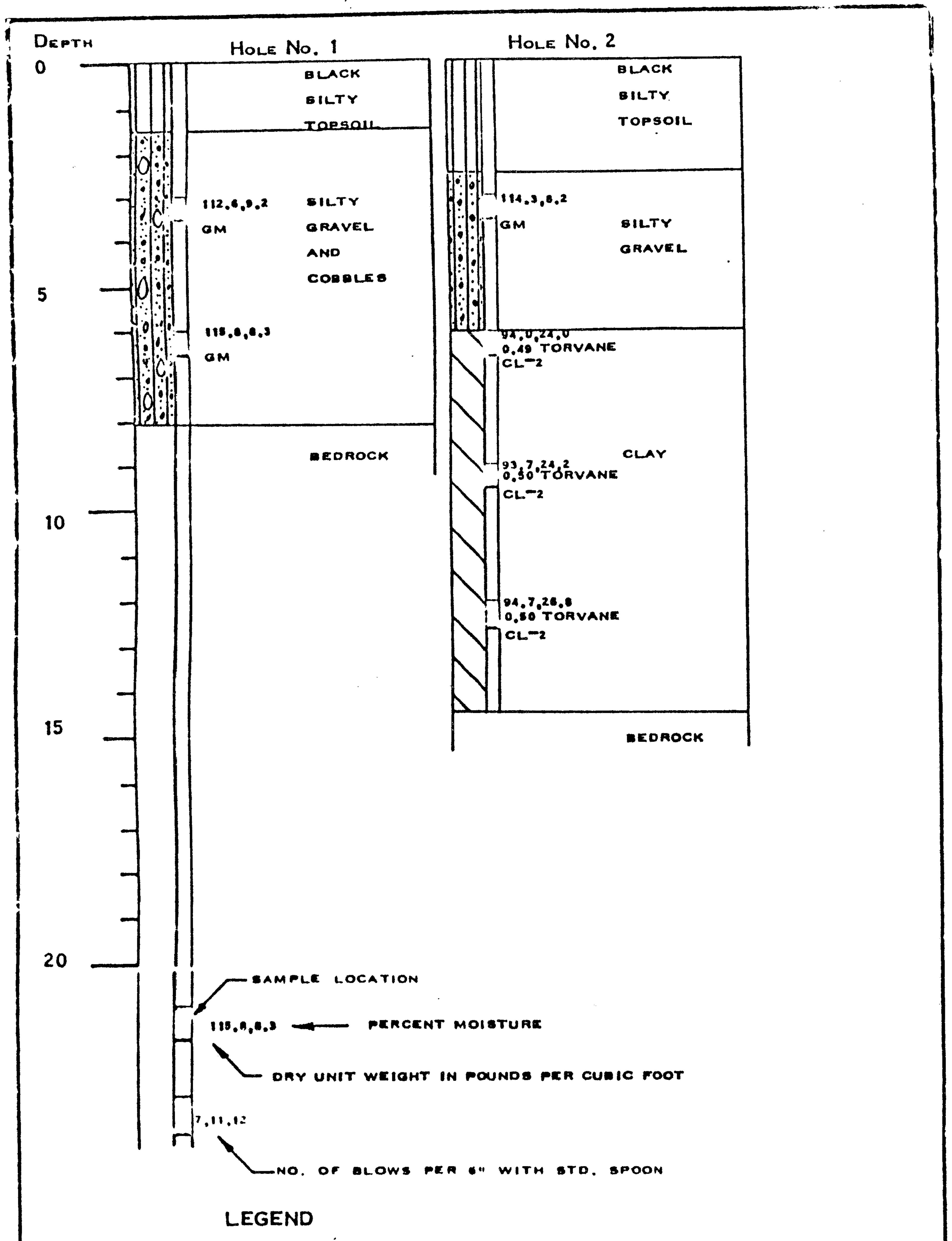
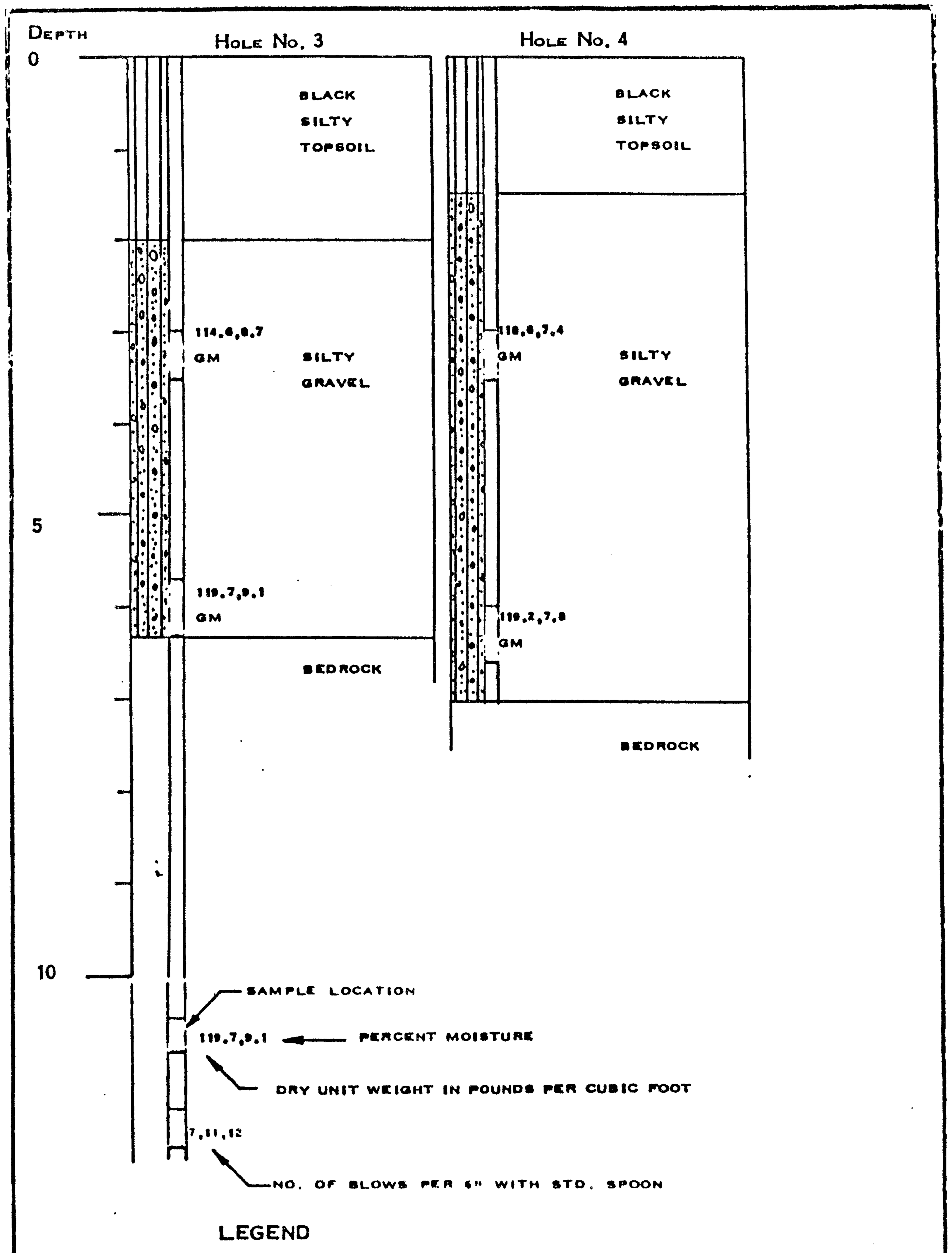
Enclosures



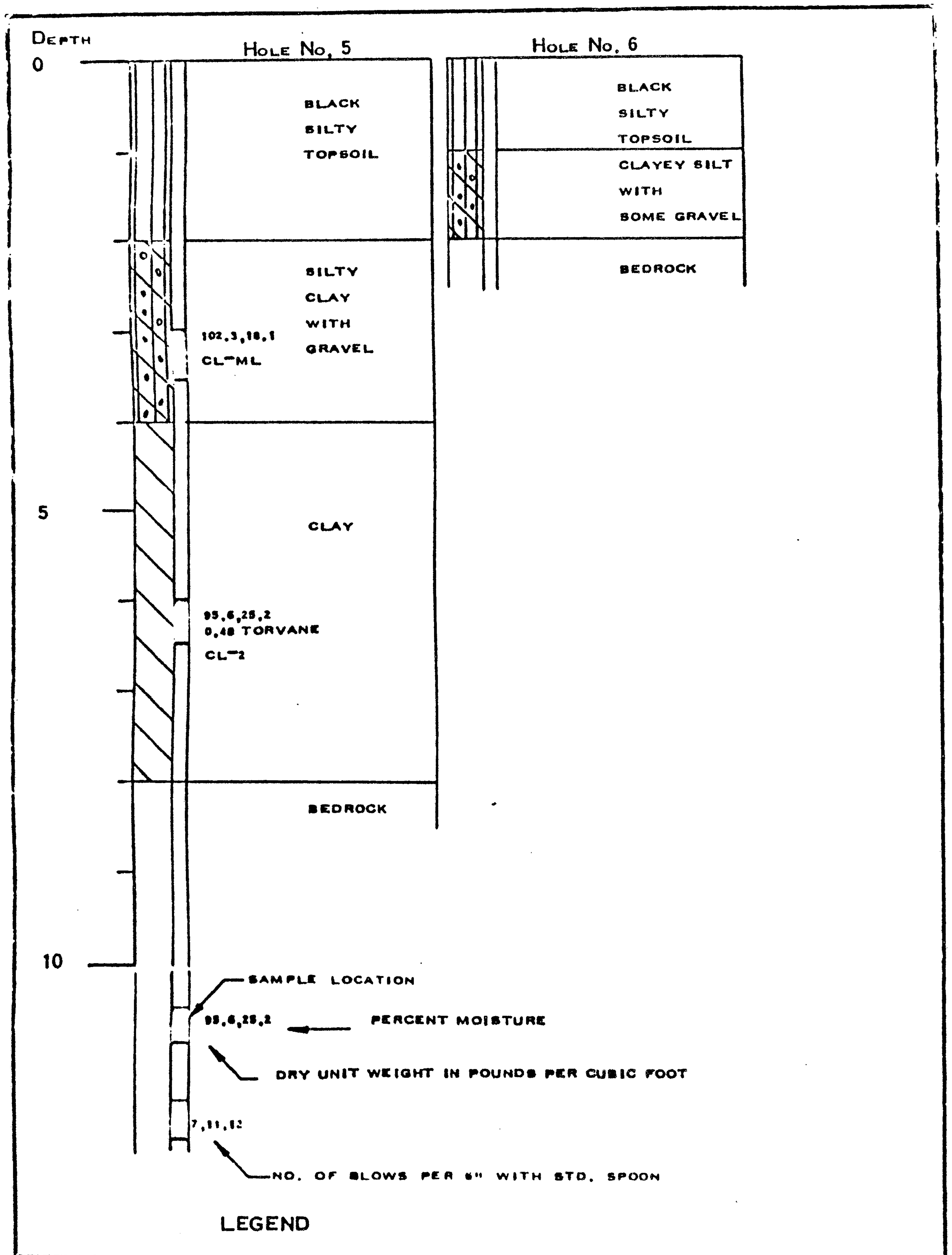
ROLLINS, BROWN & GUNNELL, INC. CONSULTING ENGINEERS	Location of Test Holes For Norfolk Avenue Located In Park City, Utah	Figure No. 1



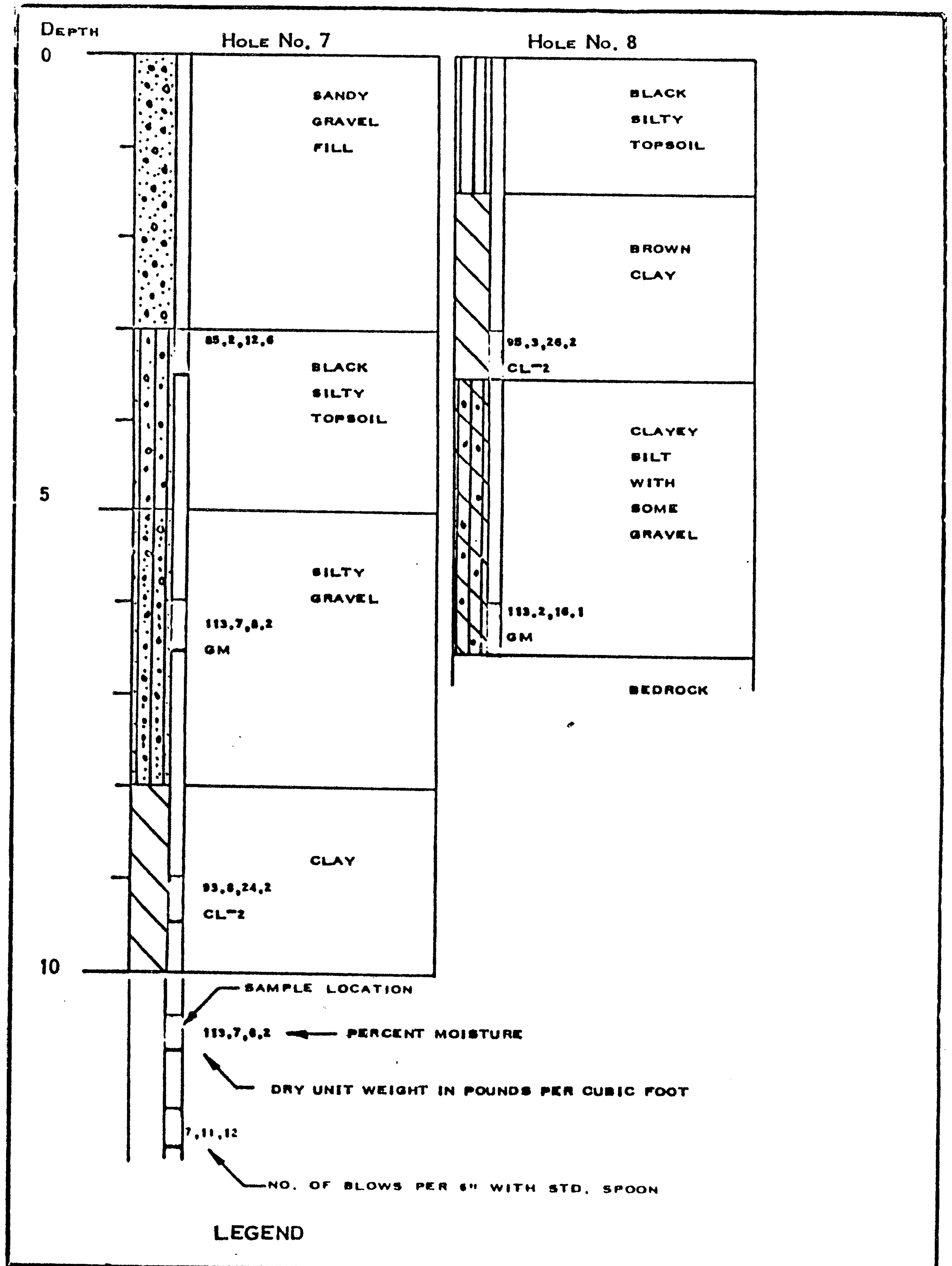
Log of Borings for: Norfolk Avenue in Park City, Utah	Rollins, Brown and Gunnell, Inc. Consulting Engineers	Figure No. 2



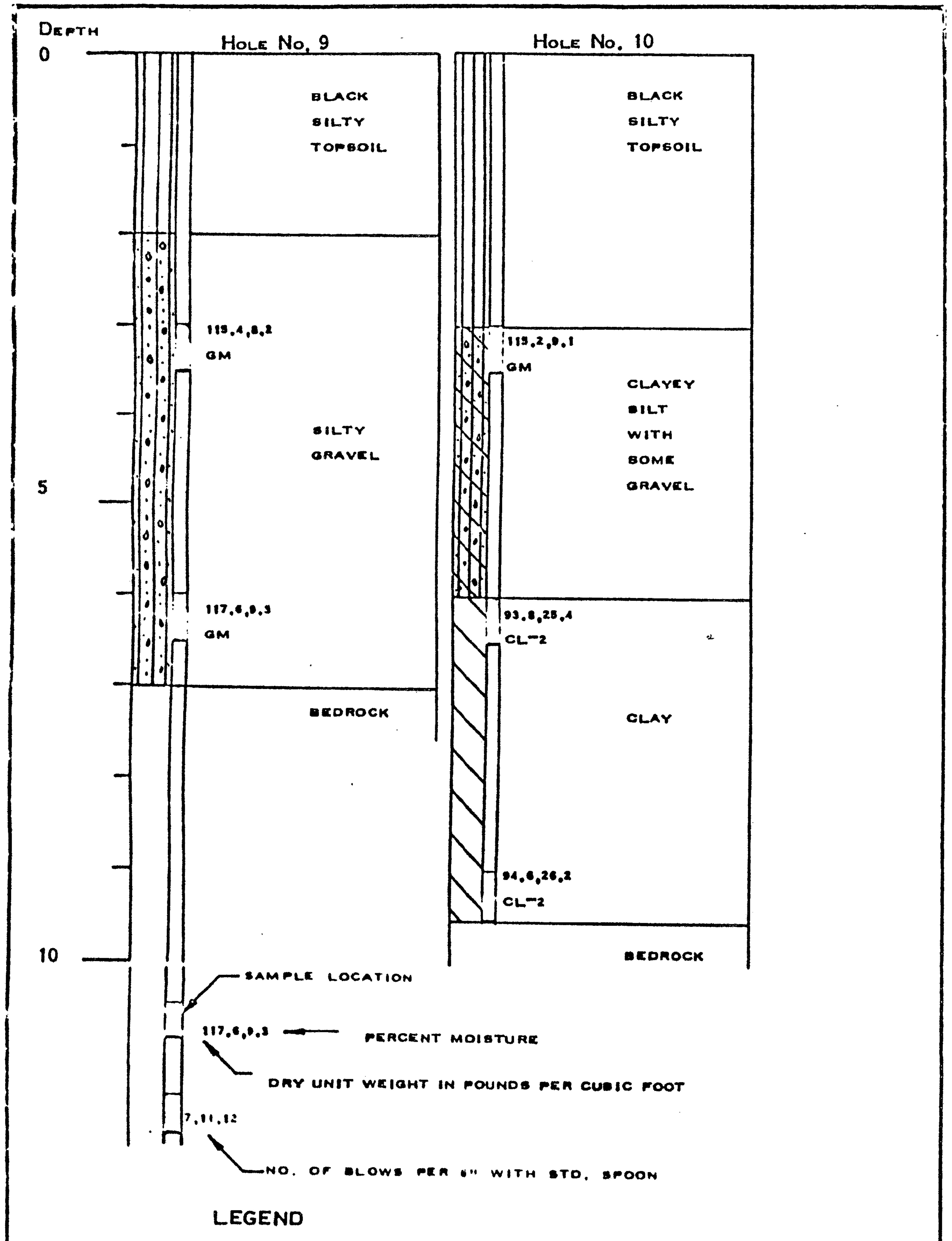
Log of Borings for: Norfolk Avenue in Park City, Utah	Rollins, Brown and Gunnell, Inc. Consulting Engineers	Figure No. 3



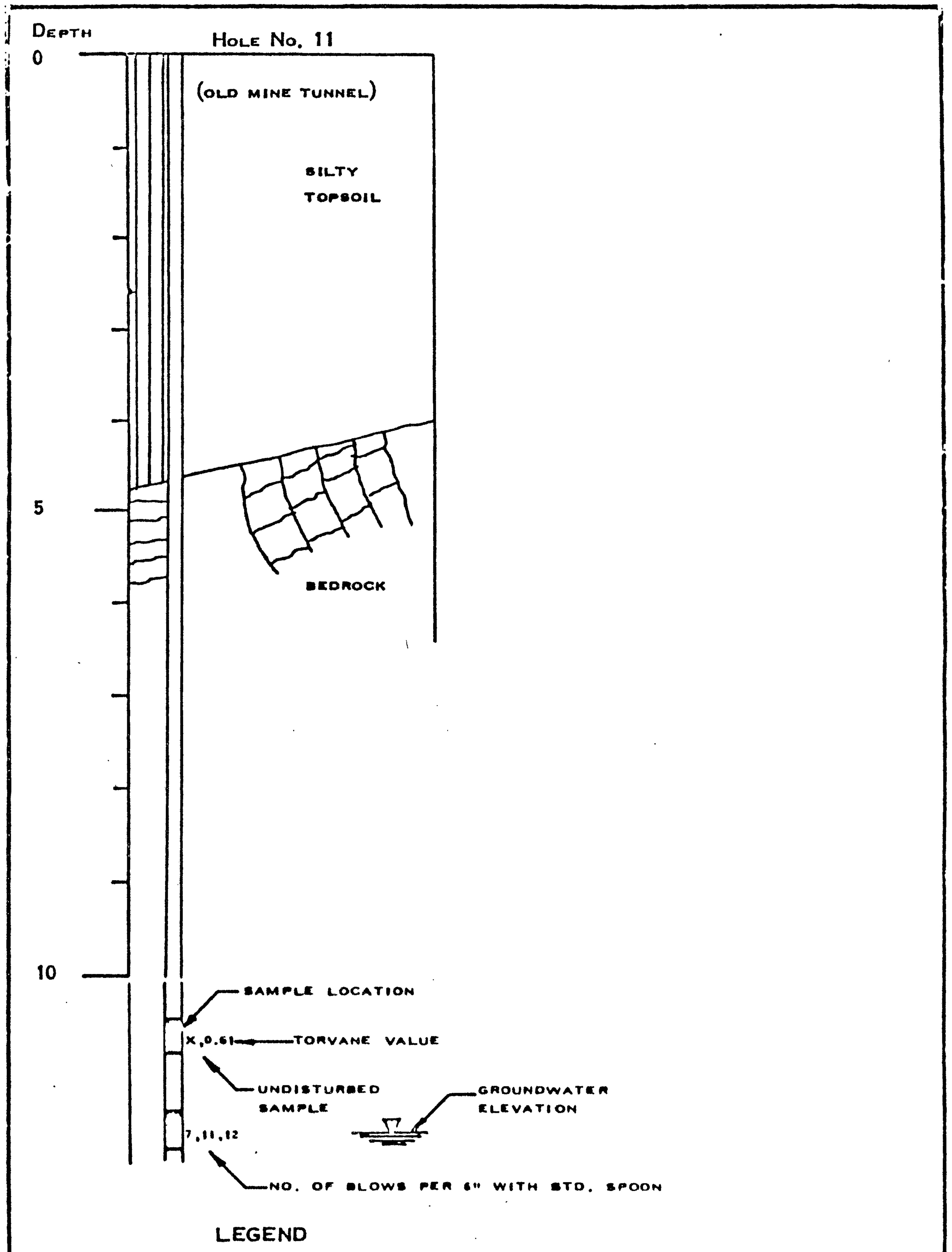
Log of Borings for: Norfolk Avenue in Park City, Utah	ROLLINS, BROWN AND GUNNELL, INC. Consulting Engineers	Figure No. 4



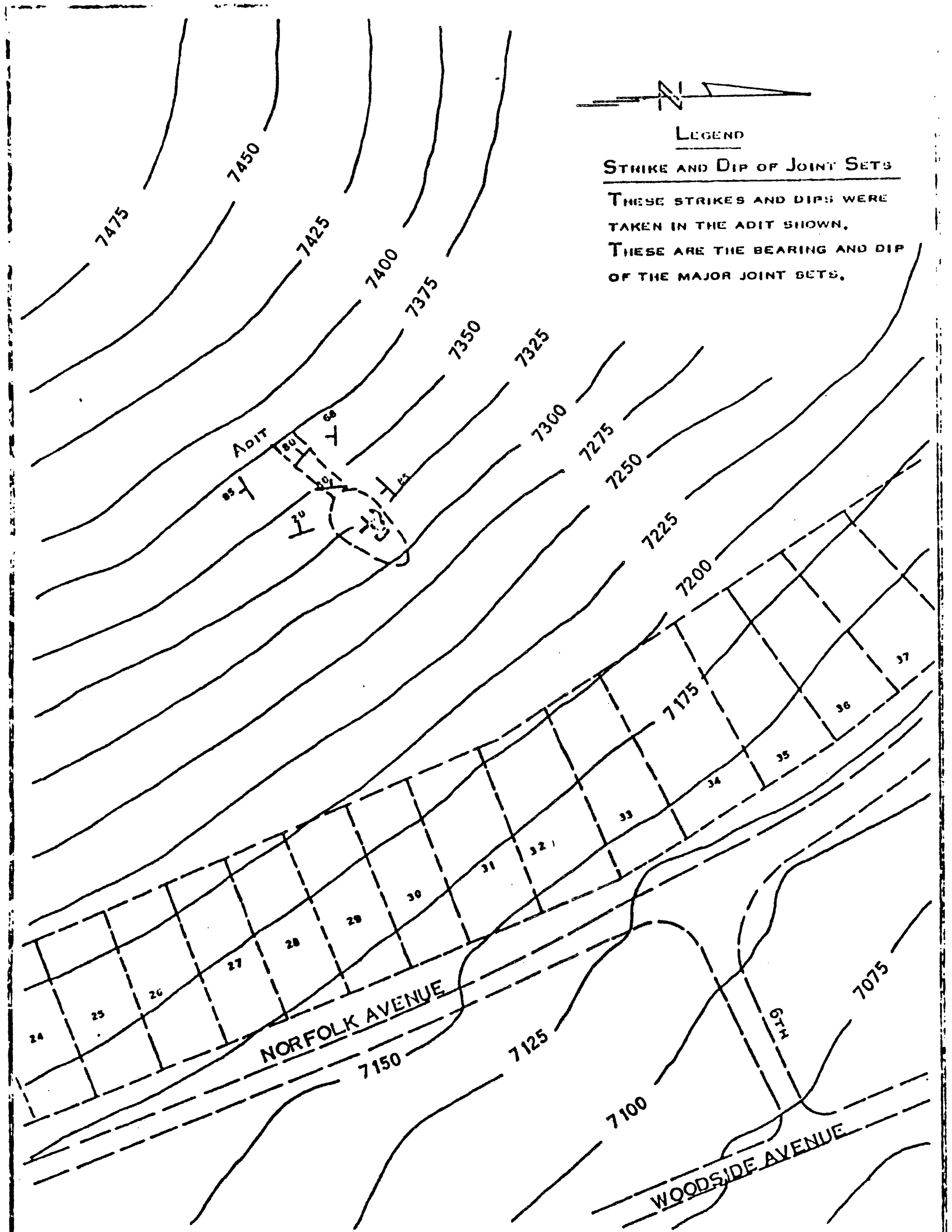
Log of Borings for: Norfolk Avenue in Park City, Utah	ROLLINS, BROWN AND GUNNELL, INC. Consulting Engineers	Figure No. 5



Log of Borings for: Norfolk Avenue In Park City, Utah	ROLLINS, BROWN AND GUNNELL, INC. Consulting Engineers	Figure No. 6



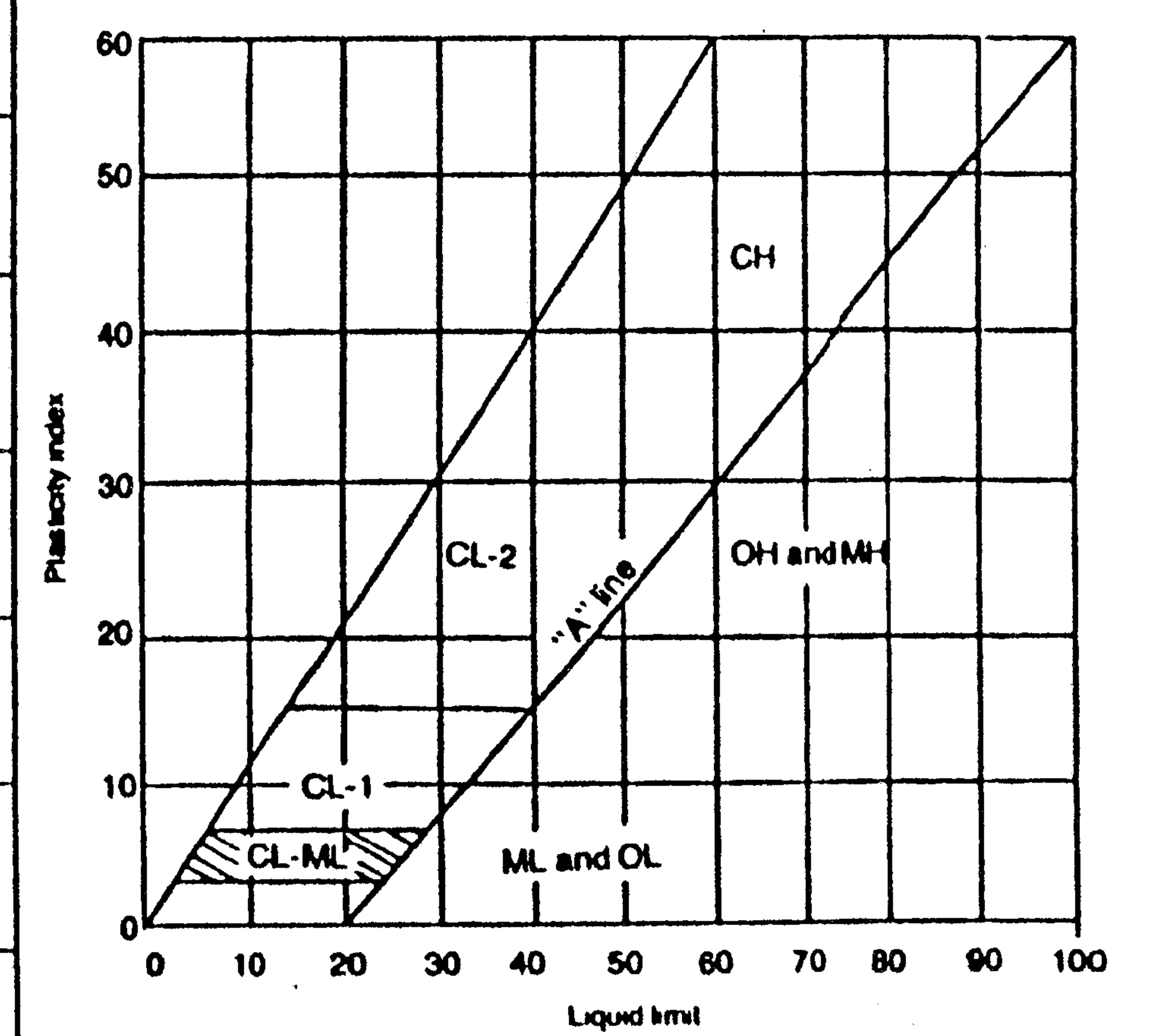
Log of Borings for: Norfolk Avenue In Park City, Utah	ROLLINS, BROWN AND GUNNELL, INC. Consulting Engineers	Figure No. 7



ROLLINS, BROWN & GUNNELL, INC. CONSULTING ENGINEERS	BEARING AND DIP OF JOINT SETS IN WEBER QUARTZIATE ABOVE PROPOSED DEVELOPMENT	FIGURE No. 8

FIGURE NO. 9
Unified Soil Classification System

Major divisions			Group symbols	Typical names	Laboratory classification criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent ..... GW, GP, SW, SP; More than 12 percent ..... GM, GC, SM, SC; 5 to 12 percent ..... Borderline cases requiring dual symbols**	$Cu = \frac{D_{60}}{D_{10}}$ greater than 4, $Cc = \frac{(D_{30})^2}{D_{10} \cdot D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM* d/u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$Cu = \frac{D_{60}}{D_{10}}$ greater than 6, $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM* d/u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual sysbols.
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. less than 7	
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity			
			CL 1/2	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt	Peat and other highly organic soils			



Plasticity Chart

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L. L. is 28 or less and the P.I. is 6 or less, the suffix u used when L. L. is greater than 28.

** Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.

TABLE NO. 1

RECOMMENDED ALLOWABLE SOIL BEARING PRESSURES FOR SPREAD FOOTINGS ON GRANULAR SLOPES (2 HORIZONTAL TO 1 VERTICAL)

B	D	D/B	Q(PSF)
2	0	0	544
2	2	1	1824
3	0	0	820
3	3	1	2736
4	0	0	1094
4	4	1	3648
5	0	0	1368
5	5	1	4460

B = Width of Footing. Applicable for rectangular or strip footings.

D = Depth of Footing Below Ground Surface

Q = Allowable Soil Bearing Pressure

Note: For D/B ratios between 0 and 1 interpolate allowable soil bearing pressures linearly between 0 and 1.

TABLE 2 SUMMARY OF TEST DATA

PROJECT NORFOLK AVENUE FEATURE FOUNDATIONS LOCATION PARK CITY, UTAH

Hole No.	Depth Below Ground Surface	Standard Penetra. Blows per Ft.	In-Place Unit Weight LB/FT^3	In-Place Moisture Percent	In-Place Void Ratio	Unconfined Compressive Strength LB/FT^3	Friction Angle ϕ	Consistency Limits L.L. %	Consistency Limits P.L. %	Consistency Limits P.I. %	Mechanical Analysis % Gravel	Mechanical Analysis % Sand	Mechanical Analysis % Silt & Clay	Soil Classification Unified System
1	3		112.6	9.2							65.0	8.7	26.3	GM
	6		115.8	8.3							58.8	7.0	34.2	GM
2	3		114.3	8.2							82.3	7.0	10.7	GM
	6		94.0	24.0		3435		40.1	19.8	20.3				CL-2
	9		93.7	24.2		3153		40.4	19.3	21.1				CL-2
	12		94.7	26.8		2987		38.5	17.8	20.7				CL-2
3	3		114.6	8.7							64.0	9.5	26.5	GM
	5.5		119.7	9.1							60.0	10.5	29.5	GM
4	3		118.6	7.4							66.2	7.8	26.0	GM
	6		119.2	7.8							62.2	10.8	27.0	GM
5	3		102.3	18.1				23.2	16.8	6.4				CL-ML
	6		95.6	25.2		2997		38.7	17.8	20.9				CL-2

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PROFESSIONAL ENGINEERS

TABLE 2 SUMMARY OF TEST DATA

PROJECT NORFOLK AVENUE FEATURE FOUNDATIONS LOCATION PARK CITY, UTAH

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRA. BLOWS PER FT.	IN-PLACE UNIT WEIGHT LB/FT³			UNCONFINED COMPRESSIVE STRENGTH LB/FT³	FRICTION ANGLE φ	CONSISTENCY LIMITS L.L. %			MECHANICAL ANALYSIS % GRAVEL			SOIL CLASSIFICATION UNIFIED SYSTEM
			UNIT WEIGHT LB/FT³	MOISTURE PERCENT	VOID RATIO			L.L. %	P.L. %	P.I. %	% GRAVEL	% SAND	% SILT & CLAY	
7	3		85.2	12.6				20.3	15.7	4.6				ML
	6		113.7	8.2							58.6	13.9	27.5	GM
	9		93.8	24.2		2919		37.9	17.6	20.3				CL-2
8	3		95.3	26.2		3299		35.6	19.2	16.4				CL-2
	6		113.2	16.1							54.6	10.4	35.0	GM
9	3		115.4	8.2							62.1	11.1	26.8	GM
	6		117.6	9.3							46.2	31.4	22.4	GM
10	3		115.2	9.1							44.8	31.2	24.0	GM
	6		93.8	25.4		2637		37.5	19.4	18.1				CL-2
	9		94.6	26.2		2738		39.6	19.0	20.6				CL-2

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WILLIAM LUND

May 1979

Urban and Engineering Geology Section
Utah Geological and Mineral Survey
Salt Lake City, Utah 84108

PRELIMINARY ENGINEERING GEOLOGIC REPORT TO PARK CITY ON THE PROPOSED QUITTIN TIME DEVELOPMENT

by
William Lund, Geologist

Done at the request of the Park City planner.
May 1979

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PRELIMINARY ENGINEERING GEOLOGIC REPORT TO PARK CITY ON THE PROPOSED QUITTIN TIME DEVELOPMENT

INTRODUCTION

This report presents the results of a geologic reconnaissance of the proposed Quittin Time residential and recreational complex located in Park City, Utah. This is to be a hillside development which includes both single family dwellings and condominiums. A ski run and other associated recreational facilities are also planned. The purpose of this reconnaissance was to determine what impact the geologic and hydrologic conditions of the site might have on the proposed development. This study was performed at the request of Mr. David Preece, Park City Planner.

SITE CONDITIONS

Location and Physiography

The proposed development encomposses about 352 acres of ground located on the west side of Park City southwest of Woodside Avenue (Figure 1). This is an area characterized by steep slopes and broad, shallow drainages. Elevations across the site range from about 7110 feet on the east edge of the property to an estimated 7600 feet on the west edge. Vegetative cover is moderate to thick and consists of buckbrush at the lower elevations and evergreens further upslope. There has been no previous residential development on the property, but two municipal waterlines and an abandoned aerial tramway cross the site, and numerous mineral prospects and old mine tunnels dot

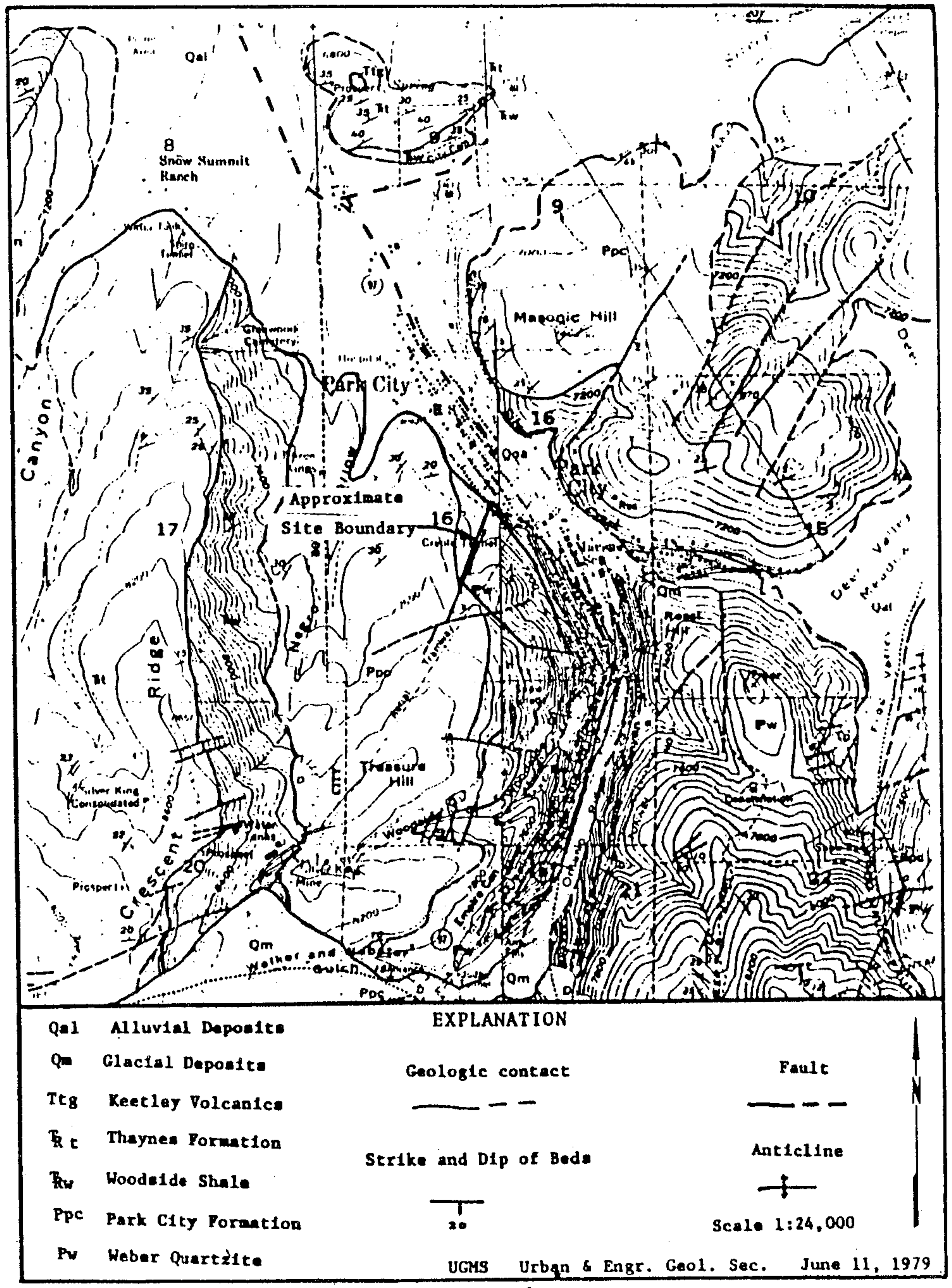

Figure 1 General Location and Geologic Map

the hillside.

Geology and Soils

Lying as it does near the intersection of two major structural lineaments, the Wasatch Front and the Uinta Mountains, the geology of the Park City District has undergone a long and complex history. The major structural features and geologic units in the vicinity of the proposed development are summarized in Figure 1. The ridge upon which the development would be built is underlain by the Weber Quartzite, a pale gray and tan formation of quartzite and limey sandstone with interbedded horizons of limestone and dolomite. The major structural feature in the area is the Park City anticline which lies southeast of the site (Figure 1). Two faults have been mapped by Bromfield and Crittenden (1971) as extending onto the site from the west; however, during the field investigation no surface evidence of these or any other faults was observed.

Bedrock exposures on site are limited to one small, highly weathered, outcrop high on the hillside along the proposed ski run, and to rock exposed in old mine workings. At these localities the quartzite was observed to be hard and durable but fractured and containing numerous, well-developed joint sets. Due to the limited extent of the exposures and to the numerous joints present it was not possible to obtain a reliable strike and dip on a bedding surface, but Bromfield and Crittenden show the quartzite in adjacent areas to be striking to the northeast and dipping 10 to 20 degrees to the northwest. The following table lists the joints observed at the surface outcrop and also

those measured during the site reconnaissance inside a mine tunnel located on the property (Figure 2).

Strike	Dip	Spacing	Fillings of Coatings	Class	Location
N10E	43NW	2-3'	none	major	surface outcrop
N85W	Vert	2-3'	none	major	surface outcrop
N22E	Vert	2-3'	none	major	surface outcrop
N47E	80SE	3"-1'	iron stain	major	mine adit
N52E to N65E	80NW	3"-1'	none	major	mine adit
N10W	83SW	1'-5'	none	*	mine adit
N80W	73NE	1'-5'	iron stain	*	mine adit
N20E to N40E	80SE to Vert	1"-6"	none	minor	mine adit
N50W	Vert	6"-1'	none	minor	mine adit
N-S	21W	1'-5'	none	**	mine adit

* Due to limited size of outcrop and width of joint spacing unable to determine if this is a major or minor joint set.
**Possibly a bedding plane

During this investigation nine of ten backhoe pits excavated by a private consulting firm which had previously prepared a report on this property were examined (Figure 2). Prior to the field reconnaissance, four of these test holes were cleaned out by the Park City backhoe. The five remaining holes were not cleaned, either because they were inaccessible due to installation of a new municipal waterline across the site, or because they could not be located by the equipment operator. The four test holes which were cleaned, nos. 1, 4, 5, and 10 of the consultant's

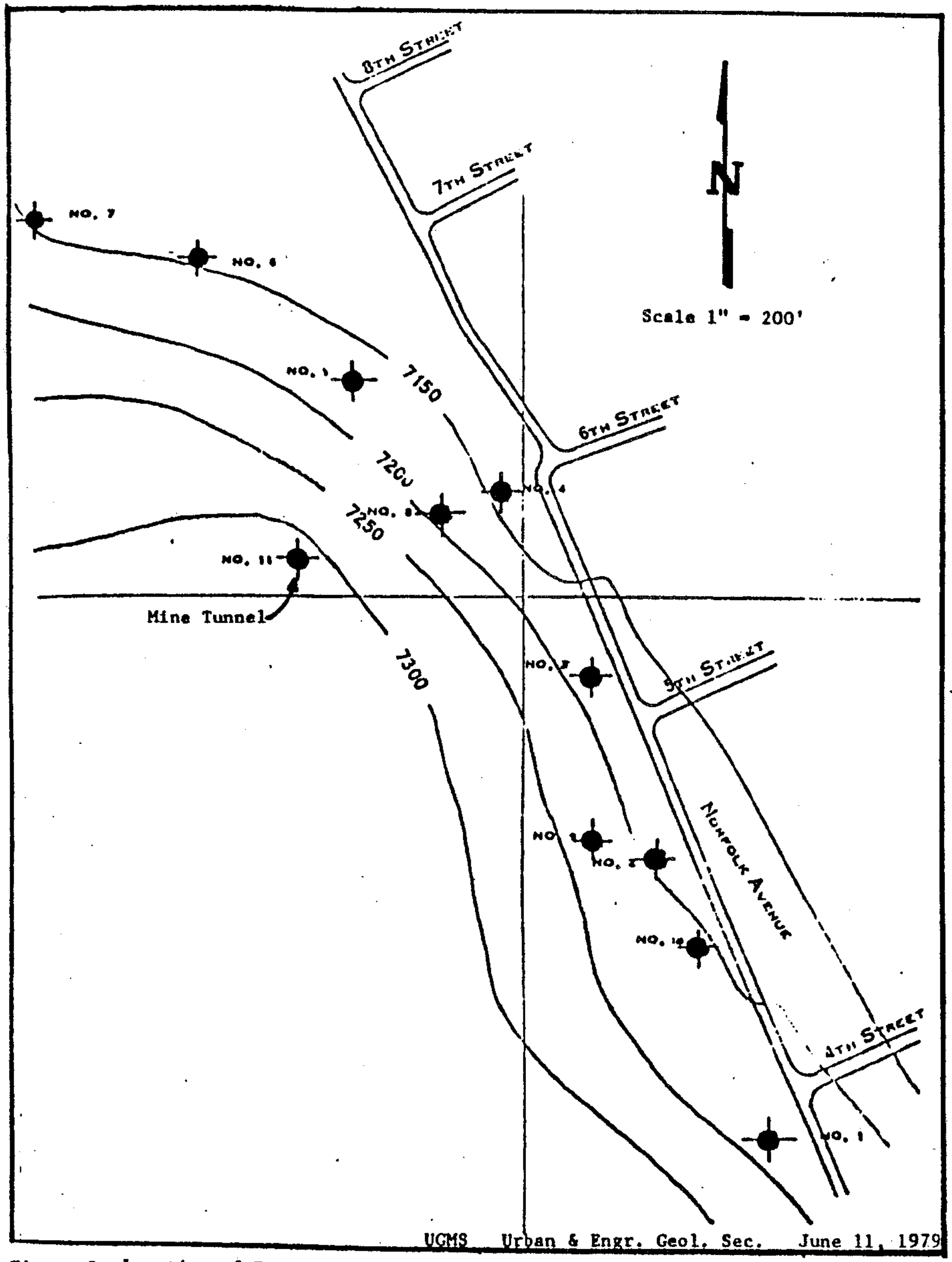

Figure 2 Location of Test Holes (Modified from consultants report)

report, were all reported to have reached bedrock at depths ranging from seven to nine feet below the ground surface. Groundwater was encountered at a depth of eight feet in test hole no.10, and this prevented the excavation from being adequately cleaned. As a result no determination could be made regarding the presence of bedrock. Test holes 1 and 4 were both cleaned to their original depths, but an inspection showed that neither of the two had reached solid, in place rock. Instead, both excavations stopped at a dense, closely packed layer of quartzite cobbles and boulders in a clay matrix. This material appears to be sufficiently compact to resist excavation by all but the largest of backhoes, and may represent the zone of broken and weathered material that commonly mantles in place bedrock. However, since the excavations did not penetrate this horizon, its true thickness and relationship to the underlying bedrock is not known. Test Hole No. 5 was excavated one foot below original grade and bedrock was not encountered. The walls of the five remaining test holes had all sluffed to some degree, consequently, bedrock could be positively identified only in test hole no. 6. The tenth test hole, no. 3 of the consultant's report, had been backfilled and could not be located.

The soils exposed in the test holes generally conform to the descriptions provided in the consultant's report (Apprendix A). The only significant difference concerns the description of the Zone 2 soils. This soil horizon is described in the consultant's report as a granular zone composed of angular fragments in a silt matrix, and is classified in their logs in accordance with the

United Soil Classification System as a silty gravel. Such a description implies that the material is non- to only slightly-cohesive and possesses no or only very low plasticity. The soils which correspond to Zone No. 2 observed in the test holes were found to contain a considerable amount of clay and as a group are probably better classified as a clayey gravel and in some localities even a gravelly clay.

Hydrology

The hillside upon which the proposed site is located receives between 25 and 30 inches of precipitation annually (Baker, 1969). Despite the relatively generous amount of moisture available, a near surface groundwater table exists beneath the site for only a short period of time each year, if it is present at all. The reason for this is the result of a combination of factors which include the manner in which the precipitation occurs at the site, the permeability of the clay-rich soils, and the steep mountain slopes. The majority of precipitation which falls on the site each year accumulates as a thick snow-pack during the winter months. In the spring, the snow melts quickly and releases a large quantity of water to the environment. A portion of this melt water infiltrates into the soil while the remainder flows downslope as surface runoff. The amount of water which soil can absorb is dependent upon its' permeability and the rate at which the water is made available to it. The clayey soils beneath the proposed development have moderate to low permeabilities. Therefore, during periods of warm temperatures and rapid snow melt near surface soils quickly become saturated and can accept no more water. This results in a marked increase in the amount of water which takes the

form of surface runoff. During a cold spring the snow melt proceeds more slowly and the soil has more time to accept the water made available to it. Regardless of whether the melt-water runs off across the surface of the ground or infiltrates into the soil it is immediately acted upon by gravity and moves rapidly downslope. In a normal year the amount and duration of the surface runoff closely parallels the rate at which the snow pack melts and is usually complete by mid- to late-spring. The downslope movement of the water which infiltrated the soil is slower, but it also travels relatively quickly so that by midsummer the soils have drained and there is no near surface groundwater remaining.

The Weber Quartzite which underlies the site is recognized as a major water producing formation in the mines surrounding Park City, however, it should be remembered that these mines drain many square miles of rock. An existing mine tunnel (Figure 2) on the property which has been advanced approximately 60 feet into the Weber Quartzite was found to be dry in mid-May.

Seismicity

Park City is located along the southern portion of the Intermountain Seismic Belt, a north trending zone of earthquakes extending from the Montana-Canada border to Arizona, and historically the second most active seismic area in the continental United States. In Utah earthquake activity associated with the ISB occurs along a complex series of steeply dipping faults having a generally north-south trend. The Wasatch Fault, which at its closest point lies about 16 miles due west of Park City, is one of the largest and most seismically active of these faults.

Although many faults have been recognized in the Park City Mining District none are known to show evidence of recent activity. A compilation of earthquake epicenters, prepared by the University of Utah Seismograph Station, covering the period from 1962 to 1978 lists a total of 22 earthquakes with magntiudes of 1.5 or greater occurring within a 13 mile radius of Park City (Figure 3). The largest of these was the Heber Valley earthquake which occurred in October of 1972 with a magnitude of 4.2. The other 22 events all had magnitudes of 3.9 or less.

ENGINEERING GEOLOGIC CONSIDERATIONS

As a part of this study, a review was made of a geotechnical report previously prepared on this property by a private consulting firm. While overall a good report, the results of our own field investigation are at odds with certain of the consultant's findings. These differences are pointed out in the text. In addition, some other geologic and hydrologic aspects of this site which were not covered in the consultant's report are discussed here.

Foundation Considerations

As previously mentioned in this report (page 6) the granular materials grouped together by the consultant as Zone 2 soils and identified as silty gravels were found to contain a considerably higher percentage of clay than is normally associated with a silty soil. For this reason, it is felt that they are better classified as clayey gravels and locally as gravelly clays. Clay bearing soils may possess a considerable shrink-swell capacity which is primarily related to their ability to adsorb or release water. In addition, many soils are susceptible to compaction and differential settlement with loading. For these reasons, it is recommended that for any

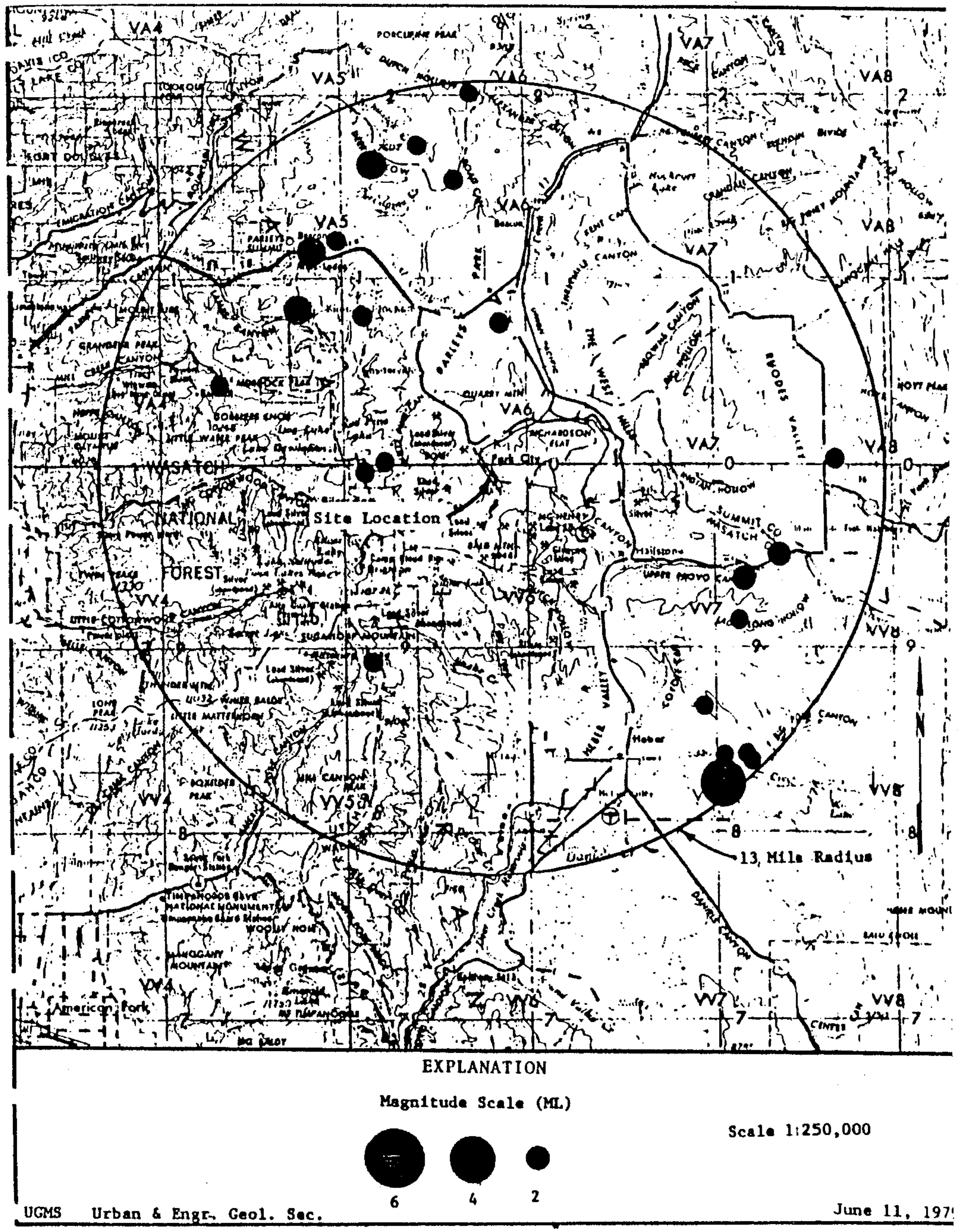

Figure 3 Location of Earthquake Epicenters, 1962 to 1978

structure which will be supported in whole or in a substantial part by a Zone 2 or Zone 3 soil (Appendix A) additional tests be performed to determine the engineering characteristics of the particular soil horizons involved, and that all foundations or retaining walls be designed accordingly. The consultant's report recommends that all structures in the development be founded on bedrock, thereby avoiding a number of foundation and slope stability problems. Based upon our inspection of the consultant's test holes and a comparison with their test hole logs it appears that in at least two instances closely packed quartzite cobbles and boulders were mistakenly indentified as bedrock. Care should be exercised during construction to insure that foundations designed to rest on solid inplace rock do so, and that the zone of broken and weathered material which commonly mantles bedrock is completely removed before the foundations are laid. Observations made at the entrances to several adits and tunnels on the property indicate that this weathered zone is from two to about six feet thick. The depth of the excavations required to reach bedrock can be expected to vary across the site; ranging from only a few feet on the steeper slopes and the high ground between drainages to greater than ten feet along stream channels and on gentle slopes.

In a hillside development of this type, numerous fills will be required both to prepare construction pads and roadbeds, and to backfill behind retaining walls. To prevent excessive settlement and failure of these fill sections it is recommended that a code of minimum construction specifications be adopted which

clearly outline the acceptable gradation limits and compaction requirements for all catagories of fill material. In this regard, it should be noted that the crushed quartzite found in the numerous small mine waste dumps on site would make a very good source of granular, nonplastic fill material. This would seem to be an excellent use for this material since the dumps are too small to provide a foundation for a house or condominium and would probably be considered unsightly in this type of development.

Slope Stability

Natural slopes on site are steep, averaging between 44 and 57 percent gradient (20 to 26 degrees), but appear to be stable under the existing conditions of landuse and vegetative cover. No indications of landsliding or slumping were observed, but there was considerable evidence to indicate that soil creep is occurring. Soil creep is the slow, nearly continuous movement of soil and broken rock downslope under the influence of gravity. It is manifested by the tipping of fence posts and similar rigid objects embedded in the ground. One of the best indicators that creep is occurring is the gentle curving of the base of trees with the convex side pointed downhill in the direction of movement. Generally, creep is confined to the upper 10 to 15 feet of the soil or broken rock mass, and is most rapid close to the ground surface. Soil creep should be considered an indicator of possible problems since it represents a quasi-equilibrium state that can be upset and turned into a much more serious slope failure by unwise construction practices. Ample evidence of this can be seen just to the north of the proposed development in an area of new construction above Lowell Avenue where over-steep cuts in unconsolidated materials are undergoing extensive sluffing and where at least one landslide/mudflow is reported to have occurred (David Preece, oral communication).

Usually, soil creep cannot be stopped, but its rate of movement can be decreased by providing ample drainage, thereby increasing soil strength and preventing periodic swelling and shrinking of the soil mass. To help insure post construction slope stability of the unconsolidated materials on site it is recommended that cut and fill slopes be designed in accordance with the recommendations of a qualified soils engineer following a detailed stability analysis of the materials involved.

The stability of a bedrock cut is highly dependent upon the orientation of any bedding planes or joints which may be present in the rock mass. Obviously, the critical relationship is one in which a joint or bedding plane strikes in a direction parallel to the cut and dips toward the open slope face. When such a situation exists, blocks of rock, the size of which are determined by the spacing of the joints, can become detached and slide or fall, producing a hazard to both buildings and people. A somewhat less critical situation occurs when joints or bedding are present, but with orientations different from those described above. In such cases there is a tendency for the slope to ravel and produce some fallout of blocks. Rock fall problems can be reduced by establishing slope angles which do not allow potentially troublesome joints or bedding planes to daylinght.

Numerous joints with various orientations (table, page 4) were measured in the bedrock at the site. Again the findings of our field reconnaissance disagree with the consultant's report, in that a joint was found that strikes more or less parallel to the hillside and dips toward the valley (N80W, 73NE). This orientation was measured in the wall of the mine tunnel (Figure 2). The joint was not strongly developed, and the spacing was such that it was difficult to determine from such a small outcrop whether or not it represents a major set of discontinuities in the rock mass. If it does, serious rockfall problems could develop in any steep bedrock cuts which parallel the mountain face. For this reason, and because the orientation of other bedrock cuts made during construction may daylight some of the remaining joint sets, it is recommended that as construction proceeds all rock cuts be inspected by a qualified engineering geologist, and that based upon his recommendations slope designs be modified as necessary to prevent daylighting of joints or bedding.

The material comprising the mine dumps on site is at or near its angle of repose. For that reason, during construction care should be taken not to undercut any dump slopes. If the slopes are undercut they could fail rapidly and at best would probably provide an almost continuous maintenance problem with raveling slopes.

A short term slope stability problem which deserves consideration here is the hazard to the homes along Woodside Avenue from rocks which become dislodged by construction activities and roll downhill. A system should be devised to catch and stop these rocks before they can cause any property damage or injure anyone.

Site Drainage

Some of the most severe problems associated with hillside developments are related to water. This is nowhere more evident than in Park City where each Spring the homes built on the surrounding hillsides suffer from erosion, sedimentation, localized flooding and water related slope stability problems. Due to the steepness of the slopes upon which it would be built, the proposed development would also be susceptible to such hazards. The number and severity of these problems can be reduced by installation of an adequate site drainage system. Such a drainage system is necessary not only to prevent problems in the new development, but also to protect the homes already in existence along Woodside Avenue from the increased runoff that can be expected to result from construction upslope.

It is recommended that interceptor drains be established both above and below the development, and that site grading be accomplished in such a manner that all surface runoff is collected and funnelled to those drains. In addition, the existing vegetation should be left undisburbed whenever possible and septic tanks are definitely not recommended.

Avalanches

Whenever a hillside is being considered for development at the higher elevations in the Wasatch Mountains, its potential for avalanche hazard must be evaluated. At least one destructive avalanche is known to have occurred on the hillsdie where the proposed development would be built. It is reported to have destroyed a large shed and damaged a house on Woodside Avenue

about 1910 or 1911 (Mrs. Bea Kunner, oral communcation). Photographs dating from the same era show that most of the vegetation on the hillside had been cut down to fire the old steam driven hoists and pumps in the surrounding mines. There has not been a large avalanche on the hillside for at least 40 years (Mr. Mel Flecher, oral communication), a period of time that more or less coincides with the reestablishment of vegetation on the slope. Since slopes with gradients steeper than 35% (approximating 16 degrees) can generate avalanches it must be assumed that if large areas of the hillside are again stripped of their vegetative cover avalanches could occur.

It is recommended that a map be prepared by the developer which shows the areas from which the vegetation will be removed. A comparison can then be made with a topographic map to determine if an avalanche hazard would be created; if it is, appropriate control methods should be implemented.

Ground Subsidence

Ground subsidence is not normally associated with a site where bedrock lies as close to the surface as it does at this one. However, the extent of past mining activity in the area raises the possibility of ground collapse over old mine workings. A number of the old prospects and tunnels observed on site during the reconnaissance have caved or collapsed near their entrances, and around others a small circular zone of subsidence has developed. No structures of any type should be built over or directly adjacent to caved, collapsed, or subsided ground nor should heavy structures be permitted directly upslope from shallow mine workings until it can be proven that no danger from ground collapse exists.

Regardless of whether or not construction activity occurs on or near old mine workings, they all should be located and sealed to protect the residents of the property from injury.

Seismic Response

The absence of active faults in close proximity to Park City means that seismic response in the area would most probably be limited to some degree of ground shaking and possible ground failure associated with a large seismic event located along the Wasatch Fault. The intensity and duration of the shaking would depend upon the location of the epicenter and the magnitude of the event. The shallow depth to bedrock at the site would act in its favor, since during an earthquake seismic effects are usually somewhat less severe at bedrock localities. However, the steep slopes upon which the development would be built represent a negative factor in terms of site safety. During strong ground shaking such slopes would be susceptible to both landslides and rock fall. If a seismic event were to occur in the winter months during a period of deep snow pack, avalanches could result.

Park City has experienced a remarkably low level of seismic activity, at least in the 100 years or so since the area has been settled. Nevertheless, because of the town's location relative to a number of active earthquake faults it lies in an area classified as Seismic Zone 3 by the Uniform Building Code, and all structures should be designed accordingly.

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Based upon the results of our field investigation, a review of the published literature pertaining to the site, and the consultant's report, the following conclusions and recommendations

are made.

Conclusions

1. Zone 2 soils should be reclassified as clayey gravels and locally as gravelly clays to reflect their cohesive nature and high clay content.

2. A dense layer of quartzite cobbles and boulders in a clay matrix exposed at the bottom of test holes 1 and 4 appears to have been erroneously identified in the consultant's report as bedrock. Bedrock could be positively identified in only one of the nine test holes examined, but five of the pits had not been adequately cleared and therefore a determination as to whether bedrock was present or not couldn't be made. Bedrock was also reported in test hole No. 5, however, the excavation was cleaned a foot below original grade and no sign of any rock was observed (see note test hole No. 5 in Appendix A)

3. A joint orientation was measured in the bedrock which strikes more or less parallel to the hillside and dips toward the valley. Due to the limited size of the exposure no determination could be made concerning the continuity or size of this joint set. However, if it is well developed across the site slope stability problems could develop in rock cuts.

4. A number of other potential geologic hazards have been identified at this site. The extent to which they will prove to be a problem depends in large measure on the degree to which they are recognized and compensated for in the developments design. The list of potential geologic hazards includes:

   a. Foundation and backfill problems associated with clayey soils.

b. Slope stability problems in the unconsolidated materials on site due to the steep hillside on which the development would be built.

c. Potential for property damage and personal injury resulting from rocks rolling down slope during construction.

d. Erosion, sedimentation, and localized flooding during the Spring snow melt.

e. Avalanche hazard, especially if vegetative cover is removed from large areas of the hillside.

f. Ground subsidence and collapse over shallow mine workings.

g. Site sensitivity to landslide, rockfall, and avalanche hazard in the event of a large earthquake along the Wasatch Fault.

Recommendations

1. Foundations of structures to be supported in whole or in a substantial part by Zone 2 and Zone 3 soils should be designed on the basis of the engineering parameters determined for the particular soil horizons involved by laboratory testing.

2. Care should be exercised during construction to insure that those foundations designed to rest on bedrock actually do so, and that the mantle of broken and weathered material lying just above bedrock is completely removed before the foundation is laid.

3. If not already in existence a code of minimum construction standards should be adopted which clearly outlines the acceptable gradation limits and compaction requirements for various catagories of backfill.

4. Cut and fill slopes in unconsolidated materials should be designed by a qualified soils engineer on the basis of detailed stability analyses.

5. As construction proceeds all rock cuts should be inspected by a qualified engineering geologist and based upon his recommendations the cuts should be modified as necessary to prevent daylighting of joints and bedding.

6. Homes located along Woodside Avenue should be protected from rolling and falling rock dislodged by construction activity.

7. Interceptor drains should be installed both above and below the development and site grading should be accomplished in such a manner that all surface runoff is collected and channelled to the drains.

8. A map should be prepared by the developer showing those areas of the site where vegetation will be removed. If, upon comparison of that map with a topographic map it is found that an avalanche hazard will be created appropriate control measures should be taken.

9. Structures should not be built over or adjacent to caved, collapsed or subsided ground, and heavy structures should not be permitted directly upslope of shallow mine workings until it can be proven that no danger from ground collapse exists.

10. All old mine tunnels, shafts, or adits on site should be located and permanently sealed to prevent injury to residents of the development.

11. Numerous small mine dumps exist on site, of these only the old Creole dump appears to be of sufficient size to support a large building. Due to the potential for creating unstable slope

conditions, it is recommended that the smaller dumps be left undisturbed, especially the side slopes, unless they are to be completely removed, possibly for use as backfill material. From a geologic standpoint there is no reason why the Creole dump could not be used as a construction site provided that the foundations for any structures errected on the dump are designed in accordance with the recommendations of a qualified soils engineer.

APPENDIX A

PART I: Summary of Subsurface Soil Conditions as Reported in the Consultants Report

Zone	Thickness	Description	Location (Test Hol
1	1.5' to 3.0'	Black Silty Top Soil	all
2	3.0' to 6.5'	Sand through Cobbles in a silt matrix	1,2,3,4,7,9
*3	1.5' to 8.5'	Medium plasticity clay and clayey silt	2,5,6,7,8,10
4	–	Weber Quartzite	1,2,3,4,5,6,8,9,10

*Soils reported as clayey silts also placed in this group.

Part II: Logs of Test Holes Examined by UGMS Personnel during May, 1979

Test Hole No.1

0.0-1.7'	Silty Sand-Sandy Silt; (SM-ML), black, loose to medium dense, non- to slightly-plastic, moist, abundant organics.
1.7-8.0'	Silty Clayey Gravel with Boulders; (GM-GC), brown, dense, low plasticity fines, moist.
8.0-9.0'	Quartzite cobbles and boulders in a clay matrix, very dense.

Bedrock was not encountered in test hole.

Test Hole No.2

0.0-2.1'	Silt with fine sand; (ML), black, soft to firm, non- to slightly-plastic, wet, abundant organics, some cobbles and boulders.
2.1-5.7'	Clayey Gravel; (GC), yellowish brown, medium dense to dense, low to moderately plastic fines, wet.
5.7-8.5	Clay; (CL), yellowish brown, stiff, medium plasticity, wet.

Backhoe did not clean test hole below 8.5 feet.

Test Hole No.3

Unable to locate, possibly destroyed during installation of waterline across site.

Test Hole No.4

0.0-1.8'	Silt with sand and clay; (ML), black, firm to stiff, low plasticity, moist, abundant organics, some cobbles and boulders.
1.8-7.0'	Clayey Gravel: (GC), yellowish brown, dense, low plasticity fines, wet, boulders to 1.5' diameter.

Bedrock was not encountered in test hole. Floor consists of densely packed quartzite cobbles and boulders in a clav matrix.

Test Hole No.5

0.0-2.0'	Silt with sand and clay; (ML), black, firm, slightly plastic, moist, abundant organics.
2.0-6.2'	Clayey Gravel; (GC), yellowish brown, dense, low plasticity, moist, boulders to 1.0' diameter.
6.2-9.0'	Clay; (CL), yellowish brown, stiff to very stiff, moderately plastic, moist.

Test hole carried 1' below original grade, did not encounter bedrock. A second backhoe pit was discovered in the vicinity of Test Hole No.5, it had not been cleaned and the soils exposed did not come close to matching the consultant's original log, so it is assumed that the log of the test hole presented here is the correct one.

Test Hole No.6

Inspection showed that this test hole encountered bedrock at depth of about 2.0 feet. Rock exposed was highly fractured.

Test Hole No.7

0.0-3.0'	Sandy Gravel; (GM), fill, portion of old Creole Mine dump.
3.0-5.0'	Silt with Sand; (ML), black, top soil material similar to that described in other borings.

Test Hole No.7 (continued)

5.0-7.5'	Clayey Gravel; (GC), yellowish brown.

Hole sluffed below 7.5'

Thickness of soil horizons approximated in this test hole due to unstable condition of mine dump material above the excavation.

Test Hole No.8

0.0-1.5'	Sandy Silt; (ML), black, firm, slight plasticity, moist, abundant organics, boulders and cobbles.
1.5-5.0'	Clay; (CL), brown, stiff to very stiff, low to moderately plastic.

Test hole has sluffed below 5.0 feet.

Test Hole No.9

0.0-1.5'	Sandy Silt; (ML), black, firm to stiff, non- to slightly-plastic, moist, abundant organics, some cobbles and boulders.
1.5-6.0	Clayey Gravel; (GC), yellowish brown, dense, low to moderately plastic fines, boulders to 1½' diameter.

Test hole has sluffed below 6.0 feet.

Test Hole No.10

0.0-2.0'	Silt with sand and clay; (ML), black, soft, non- to slightly-plastic, wet, abundant organics.
2.0-6.0'	Clayey Silt and Silty Clay; (ML & CL), yellowish brown, firm, low plasticity, wet, some gravel.
6.0-8.0'	Clay; (CL), yellowish brown, firm to stiff, moderately plastic, wet.

Water sanding in test hole at 8.0 feet.

SHBA AGRA

April 27, 1994

REPORT
ENGINEERING GEOLOGY RECONNAISSANCE
SWEENEY PROPERTIES
WEST OF "OLD TOWN AREA"
PARK CITY, UTAH

Prepared For:

Sweeney Properties
115 Woodside
Park City, Utah 84060

SHB AGRA Job No. E93-2267

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April 22, 1994

SHB AGRA Job No. E93-2267

Sweeney Properties
115 Woodside
Park City, Utah 84060

Attention: Dr. Patrick Sweeney

Re: Report
Engineering Geology Reconnaissance
Sweeney Properties
West of "Old Town Area"
Park City, Utah

Gentlemen:

1. INTRODUCTION

1.1. General

Presented in this report are the results of our engineering geology reconnaissance of the Sweeney Properties site which is located west of the Old Town portion of Park City, Utah. The general location of the site with respect to major topographic features and existing facilities, as of 1975, is shown on Figure 1, Vicinity Map. A more detailed layout of the site showing general topography, ski trails, major outcrops, and mine workings, are presented on Figure 2, Site Plan.

1.2. Objectives and Scope

The objectives and scope of this study were planned in discussions between Dr. Patrick Sweeney of Sweeney Properties, and Dr. Jeffrey R. Keaton of SHB AGRA, Inc. The objectives of this study were to:

1. Inventory and evaluate the engineering geology parameters of bedrock exposed at abandoned mine openings and primary bedrock outcrops at the site.
2. Provide initial discussions pertaining to the engineering geology characteristics of the site.

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

1. An initial office program including a review of the geologic literature, existing mine opening inventories, geologic maps, and the examination of stereoscopic aerial photographs.

2. A field program consisting of an engineering geologic reconnaissance of mine openings and outcrops.

3. Preparation of this summary report.

1.3. Authorization

Authorization was provided by Dr. Patrick Sweeney by signing a copy of our Professional Services Agreement dated June 28, 1993.

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 geologic conditions encountered at the mine openings and outcrops, and our other reconnaissance data.

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

If additional information is found at the site during the construction phase of the project, we need to be notified immediately so that our recommendations can be reviewed and modifications can be made to this report, if necessary.

2. SITE DESCRIPTION

The site is a moderately to moderately steeply sloping trapezoidal-shaped parcel of land having an area of approximately 125 acres. The boundaries of the site and existing site topography are shown on Figure 1. Elevations on the property range from 7040 feet on the northeast side of the site, to 7800 feet on the southwest side. Vegetation consists of scrub oak, aspen, fir, and spruce, with open areas occupied by sagebrush and grasses. An operating ski-lift and an abandoned mine gondola cross the northern portion of the site. A loading platform for the ski lift and three ski trails are also present on the site. Scattered on the site are several abandoned mine openings.

3. PROPOSED DEVELOPMENT

At the time of this study, overall detailed site development plans had not been finalized. It is our understanding that 15 acres within the northeast portion of the site will be developed for residential home sites, with lots ranging from one-quarter to over one-half acre in size. In other areas, clusters of two to three level condominium structures and possibly high density four to six level resort type structures have been considered.

Homes and two to three level condominium structures will generally be of wood-frame construction above grade, and reinforced concrete construction below grade. Loads imposed by bearing walls and columns will generally be light to moderately light.

The four to six level structure could be of wood-frame or possibly reinforced concrete construction, and would impose moderate to moderately high loads.

Site development will require the construction of primary access and secondary roads. Everything will be done to minimize cuts and fills associated with the roadway. However, in many areas, cuts and fills of 15 to 20 feet may be required. Similar cuts and fills may be required in the higher density building development areas.

4. INVESTIGATIONS

4.1. Field Program

Prior to our field program, a detailed review of literature, inventory reports, and aerial photography were performed. This was followed by a general site reconnaissance of mine openings and rock outcrops. The mine openings and rock outcrops examined during our reconnaissance were selected on the basis of proximity to the portion of the site that will be developed. These locations are shown on Figure 2.

5. SITE GEOLOGY

The prominent rock type of the site area is the Weber Quartzite. This formation has been described as "pale gray and tan weathering quartzite and limy sandstone; some interbedded gray to white limestone and dolomite" (Bromfield and Crittenden, 1971). The Weber Quartzite is estimated to be from 1,300 to 1,500 feet thick and comprises the oldest exposed rocks in the area. Overlying the Weber Quartzite is the Park City Formation, which is comprised of limestones, cherts, sandstone, and shale, that ranges from 550 to 650 feet in thickness. The Weber formation was deposited during the late Pennsylvanian, and the Park City formation consists of Permian age rocks.

The area of the site has been subject to anticlinal folding and thrust faulting (Bromfield, 1968). The most prominent structural feature in the area is the Park City anticline, which runs on a northeast plunging axis. The axis of the anticline is located approximately 700 feet to the west of the site (Bromfield and Crittenden, 1971; Crittenden, Calkins, and Sharp, 1966).

The nearest know active faults are associated with the Wasatch fault zone which lies approximately 12 miles west of the site.

The geology exposed in the mine openings and outcrops consisted of massive bedded quartzite containing some interbeds of fine-grained sandstone laminae. The beds were found to be near horizontal, generally dipping gently to the southwest. Vertical to near vertical joints spaced 1 to 3.5 feet apart were observed in the exposures. Two joint orientation trends appear to have developed in the quartzite beds. A primary trend is oriented at roughly 230 degrees, and a secondary trend is oriented at about 70 degrees. Near the surface, the quartzite was observed to be more highly fractured from weathering and spalling processes.

A tabulation of the engineering geology parameters of the exposures is presented on Table 1, Engineering Geology Parameters of Mine Openings and Outcrops. Additional information with respect to observations taken at the exposures is presented in Appendix A, Site Exposure Inventory. The bedrock exhibits high strength and low compressibility characteristics, and is not moisture sensitive. The bedrock is overlain by colluvial soils, which can best be described as mixtures of silt; some clay; and angular sand, gravel, rubble, and boulder sized pieces of quartzite.

The soils are not moisture sensitive and generally exhibit moderately high strength and low compressibility characteristics.

The true static groundwater table is at significant depth and should not affect design, construction, or performance of the proposed facilities. Near surface perched groundwater conditions which will be most prevalent during the late spring and summer months will, however, be significant.

The combination of fairly steep slopes, colluvial soils, and near surface perched groundwater, has resulted in some relatively shallow soil slope instability in the area of the proposed development.

6. DISCUSSIONS AND RECOMMENDATIONS

6.1. Discussions of Findings

By far, the most significant geotechnical aspects of the site which will affect design and development, are 1) cuts and fills, 2) slope stability and 3) groundwater. All attempts must be made in the layout of the primary and secondary roadways to minimize the amounts of cuts and fills which will be required. However, considering the magnitude of the site and the overall slopes, even with very careful alignment detailing moderate cuts and fills will be required in some areas. To minimize areas of disturbance and thus make the development most aesthetically pleasing, we strongly recommend the utilization of reinforced earth systems, retain downslope fills.

Some instability has been noted within and immediately adjacent to the site. The slope instability, in all cases, has been related to the movement of the surficial colluvial soils over the underlying bedrock. In all cases, to our knowledge, the movement has been associated with either long time or seasonal near surface groundwater conditions. Therefore, in conjunction with overall site development, it will be necessary to install subdrains.

In all anticipated conditions, the proposed structures may be supported upon conventional spread and continuous wall foundations established upon suitable colluvial soils, bedrock, and/or structural fill extending to suitable materials. Foundation conditions should generally not have any significant affect on overall site development.

There are numerous abandoned mine workings in the site area. Obviously, the structures should not be established over these workings, unless the workings are at extreme depth. Individual workings will have to be evaluated on a site specific basis, when they fall within the area of the proposed structure.

In the following sections, detailed discussions pertaining to stability, subdrains, earthwork, foundations, and other geotechnical aspects which will effect initial site development, are presented.

6.2. Slope Stability

6.2.1. General

Instability, where it has been observed within or adjacent to the site, and in the general geologic setting, has been related to the colluvial soils overlying bedrock. To the best of our knowledge, no mass instability within the quartzite bedrock has occurred at this site or in other immediate areas in the same geologic setting.

The instability of the colluvial soils is also generally related to near surface groundwater conditions. A combination of groundwater colluvial soils, and steep slopes, has and can lead to "natural" instability. Extensive earthwork operations, especially cutting soils out of the toe of these potentially unstable areas, loading the heads of slide areas, or directing water to these areas, significantly increases the potential for instability. Overall stability in these conditions is best maintained or improved by 1) the installation, in some cases, of some very extensive and deep subdrains, and 2) very cautious earthwork operations. From an overall site development standpoint, even though the stability of these potentially unstable areas, can be improved. Our strongest recommendation for site development is to avoid these areas. The site is large enough, and the number of potentially unstable areas few enough, that this should not drastically affect site development plans. Areas of potentially unstable colluvial soils can best be identified and related to areas of major or even shallow natural drainages.

6.2.2. Bedrock

Both deep seated (mass) and shallow or erosional-type stability must be considered. The quartzite bedrock found at the site, based upon our field observations, seems to be of higher quality, that is less fractured, then typically encountered in other portions of Park City.

Mass, that is deep seated instability, has not been a problem in quartzite in the Park City area, and is not anticipated at this site for the maximum depths of cuts projected. The greatest concern is related to surficial instability, that is erosion, sloughing, etc. Highly fractured quartzite of the type encountered in other portions of Park City, can be cut to very steep slopes, even near vertical, and will remain stable in a mass stability standpoint, indefinitely. However, highly fractured bedrock will ravel and within a few years, will result in erosional slopes on the order of 1.2 to 1.4 horizontal to 1 vertical. This is essentially the angle of repose of the angular fractured pieces of quartzite bedrock. Generally from an overall highway maintenance standpoint, slopes in highly fractured quartzite bedrock are generally designed to be cut at one horizontal to one vertical, with the understanding that some clean-up of ravelled materials will be periodically required. Park City's philosophy has, however, been that final grading should be such as to minimize the amount of long-term maintenance. If this philosophy prevails, cuts in these highly fractured soils will generally have to be 1.3 to 1.4 horizontal to 1 vertical. As stated previously, the bedrock at this site does not appear to be as highly fractured as others. Still, even though Park City might recommend flatter slopes, we will, from a planning standpoint, recommend that the final cut slopes in fractured bedrock be one horizontal to one vertical.

Much steeper slopes, on the order of one-quarter to one-half horizontal to one vertical, to heights of 15 to 20 feet, may be utilized in the more massive quartzite bedrock. If cuts greater than these depths are required, then benches sloping slightly back into the overall slope, and at least four feet wide, are required every 15 feet in total vertical height. Some chain netting, or other precautions may be required to catch and retain small to moderately sized bedrock particles from spalling off at steep cut slopes.

6.2.3. Soils

In natural colluvial soils, where groundwater is not a problem, excavations of as much as six feet in height can be constructed at slopes of one-half horizontal to one vertical, and maintain mass stability. However, these slopes are extremely susceptible to erosion and sloughing, and would, therefore, have to be covered by rock walls or other similar type structures. Cuts in excess of six feet in height, should generally be no steeper than one and one-half horizontal to one vertical. Again, the surface must be protected against erosion. Where groundwater is encountered, similar type construction can be employed, only after the groundwater conditions have been controlled by extensive subsurface drainage. Any kind of cut activities in colluvial soils with uncontrolled groundwater most likely will lead to some long-term instability.

Fill slopes should be held to a minimum whenever possible. Where angular pieces of quartzite bedrock are utilized, the fill slopes can be constructed at one and one-half horizontal to one vertical and provide both mass and surficial stability. In soils, the fill slopes would generally have to be constructed at least two horizontal to one vertical to provide mass and surficial stability. Because of the steepness of the site, these slopes would essentially "chase" the natural slopes, and would result in extensive disruption to natural terrain and vegetation. Therefore, whenever substantial fills are required, we strongly recommend the consideration be given to reinforced soil structures. These structures can range from rough finish wire wall or reinforced timber crib walls, to structures faced with reinforced concrete panels of different types. Numerous examples are present within the Park City area. The general soils available are suitable to construct reinforced earth structures, provided that appropriate drainage is part of the overall design. Costs, assuming that fairly substantial amounts of reinforced earth structures will be utilized, could range anywhere from approximately $13.00 per face foot, to $30.00 per face foot, considering the type of facing. Considering the mining heritage, the rustic-look of properly engineered and designed treated timber-facing might be quite acceptable.

6.3. Earthwork

6.3.1. Excavations

Excavations of surficial highly fractured bedrock, generally to depths of no more than three to four feet and the colluvial soil, can be carried out utilizing heavy track-mounted equipment. Excavations of more than a few feet into the bedrock, will in nearly all cases, require drilling and blasting.

Temporary construction excavations not exceeding four feet in depth in cohesive soils above the water table, may be constructed with near vertical sideslopes. Deeper excavations up to 10 to 12 feet, again within predominantly cohesive soils above the water table, should be constructed with sideslopes on the order of one-half to three-quarters horizontal to one vertical. If groundwater is encountered in any excavations, significantly flatter sideslopes will be required.

Temporary construction excavations up to 10 to 15 feet in bedrock can generally be constructed with near vertical to one-quarter horizontal to one vertical sideslopes. Deeper excavations should incorporate minimum of 4-foot wide benches on 15-foot vertical increments. For temporary excavations, proper control of spall of the rock off the steep walls must be provided.

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

6.3.2. Fill Material

Structural fill will be required as backfill over foundations and utilities, and site grading fill. All structural fill must be free of sod, rubbish, construction debris, frozen soils, and other deleterious materials. Structural site grading fill is defined as fill which is placed over fairly large open areas to raise overall site grade. Generally, for this type of fill, we recommend that the maximum particle size generally not exceed four inches, although occasional larger particles of up to six to eight inches may be incorporated provided that they do not result in "honeycombing" or preclude the obtainment of the desired degree of compaction. In confined areas, we recommend that the maximum particle size generally not exceed two and one-half inches. For fairly substantial structural site grading fills in parking or roadway areas, larger particles can be incorporated into the structural fill with the understanding that these types of fills would be subjected to totally unacceptable settlements for structures, but acceptable settlements for roadways and parking areas.

6.3.3. Fill Placement and Compaction

Soil meeting the fairly stringent maximum particle size requirements, as stated above, should be placed in lifts not exceeding eight inches in loose thickness. Under buildings, we generally recommend that the fills 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. As backfill over foundations and utilities, compaction of at least 90 percent of the above defined criteria is recommended. The 90 percent criteria is also applicable for roadways and parking areas.

Where materials with large particle sizes and thicker lifts are utilized, procedural specifications will be developed.

6.4. Subdrains

From a geotechnical standpoint, that the most cost effective systems or facilities which can be utilized are subdrains. Wherever there is any concern with regard to significant near surface groundwater flows in cut and fill areas, and upgradient of below grade structures, it is essential that extensive subdrain systems be employed. The subdrains generally will consist of a minimum four to six inch diameter slotted or perforated plastic or other durable material pipe encased in a free-draining granular materials, such as "pea" gravel or three-quarter to one inch minus clean gap-graded gravel. The gravel will generally extend 2 inches below and laterally, and at least 12 to 18 inches above the pipe. To reduce the possibility of long-term plugging, the gravel should be wrapped in a geotextile fabric such as Mirafi 140N or equivalent. The slope of the subdrain pipe should generally be at least 0.5 percent, to a suitable point of gravity discharge.

1 American Association of State Highway and Transportation Officials

2 American Society for Testing and Materials

The backfill, in most cases, will act as a chimney drain portion of the overall system, and must consist of a free-draining sand and gravel extending to within two feet of final grade. The subdrains must be installed as far in advance of other construction as possible.

6.5. Spread and Continuous Wall Foundations

6.5.1. Design Data

All indications are that the structures, as proposed, can be supported upon conventional spread and continuous wall foundations established upon suitable natural soils, bedrock, and/or structural fill extending to suitable natural soils or bedrock. All footings exposed to the full effects of frost and established upon soils or highly fractured bedrock, should be established at a minimum of three and one-half feet below lowest adjacent final grade. Footings protected from the full effects of frost may be established at a higher elevation, although a minimum depth of embedment of 18 inches is recommended for confinement purposes. Floor slabs and pavements may be considered equivalent to soil in determining depth of embedment. Minimum recommended width for footings established upon soils is 18 inches for continuous wall footings, and 24 inches for isolated spread footings.

Where sound, that is only slightly fractured bedrock is encountered, the footings may be established directly upon the bedrock without any specific depths of embedment. To resist lateral loading, and to provide passive resistance, however, we do recommend that the footings be tied with anchors to the bedrock, and that some outside backfill be utilized to minimum thicknesses of approximately 18 inches. Minimum recommended widths for footings established on massive bedrock are 12 inches for continuous wall foundations, and 18 inches for isolated spread footings.

For preliminary design, the following bearing pressures for real vertical loads may be utilized:

Suitable soils	- 3,000 psf
Highly fractured bedrock	- 5,000 psf
Massive bedrock	- 12,000 psf

The above bearing pressures for footings established on soils may be increased by 50 percent for total load conditions. Real loads are defined as the total of all dead plus frequently applied (reduced) live loads. The term "net bearing pressure" refers to the pressure imposed by the portion of the structure above lowest adjacent final grade. Therefore, the weight of the footing and backfill to lowest adjacent grade, need not be considered. For bedrock, the real load pressure may be increased by 100 percent for total load conditions. Maximum edge bearing pressures which can be utilized must be evaluated depending upon the type of loading, and the materials upon which the footings are established.

6.5.2. Settlements

Settlements of foundations designed and installed in accordance with the above recommendations, will ultimately be designed and selected to induce settlements generally no more than five-eighths to three-quarters of an inch.

6.6. 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 friction of 0.40 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.

6.7. Lateral Pressures

The lateral pressure parameters, as presented within this section, assume that the backfill will consist of a drained 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 35 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 45 pounds per cubic foot. For very rigid nonyielding walls, granular backfill should be considered equivalent to a fluid with a density of at least 60 pounds per cubic foot. The above values assume that the surface of the soil slope behind the wall is horizontal, 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 45 pounds per cubic foot, 60 pounds per cubic foot, and 120 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. All of the equivalent fluid pressures should be increased by 18 pounds per cubic foot for dynamic lateral pressures which would be imposed during a moderately severe earthquake. It should be noted that the lateral pressures, as quoted, assume that the backfill materials will not become saturated.

6.8. Additional Studies

The primary purpose of this report was to provide general geotechnical parameters which can be utilized in overall site development planning. Obviously, for any significant structure, whether building, roadway, retaining wall, etc., site specific studies will be required.

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

Respectfully submitted,
SHB AGRA, Inc.

By ______________________
William J. Gordon
Professional Engineer No. 3457
State of Utah

WJG/sp (94-4-6)

Copies: Addressee (3)

Attachments:
References
Table 1, Engineering Geology Parameters of Mine Openings and Outcrops
Figure 1, Vicinity Map
Figure 2, Site Plan
Appendix A1 through A8, Site Exposure Inventory

REFERENCES

Bromfield, C.S., 1968, General geology of the Park City Region, Utah, in Erickson, A.J., Phillips, W.R., and Garmoe, W.J., eds., Guidebook to the Geology of Utah, No. 22, Park City District, Utah: Utah Geological Society, Salt Lake City, Utah, p.11-29.

Bromfield, C.S., and Crittenden, M.D. Jr., 1971, Geologic map of the Park City East Quadrangle, Summit and Wasatch Counties, Utah: U.S. Geological Survey, Geologic Quadrangle Map GQ-852, Scale 1:24,000.

Crittenden, M.D. Jr., Calkins, F.C., and Sharp, B.J., 1966, Geologic map of the Park City West Quadrangle, Utah: U.S. Geological Survey, Geologic Quadrangle Map GQ-535, Scale 1:24,000.

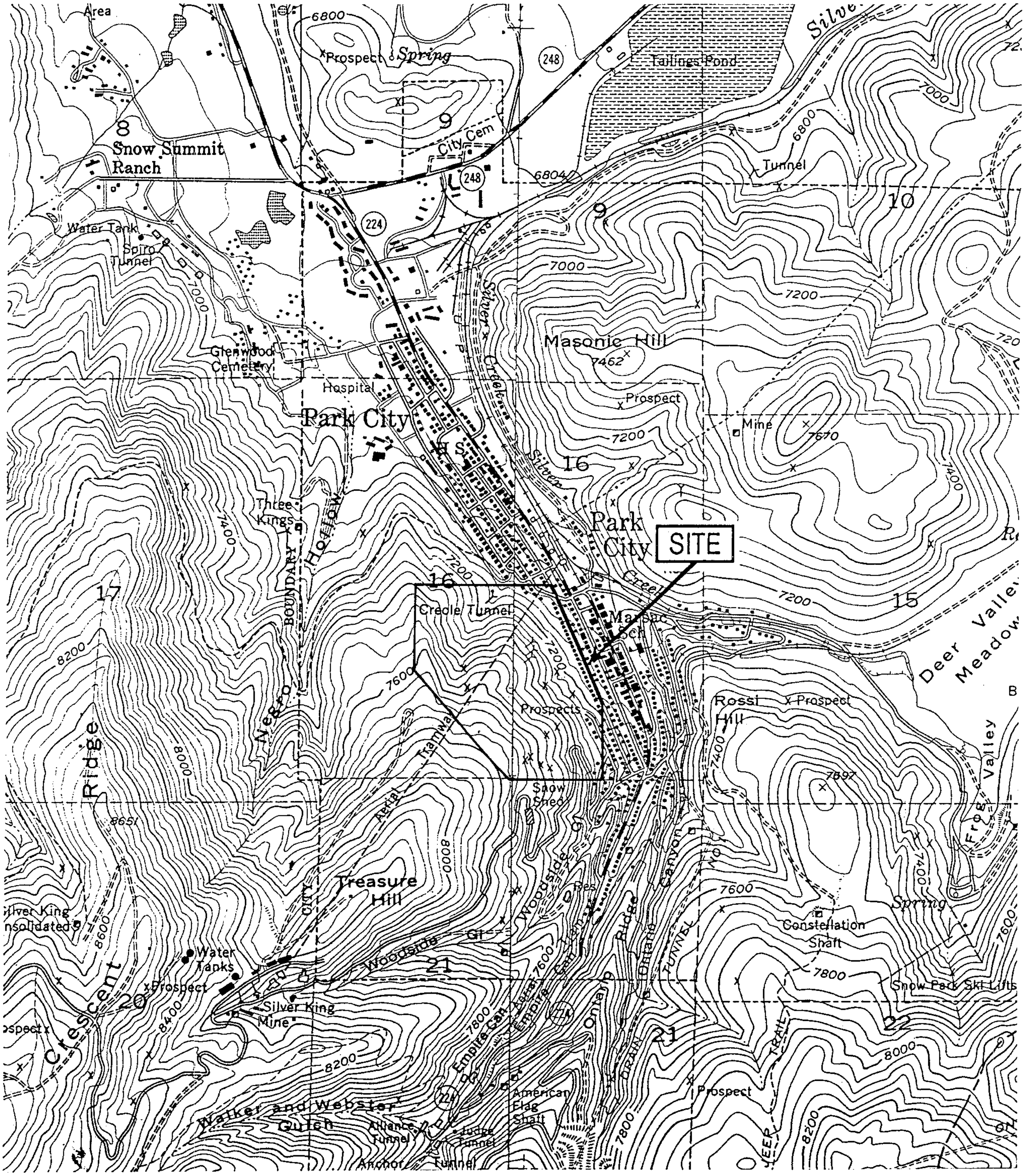
Table 1

Engineering Geology Parameters of Mine Openings and Outcrops

Exposure	Adit Orientation	Rock Type	Bedding Stike	Bedding Dip	Joint-set 1	Joint-set 2	Joint Spacing
Adit HO30	273°	Quartzite	255°	16°	317°	235°	1.5'
Adit HO31	207°	Quartzite	109°	22°	249°	190°	3.5 - 2.0'
Adit HO32	252°	Quartzite	28°	17°	252°	214°	*
Adit HC13	137°	Quartzite	250°	13°	290°	*	*
Outcrop 1	*	Quartzite	258°	5°	334°	200°	*
Outcrop 2	*	Quartzite	213°	10°	244°	182°	*
Outcrop 3	*	Quartzite	245°	10°	273°	210°	*

* Not observed

SWEENEY PROPERTIES
SHB AGRA JOB NO. E93-2267



SCALE IN FEET
1000 0 1000 2000

FIGURE 1
VICINITY MAP

SHB AGRA, INC.
Engineering & Environmental Services

REFERENCE:
USGS TOPOGRAPHIC MAPS TITLED "PARK CITY EAST, UTAH", 1955;
AND "PARK CITY WEST, UTAH", 1955, PHOTOREVISED 1975

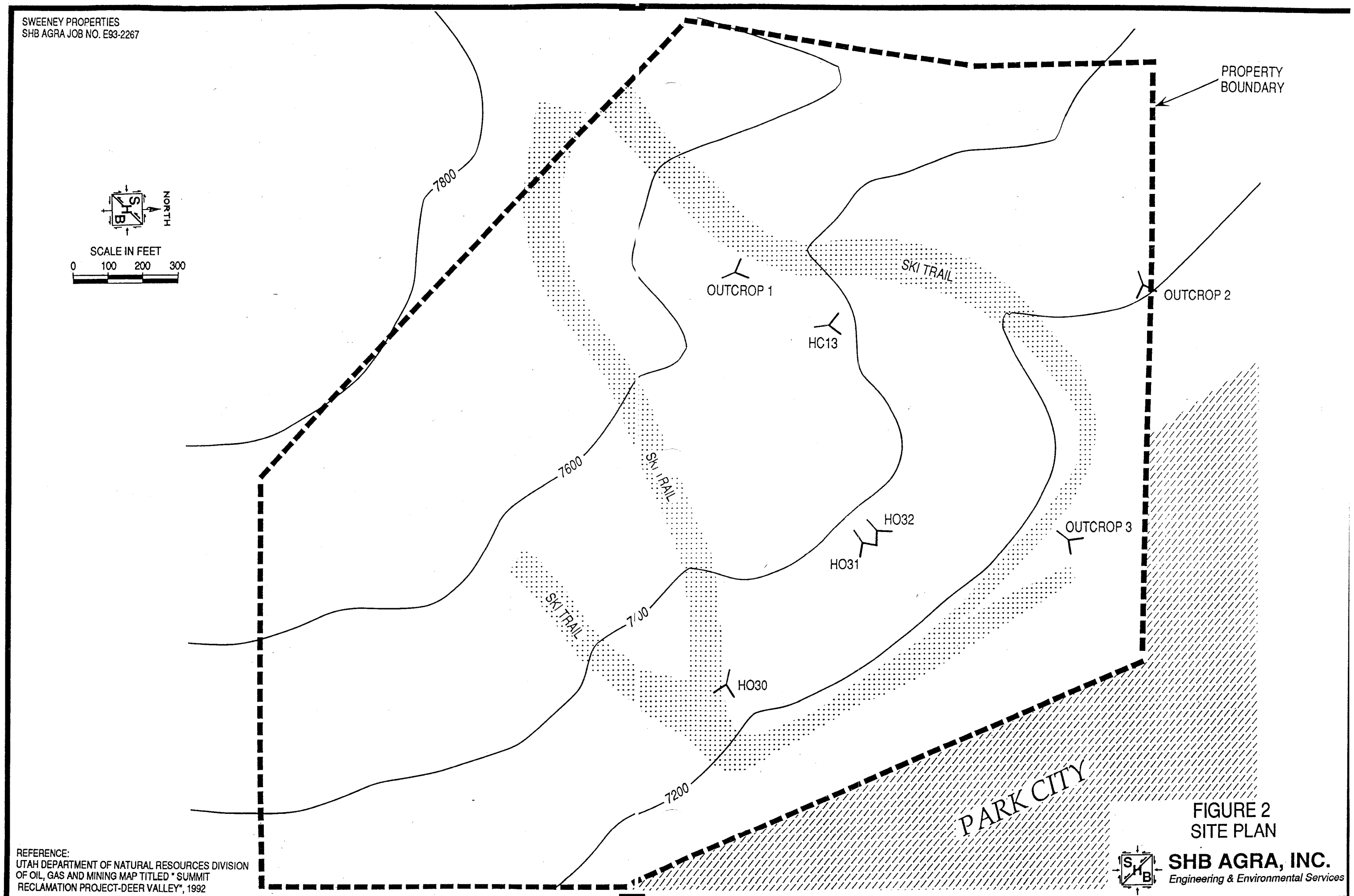


FIGURE 2
SITE PLAN

N69310

Site Exposure Inventory A1

Exposure: HO30

Location: SW1/4, NW1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.,

Exposure Type: Shaft

Exposure dimensions:

Width 5' Height 6' Length 54'

Orientation 273° Inclination 3°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with interbedded fine-grained sandstone laminations, and near vertical joints spaced 1.5 to 3 feet apart

Bedding:

Bed form Massive-Laminated

Bedding strike 255°

Bedding dip 16°

Jointing:

Joint strike 317° & 235°

Joint angle 69° & 81°

Joint spacing 1.5' & 1.5'

Remarks: No signs of groundwater seepage; no timbering observed

Site Exposure Inventory A2

Exposure: HO31

Location: SW1/4, NW1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.,

Exposure Type: Shaft

Exposure dimensions:

Width 4.5'   Height 5.5'   Length +100'

Orientation 207°   Inclination 7°-32°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with interbedded fine-grained sandstone laminations, and near vertical joints spaced 2.0 to 3 feet apart

Bedding:

Bed form Massive-Laminated

Bedding strike 109°

Bedding dip 22°

Jointing:

Joint strike 249° & 190°

Joint angle 87° & 79°

Joint spacing 3.5' & 2.0'

Remarks: No signs of groundwater seepage; no timbering observed; Depth to bedrock 2.0' to 3.0'

Site Exposure Inventory A3

Exposure: HO31

Location: SW1/4, NW1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.,

Exposure Type: Shaft

Exposure dimensions:

Width 4.5'   Height 5.5'   Length +100'

Orientation 207°   Inclination 7°-32°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with interbedded fine-grained sandstone laminations, and near vertical joints spaced 2.0 to 3 feet apart

Bedding:

Bed form Massive-Laminated

Bedding strike 109°

Bedding dip 22°

Jointing:

Joint strike 249° & 190°

Joint angle 87° & 79°

Joint spacing 3.5' & 2.0'

Remarks: No signs of groundwater seepage; no timbering observed; entrance partially collapsed; Depth to bedrock 0.5' to 1.0'

Site Exposure Inventory A4

Exposure: HQ32

Location: SW1/4, NW1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.,

Exposure Type: Shaft

Exposure dimensions:

Width 4.5' Height 4.0' Length 15'

Orientation 252° Inclination 23°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with near vertical joints

Bedding:

Bed form Massive

Bedding strike 28°

Bedding dip 17°

Jointing:

Joint strike 252° & 214°

Joint angle 66° & 84°

Joint spacing 3.5' & 2.0'

Remarks: No signs of groundwater seepage; no timbering observed; entrance nearly closed; Depth to bedrock 0.5'

Site Exposure Inventory A5

Exposure: HC13

Location: SW1/4, NW1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.,

Exposure Type: Adit

Exposure dimensions:

Width 6.5' Height 5.0' Length 14'

Orientation 137° Inclination 21°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with near vertical joints

Bedding:

Bed form Massive

Bedding strike 250°

Bedding dip 13°

Jointing:

Joint strike 290°

Joint angle 77°

Joint spacing NA

Remarks: No signs of groundwater seepage; No timbering observed; Entrance nearly closed; Depth to bedrock 0.5'

Site Exposure Inventory A6

Exposure: Outcrop 1

Location: SW1/4, NW1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.; On south side of Town Run ski trail

Exposure Type: Outcrop

Exposure dimensions:

Width 30.0' Height 30.0' Length NA

Orientation NA° Inclination NA°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with near vertical joints

Bedding:

Bed form Massive

Bedding strike 258°

Bedding dip 13°

Jointing:

Joint strike 334 ° &200

Joint angle 74° & 79°

Joint spacing NA

Remarks: Roughly triangular shaped outcrop

Site Exposure Inventory A7

Exposure: Outcrop 2

Location: SW1/4, NW1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.; North of ski-lift loading platform

Exposure Type: Outcrop

Exposure dimensions:

Width 30.0' Height 7.0' Length NA

Orientation NA° Inclination NA°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with near vertical joints

Bedding:

Bed form Massive

Bedding strike 213°

Bedding dip 10°

Jointing:

Joint strike 244 ° & 182

Joint angle 84° & 88°

Joint spacing NA & 2.0'

Remarks: Road cut outcrop

Site Exposure Inventory A8

Exposure: Outcrop 3

Location: SE1/4, SE1/4, SE1/4; Sec. 16; T. 2 S., R.4 E.; Below power-line

Exposure Type: Outcrop

Exposure dimensions:

Width 8.0'   Height 3.0'   Length NA

Orientation NA°   Inclination NA°

Rock Type: Quartzite

Description of Rocks: Massive pale-colored quartzite with near vertical joints

Bedding:

Bed form Massive

Bedding strike 245°

Bedding dip 10°

Jointing:

Joint strike 273 ° & 210

Joint angle 84° & 89°

Joint spacing NA

Remarks: Road cut outcrop; Depth to bedrock 1.0"

AGEC

October 7, 2003

AGEC

Applied Geotechnical Engineering Consultants, Inc.

October 7, 2003

MPE Incorporated
P.O. Box 2429
Park City, Utah 84060

Attention: Pat Sweeney
FAX (435) 649-6215

Subject: Geotechnical/Geological Consultation
Treasure Hill, Phase 3
Park City, Utah
AGEC Project No. 1030820

Gentlemen:

Applied Geotechnical Engineering Consultants, Inc. (AGEC) has been requested to provide geotechnical and geologic consultation in regards to the design and construction of the Treasure Hill, Phase 3 development to be located near the town lift and west of Lowell Avenue in Park City, Utah.

AGEC is currently in the process of reviewing the geologic reports that have been developed in the area along with reviewing published geologic literature. Our preliminary review to date indicates that:

> The stratigraphy of the site generally consists of Pennsylvanian age Weber Quartzite and Permian age Park City Formations. The Weber Quartzite consists of medium- to thin-bedded, pale gray to tan, fine-grained quartzite and sandstone while the Park City Formation consists largely of pale-gray fossiliferous limestone with some chert and sandstone.
>
> Bedrock exposed at localized areas of the site in road cuts, along the hillside and in abandoned mine workings consist predominately of massively bedded Weber Quartzite. Vertical to near vertical joints were observed and measured in the exposures. Two dominant orientations are apparent in the data (although more data is necessary for a good statistical sample) with trends of roughly 220 and 330 degrees.
>
> The Weber Quartzite is the dominant unit in the area proposed for development. The northwest portion of the property is largely underlain by Park City Formation. This includes some of the area proposed for fill placement. Quaternary age colluvial soils composed of clay, silt, sand, gravel, cobbles, and boulders overly the bedrock units over most of the site.

Based on the information currently available, it is our professional opinion that the proposed development is feasible from a geologic and geotechnical perspective. We anticipate that practical engineering solutions will be developed to provide:

600 West Sandy Parkway • Sandy, Utah 84070 • (801) 566-6399 • FAX (801) 566-6493

- stable construction slopes,
- stable long term slopes and "cliff" like landscaping,
- suitable foundation support,
- stable lateral support for the deep cuts,
- stable excavation waste disposal area.

Once the available information has been reviewed, an exploration program will be proposed to investigate the subsurface conditions in the area for the proposed development. The investigation will provide information for us to develop appropriate design parameters for design and construction of the proposed facility.

We look forward to working with you on this project. If you have any questions, please call.

Sincerely,

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
No. 143916
JAMES E.
NORDQUIST
STATE OF UTAH

James E. Nordquist, P.E.

JEN/sc
cc: Rob McMahon (Alliance Engineering, FAX (435) 649-9475)