SOILS STUDIES & OPINION REPORTS

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

by
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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

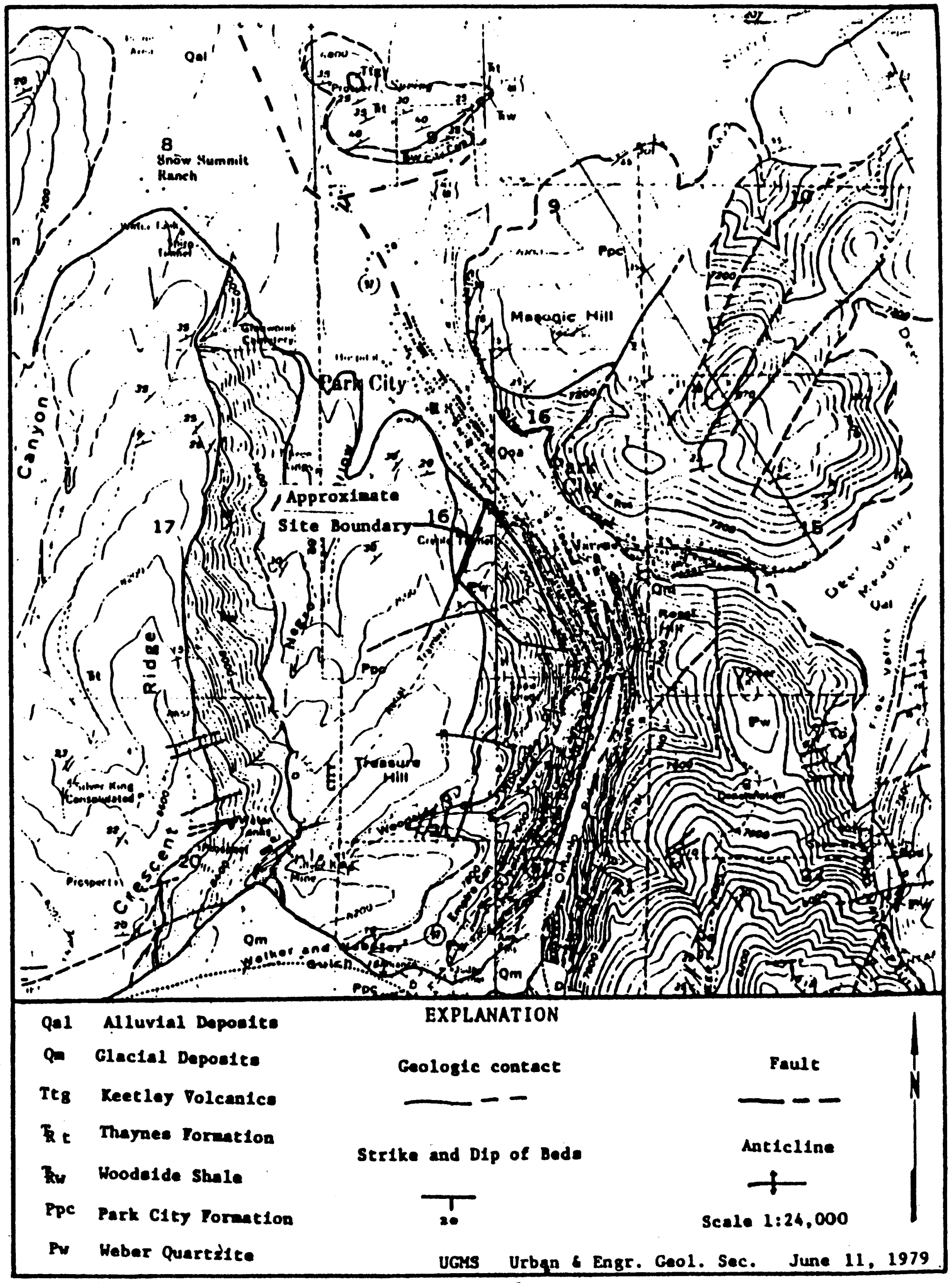

Figure 1 General Location and Geologic Map

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

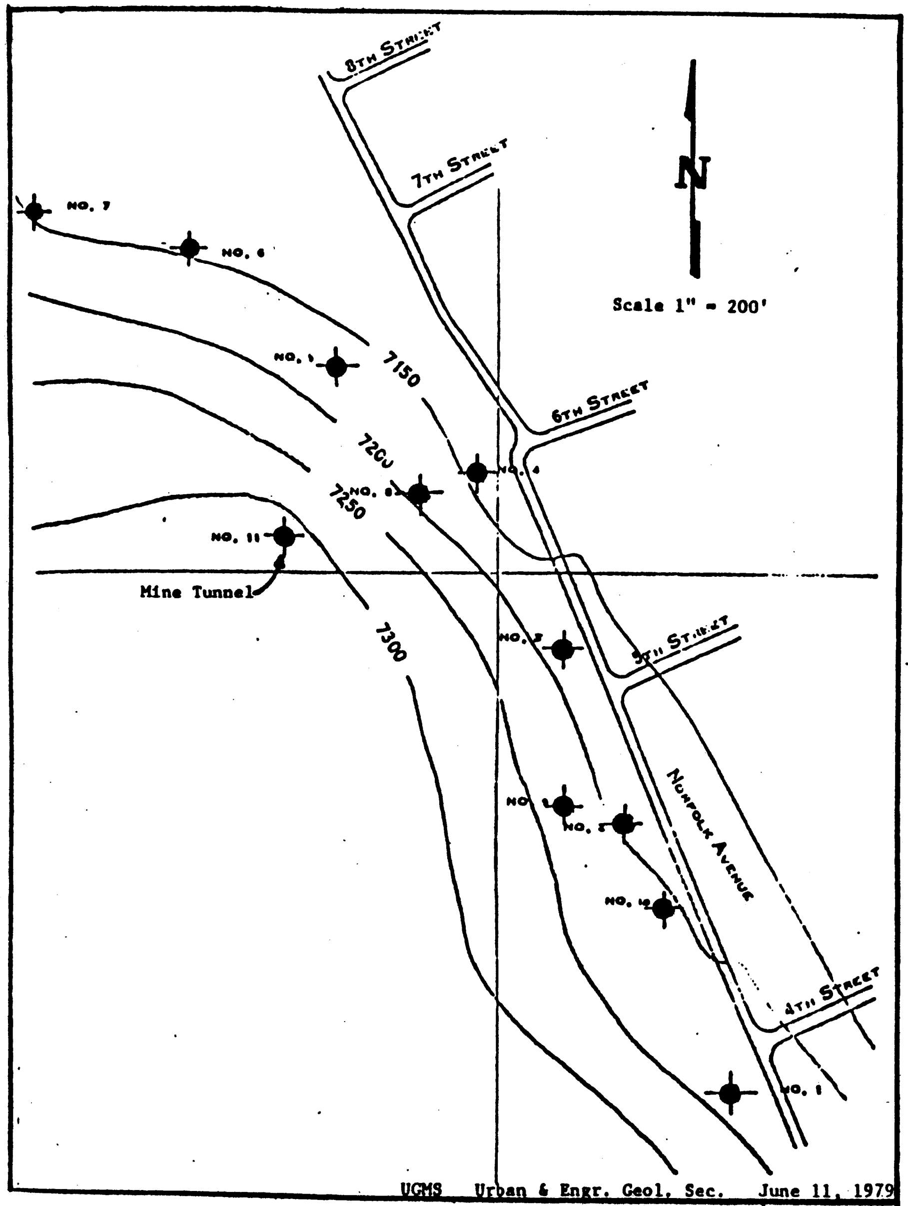

Figure 2 Location of Test Holes (Modified from consultants report)

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

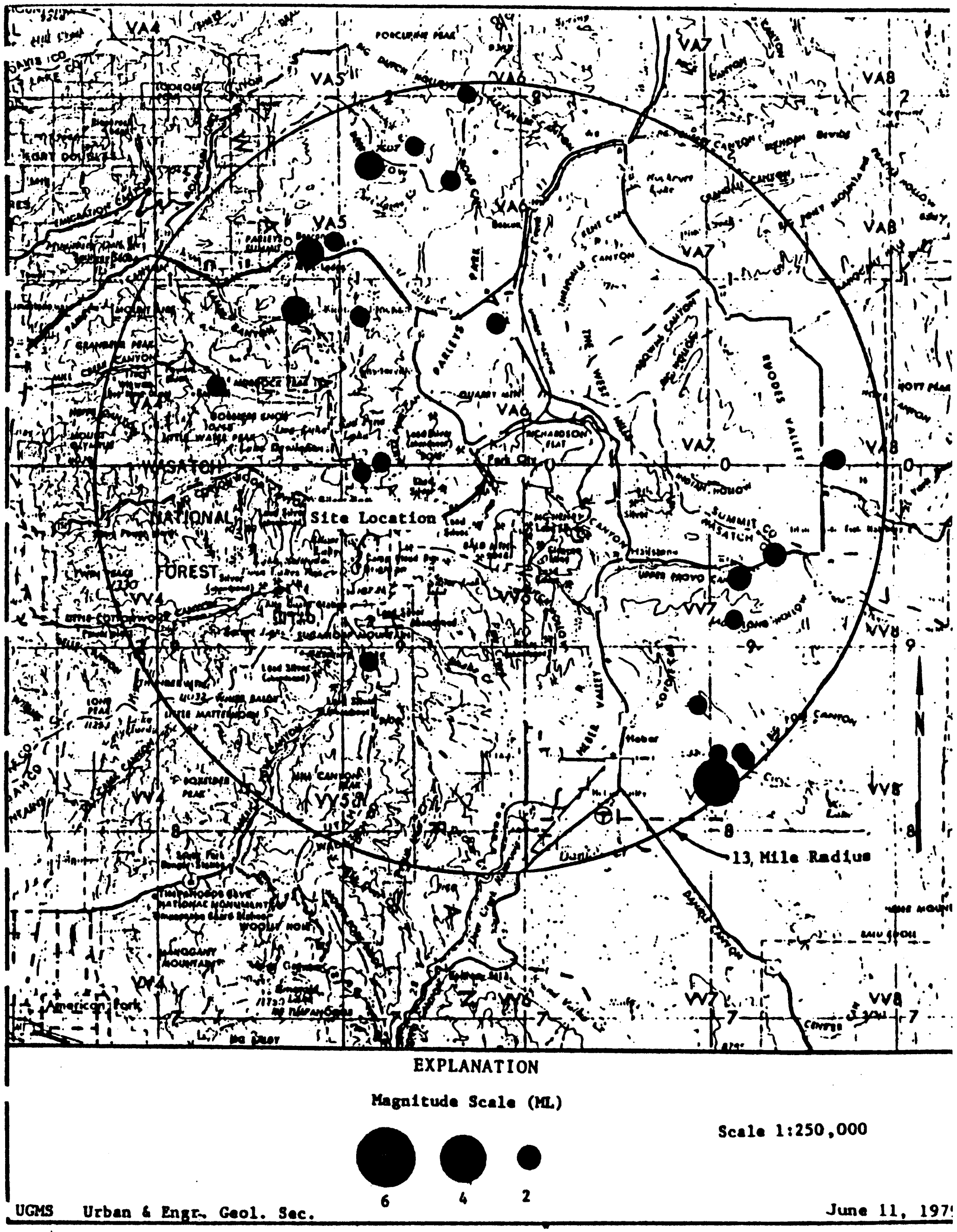

Figure 3 Location of Earthquake Epicenters, 1962 to 1978

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 rêmaining 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 itis 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

<u>Test Hole No.1</u>

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.

<u>Test Hole No.2</u>

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.