FEASIBILITY STUDY

PAYSON CITY RESERVOIR

Payson Canyon, Utah

Prepared for: Payson City

July 2015
July 31, 2015

Payson City
Attn: Travis Jockumsen
439 West Utah Avenue
Payson, Utah 84651

Re: Payson City Reservoir Feasibility Study

Dear Mr. Jockumsen:

A Feasibility Study has been completed for the proposed Payson City Reservoir to be located in Payson Canyon, Utah. This study was performed to evaluate the geologic and geotechnical feasibility, and the results are summarized in the report transmitted herewith. This report was prepared by RB&G Engineering, Inc., with primary contributors including:

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- S. Robert Johnson, P.E.
- Michael N. Hansen, P.G.
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- Bruce C. Barrett, P.E.

We appreciate the opportunity of providing this service for you. If there are any questions relating to the information contained herein, please call.

Sincerely,

[Signature]

Bradford E. Price, P.E.

bep/jal
FEASIBILITY STUDY

Payson City Reservoir

Payson Canyon, Utah

Prepared for: Payson City

July 2015
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PAYSON DAM & RESERVOIR
Payson, Utah
Geological and Geotechnical Feasibility Study

EXECUTIVE SUMMARY

A study has been completed to evaluate the geologic and geotechnical feasibility of constructing a reservoir in Payson Canyon, across Peteetneet Creek, southeast of Payson, Utah. The purpose of this study was to determine if a dam and reservoir can be constructed that will meet the storage and operation requirements of the owner while complying with all pertinent safety requirements of the Utah State Engineer-Dam Safety Division. The scope of field investigations defined for the feasibility study resulted in the development of limited geologic and geotechnical data from which to draw conclusions.

An initial geologic assessment was performed to evaluate the proposed dam site (Site No. 1) located about 1500 feet upstream of the existing power plant. Two additional sites were evaluated at locations about 3,000 and 5,000 feet upstream of the power plant. The initial assessment found no geologic fatal flaws that would prevent construction of a large dam in Payson Canyon. Due to geologic conditions which pose significant challenges and undoubtedly increase exploration and construction costs, the alternate sites were determined to be preferable.

Site No. 1 requires a dam height of about 198 feet to store 8660 acre feet of water. Site 3 requires a maximum dam height of about 158 feet to store the same volume of water. Site 3 is also the only site of the three which would allow construction of the dam and reservoir without encroaching on U.S. Forest Service land. Avoiding impact to federal lands would save the project several years and millions of dollars. Site No. 3 was selected as the best site to initiate subsurface investigations. The alignment was adjusted a few hundred feet upstream to Site No. 3A based on the results of initial investigations and field reconnaissance.

We conclude, based upon studies performed, that it is feasible to construct a dam and reservoir that will meet or exceed dam safety requirements regulated by the Utah State Engineer-Dam Safety section. The dam site selected avoids impacting federal lands. Extensive foundation treatment, including a deep cutoff excavation extending into bedrock (ranging from 20 to 120 feet deep) and a 100 to 150 foot deep grout curtain will be required to reduce seepage to an acceptable level.
A storage capacity of 9,700 acre feet with a maximum dam height of 158 feet is feasible. This includes 1,050 acre feet resulting from removal of embankment fill from the reservoir basin. The site is best suited for a zoned Earthfill dam. Sufficient quantities of granular soils (Zone II) and rock for riprap exist from required excavations and within or adjacent to the reservoir basin to construct the outer shells and process filter and drain material. Sufficient quantity of lean clay likely exists in the left abutment cutoff excavation and reservoir basin to construct the impervious core (Zone I).

The engineer’s opinion of probable cost for the embankment and appurtenant hydraulic structures (outlet works and spillway), including preliminary and final explorations, engineering design, and contingencies, is approximately $50 million dollars. This results in a cost per acre foot of about $5155. This cost per acre foot is approximately 1.5 times the cost of other recently constructed dams we are aware of in the State of Utah having a capacity less than 10,000 acre feet. Costs for property acquisition, gas line relocation and road relocation are not included in this cost.

Preliminary design investigations including trenching, additional drill holes and test pits are recommended prior to proceeding with Final Design and Construction. Trenching the lineament feature on the left side of the basin and left abutment to determine if it is depositional or a fault trace should be a high priority. The City might consider approaching the Utah Geological Survey to investigate the feature and update the published geologic map if necessary. Prior to initiating preliminary design investigations, we recommend that the City investigate potential funding sources.

Additional investigations may encounter conditions that could not be identified with the scope of work requested for this feasibility study. We reserve the right to modify conclusions and recommendations based upon data obtained from future investigations.
I. GEOLOGIC ASSESSMENT

I.1 INITIAL GEOLOGIC ASSESSMENT

I.1.1 PURPOSE

An initial geologic assessment of the proposed dam site was performed in March 2015. The purpose of the initial assessment was to identify any geologic condition which would be considered a potential fatal flaw, thus eliminating the site from consideration. The following report presents our assessment with regard to the geologic suitability of the proposed site.

I.1.2 BACKGROUND

The proposed Payson Reservoir Dam site is located in Payson Canyon between Tithing Mountain and Dry Mountain, southeast of Payson, Utah, as shown on the Vicinity Map in Figure I-1. The dam site is located near the western flank of the Wasatch Range. The active, normal, west-dipping Wasatch Fault of the Intermountain Seismic Zone forms the boundary between the Wasatch Range to the east and the Basin and Range Province to the west. Two segments of the Wasatch Fault are in close proximity to Payson Canyon. Mountain building processes which created the Wasatch Range have uplifted, deformed and faulted the sedimentary and volcanic rocks present in the vicinity. Subsequent erosion of these rocks has resulted in the formation of Payson Canyon and the deposition of alluvial/colluvial materials along the stream channel and on the canyon slopes.

I.1.3 REVIEW

The following materials were reviewed as part of this geologic assessment;

- Engineering Geology Reconnaissance, Proposed Peteetneet Creek Reservoir, Payson Canyon, Utah. AMEC Earth and Environmental, Inc., May 24, 2002
- Payson Reservoir Feasibility Study, Jones & DeMille Engineering, January 2013
- USGS Map I-2095, Surficial Geologic Map of the Wasatch Fault Zone, Eastern Part of Utah Valley, Utah County and Parts of Salt Lake and Juab Counties, Utah by Michael N. Machette, 1992
- UGS Map 227, Geologic Map of the Spanish Fork Quadrangle, Utah County, Utah by Utah Geological Survey, 2007
- Satellite photographs, Google Earth
From a review of available literature/photographs and a brief reconnaissance of the canyon, we find the site conditions as described by the 2002 AMEC and 2013 Jones & DeMille reports to be accurate portrayals of the geologic setting, and found no fatal flaws which would preclude construction of a large dam in Payson Canyon. However, any site has geologic and topographic details which may make one site or dam axis orientation preferable to another. Some of these details will be discussed under Additional Considerations. Issues dealing with land ownership and acquisition, roads, or utilities as they relate to eventual site selection are beyond the scope of this report and will not be discussed.

There are three significant geologic conditions present at potential dam sites within the canyon which would have a direct bearing on the design, constructability, project cost, and long-term performance of the structure and require additional emphasis. These conditions are (1) the effects of seismic loading from Wasatch Fault earthquakes, (2) the presence/location of the Payson Canyon Thrust Fault and (3) the steeply inclined bedrock formations present on both abutments.

I.1.4 WASATCH FAULT

Payson Canyon is located between, and in close proximity to, two segments of the active Wasatch Fault; the Nephi segment to the south and the Provo Segment to the north. The fault segments are capable of generating earthquakes up to magnitude 7.3, with recurrence intervals of about one to two thousand years. Any dam constructed in the canyon could therefore be subjected to potentially damaging earthquakes and peak ground accelerations approaching 1g. Robust defensive measures must be incorporated in the dam’s design to prevent damage or catastrophic failure as a result of crest settlement, flow slides, embankment cracking leading to internal erosion/piping, and foundation liquefaction. In addition, the canyon could potentially be subjected to earthquake loading from what is commonly referred to as the local or “floating earthquake”- an earthquake, which is not associated with any known fault, is incapable of surface rupture, and is capable of generating an earthquake with a maximum magnitude up to the low 6’s. A dam designed to perform satisfactorily under Wasatch Fault loading would probably be adequate under loading from a local or floating earthquake.

The bedrock surface in the canyon bottom is blanketed by a significant but unknown thickness of alluvium. Recent investigations of many existing dam foundations along the Wasatch Front and back valleys within the Intermountain Seismic Zone have shown that stream alluvium containing
sand and silt layers and lenses are potentially liquefiable. In addition, it has been found that gravelly deposits may also be liquefiable if the gravel is suspended in a sandy or silty matrix. The design seismic loading anticipated for Payson Canyon may be capable of liquefying susceptible soils. Detailed investigations of the stream alluvium will be required to determine the thickness, physical properties, and lateral extent of layers and lenses which may be potentially liquefiable. Because of the alluvium’s gravel content, exploration will be difficult and may require more than one exploration method (SPT, BPT, shear wave) to adequately describe the physical properties. If extensive deposits of potentially liquefiable materials are identified, the design may require removal of all alluvial deposits from the entire footprint of the dam, resulting in increased construction costs.

I.1.5 PAYSON CANYON THRUST FAULT

The 2002 AMEC report briefly describes the possibility of an east-west trending thrust fault along the bottom of Payson Canyon. Detailed investigations and mapping of the Spanish Fork Quadrangle subsequent to the AMEC report has shown that the postulated thrust fault does exist, as shown in Figure I-2. The older limestone deposits which make up Dry Mountain have been thrust over the younger limestone deposits of Tithing Mountain. The faulting occurred during the Laramide Orogeny, about 63 million years ago and is therefore interpreted to be very old and inactive, as shown in Figure I-3.

Because of the large fault displacement involved, it is expected that the rock along the trace of the fault is intensively fractured and may be so for a considerable distance away from the faulted contact between the two formations. Erosion of the fractured rock along the trace of the fault has resulted in the formation of Payson Canyon; therefore these conditions exist at any potential dam site in the canyon. Depth to bedrock may also be greater than what would normally be expected in a canyon this wide because of this fractured condition. A deeply incised bedrock channel cannot be ruled out without further exploration.

The orientation of the fault is also problematic because the fault trends upstream/downstream with respect to the dam and reservoir and provides a potential seepage path, in addition to the paths created by rock fracturing due to the folding and tilting of the bedrock during the Laramide. Additional foundation excavation, treatment, and grouting will probably be required to remove unsuitable materials and seal potential seepage paths. It is possible that even with an effective grout curtain carried to a suitable depth at the dam site a potential seepage path could still be present allowing water in the reservoir, which is in contact with un-grouted fractured
rock, to travel along the fault’s path underneath the grout curtain. This condition would result in reservoir leakage but should not present a significant potential dam failure mode.

**I.1.6 STEEPLY INCLINED BEDROCK FORMATIONS**

Limited structural geology data presented on UGS Map 227 (Figures I-2 and I-3) show that the bedrock formations dip toward the canyon from both abutments at angles from 40-50 degrees. A right abutment limestone outcrop, located near the debris basin, was examined during a site reconnaissance visit as shown in Photo I-1, and is probably a representation of what can be expected throughout the canyon. Note that the rock is not only steeply dipping toward the canyon but extensively jointed with joints sets spaced 2-3 feet apart. Not only do these characteristics present grouting challenges because of the number and spacing of joints, but makes shaping of the bedrock surface more difficult since a relatively smooth bedrock surface is required for dam embankment placement. Potential seepage paths are created at the bedrock/dam contact when stair-steps or overhangs exist. Dental concrete will probably be required to provide a suitable surface for embankment or RCC placement.

Additionally, the dip of the beds into the cutoff trench could lead to block slides and wedge failures during construction. Detailed investigations will be required to determine if shale and clay layers are present which would increase sliding potential.

**I.1.7 ADDITIONAL CONSIDERATIONS**

**I.1.7.1 SITE 1**

See Figure I-4 and Photo I-2. A comparison of the 2007 geologic map and the proposed axis indicates that the Manning Canyon Shale will probably be encountered within the footprint of the dam on the left abutment. This formation and the clayey soils derived from it are generally unstable when saturated and have resulted in slope failures ranging in size from small slumps to major landslides as exhibited in Provo Canyon. The geologic map shows only a limited lateral extent or thickness of this unit. However, if this unit is exposed in the cut-off trench during construction, it could present significant challenges to the work due to air-slaking and slope instability.

There is a large ravine on the left abutment located immediately downstream and parallel to the proposed axis. The ravine is controlled by a low-angle normal fault and the presence of the easily eroded shale. Runoff or debris flows from this drainage will likely impinge on the downstream embankment shell causing erosion of the dam. Also, buildup of sediment or
Impoundment of water could occur should the embankment block part of the ravine. Runoff from the ravine has created a relatively large alluvial fan on the left abutment. Although these materials may be suitable for use in the construction of the dam, in order to create a positive cut-off to bedrock, a significantly greater amount of material may have to be excavated to reach bedrock. Also, portions of this deposit may be liquefiable during a large earthquake, and could negatively affect the dam’s performance, if left in place under the downstream shell.

The intersection of the left abutment normal fault and the Payson Canyon Thrust Fault has probably resulted in intense fracturing of the bedrock. Another normal fault has been mapped in the right abutment immediately downstream of the proposed axis. An intensely fractured bedrock outcrop can be seen near the ravine. Because the property was posted, a closer examination was not made to determine if the fracturing was due to faulting or sliding. In order to avoid (to the extent possible) the influence of faulting, the alluvial fan and the shale, a shift of the proposed axis several hundred feet upstream is recommended.

I.1.7.2 SITE 2
See Figure I-4 and Photo I-3. This alternative site is located immediately downstream of the debris basin. Excavation in the basin has exposed stream alluvium to a depth of about 20 feet. No bedrock was observed in the basin or in the channel downstream of the spillway structure. However, scattered limestone outcrops occur over the right abutment a very short distance to the right of the stream channel, which may indicate that the bedrock surface may be at a shallow depth on the right side of the stream channel, as shown in Photo I-1. No bedrock outcrops were observed on the left abutment.

I.1.7.3 SITE 3
See Figure I-4 and Photo I-4. This alternative site is located a short distance downstream of Walker Flat and the proposed axis crosses the city park. The left abutment axis is heavily vegetated and covered with colluvium of an unknown thickness, and at the downstream edge of a large alluvial fan- no outcrops were visible from the park or on satellite photos. The elevation of the stream channel at the axis is about 5,070 to 5,080 feet, which is the approximate elevation of the Bonneville Stage of ancient Lake Bonneville. The lake probably played a significant role in the deposition of the alluvial materials and the topography of Walker Flat. Deposits within this canyon reach (including Walker Flat) include recent stream/fan alluvium, probably remnant shoreline and near-shore gravels and sands, lacustrine sands and silts, and possibly clay. Many of these deposits are potentially liquefiable if sufficiently thick and laterally extensive. However, these deposits are probably the best source for borrow materials in the canyon, although a high water table may be present at the lower elevations.
I.1.8 INITIAL CONCLUSIONS

As previously stated, we found no geologic fatal flaws that would prevent the construction of a large dam in Payson Canyon. However, to the extent possible, avoidance of adverse geologic conditions is always preferable to mitigation. In our opinion, Site 1 presents significant challenges that would undoubtedly increase exploration and construction costs. Because of the intersection of two, and possibly three faults in the foundation, the proximity to the Manning Canyon Shale, the ravine’s location, and the large alluvial fan, the two alternate sites would be preferable to Site 1 from a geology perspective. Even shifting the axis several hundred feet upstream may be insufficient to avoid all of the difficulties that may be present at Site 1. However, these conclusions were reached based on limited geologic information, and more detailed investigations are required before a final site selection can be made.

I.2 UPDATE TO INITIAL GEOLOGIC ASSESSMENT

I.2.1 PURPOSE

Since submittal of the March, 2015 Geologic Assessment, geotechnical investigations have been conducted to further assess the geologic suitability of a reservoir with the dam embankment at Proposed Site 3A. This work consisted of drilling, sampling, water testing, and logging of surficial deposits and bedrock of 4 drill holes; excavating, sampling, and laboratory testing of 8 test pits to evaluate potential borrow sources; and a more detailed examination of the stratigraphy and structure of the abutments. This information is presented in the following sections of this report.

I.2.2 FIELD EXPLORATION PROCEDURES

A subsurface investigation in the vicinity of the dam embankment was completed during this feasibility study. Drilling was performed with a CME 55 truck-mounted rotary drill rig using various combinations of NW and HW casing HWT rod and a tri-cone rock bit with water as the drilling fluid to advance the boring through overburden soils. Continuous coring of the bedrock was performed using NQ and HQ size core barrels. The percent recovery and Rock Quality Designation (RQD) obtained in each core run is shown on the boring logs. The RQD is defined as the percent of material within the core interval which has core lengths greater than twice the diameter of the core (NQ 3.75 inches, HQ 4.8 inches).
Sampling was performed at five-foot intervals throughout the overburden and weathered bedrock, and continuous coring was performed in the bedrock. Disturbed samples were obtained by driving a 2-inch split spoon sampling tube through a distance of 18 inches using a 140-pound weight dropped from a distance of 30 inches. The number of blows to drive the sampling spoon through each 6 inches of penetration is shown on the boring logs. The sum of the last two blow counts, which represents the number of blows to drive the sampling spoon through 12 inches, is defined as the standard penetration value. The standard penetration value, corrected for overburden and hammer energy, provides a good indication of the in-place density of sandy material; however, it only provides an indication of the relative stiffness of the cohesive material, since the penetration resistance of materials of this type is a function of the moisture content. Considerable care must be exercised in interpreting the standard penetration value in gravelly-type soils, particularly where the size of the granular particle exceeds the inside diameter of the sampling spoon. If the spoon can be driven through the full 18 inches with a reasonable core recovery, the standard penetration value provides a good indication of the in-place density of gravelly-type material.

It will be noted that it was not possible to drive the sampling spoon through the full 18 inches at some sampling locations. Where the sampling tube could not be driven through the full 18 inches, the number of blows to drive the spoon through a given depth of penetration is shown on the boring logs.

Miniature vane shear tests, which provide an indication of the undrained shearing strength of cohesive materials, were performed on samples of the clay soil during the field investigations. The results of these tests are shown on the boring logs as the torvane value in tons per square foot (tsf).

Field permeability tests were performed at about 5 to 10 foot intervals throughout the depth investigated using procedures outlined in Designation 7310 of the Bureau of Reclamation Earth Manual – 3rd Edition. The results of the permeability tests are also shown on the boring logs.

Test pits were excavated in the reservoir basin using a CAT 320 trackhoe. The test pits were logged by a geotechnical engineer and bulk samples were obtained at select locations for laboratory testing.
I.2.3 DAM SITE – SUBSURFACE SOIL, BEDROCK, & WATER CONDITIONS

Four borings were drilled during this feasibility study. The first two drill holes (15-2 and 15-3) were located along an alignment designated as Site 3 on Figure I-5. Based upon the subsurface conditions encountered in these drill holes and geologic conditions observed during field reconnaissance, it was determined that moving the alignment about 500 feet upstream would likely result in better foundation conditions. Drill Holes 15-1 and 15-4 were drilled near Site 3A, as shown on Figure I-5. All subsequent references to dam axis refer to Site 3A.

The drill holes ranged in depth from about 141 to 200 feet. Drilling and sampling was performed under the direct supervision of an engineering geologist or geotechnical engineer. A Geologic Cross Section along centerline showing the drill holes is presented in Figure I-6. The logs for these drill holes are presented in Section III – Geotechnical Data. Photos of the recovered core are also included in Section III. Photos obtained from drill holes show the bedrock to be predominately gray limestone which is highly fractured and broken with rusty staining on some joints. Drill Hole 15-4 is significantly less fractured and broken when compared to the other three drill holes. The drill holes are discussed below as follows:

**Drill Hole 15-1**  
*(Left abutment, 21 feet upstream of dam axis, elev. 5201 feet)*

This hole was drilled with a tricone bit while advancing HW casing down to a depth of 30 feet and then NW casing to 90 feet. Bedrock in the hole was cored with a 5 foot NQ core barrel. Due to the highly fractured and broken characteristics of the rock, many of the core runs were less than the full 5 feet. Bedrock consists predominately of very highly fractured and broken, gray limestone, with some very hard quartzite layers and some possible chert nodules and lenses. The Limestone appears to be about average hard rock. Many of the open fractures are coated with a rusty stain to a thin calcareous coating. A few fractures have a thin clayey coating or film. Whitish calcite and dolomite stringers and veins cut across the core and also infill healed joints.

The percent of core recovered per drilled run ranged from 20% to 76%, with an average of 37%. The RQD in bedrock was 0% as there were no core segments 3.75 inches or longer in length. The largest segments were 2 to 3 inches. There were no significant slicken-sides noted within the core recovered. The largest samples cored were within a cobble and boulder layer just above the bedrock. This layer contained gravels, cobbles and boulders of quartzite, conglomerate sandstone and volcanic rock.
The permeability of the overlying alluvial deposits ranged from 4 to 2047 ft/yr with an average of 440 ft/yr. In bedrock, permeability values ranged from 473 to 783 ft/yr with an average of 628 ft/yr. Groundwater was measured at a depth of 106.3 feet at the time of drilling.

**Drill Hole 15-2**  
(Near Max. Section, 496 feet downstream of dam axis, elev. 5105.6 feet)  
This hole was drilled not far from the existing creek and within the stream alluvium of the modern day flood plain of the creek. The overburden alluvial material consisted predominately of gravel with silty sand, with some cobbles and boulders. During drilling, the cobbles and boulders which were surrounded by a softer matrix would cause the drill rods and casing to be deflected off center, making the hole too irregular to continue drilling. In an effort to obtain a relatively straight hole down to bedrock, a Mobile B-80 drill was brought in to advance the hole using Odex drilling methods to a depth of 100 feet. The hole was then advanced with a NQ core barrel down to 184 feet.

The percent of core recovered per drilled run ranged from 10% to 100%, with an average of 50%. RQD ranged from 0 to 25% with an average of 2%. There were no significant slicken-sides noted within the core recovered.

Permeability of the overlying alluvial deposits ranged from 741 to 5388 ft/yr with an average of 3111 ft/yr. Due to the difficult drilling conditions, permeability values were not obtained in the alluvial deposits between depths of 55 to 104 feet. The bedrock permeability ranged from 296 to 7013 ft/yr with an average of 2071 ft/yr.

The groundwater level was measured at a depth of 12.7 feet below the ground surface. Temporary PVC observation wells were installed in the boring at two depths to determine if the groundwater level was perched. After monitoring for about one month, the level in both the shallow and deep observation wells measured within a few inches of each other.

**Drill Hole 15-3**  
(Base of Right Abutment, 564 feet downstream of dam axis, elev. 5114.5 feet)  
This hole was drill with a tricone bit while advancing NW casing down to a depth of about 40 feet. Bedrock in the hole was cored with a 5 foot HQ triple tube core barrel. Due
to the highly fractured and broken characteristics of the rock many of the core runs were less than the full 5 feet.

Bedrock consists predominately of very highly fractured and broken, gray limestone, with some very hard quartzite layers and some chert nodules and lenses with a hardness that ranges from about hard to extremely hard rock. The Limestone classifies as average hard rock. Many of the open fractures are coated with a rusty stain to a thin calcareous coating. A few fractures have a thin clayey coating or film. Whitish calcite and dolomite stringers and veins cut across the core and also infill healed joints. As shown on drill logs there are also some very soft clayey to shale mudstone layers. Some of the mudstone is calcareous. Some of the softer clay may be fault gouge related. The percent of core recovered per drilled run ranged from 32% to 100%, with an average of 74%. RQD ranged from 0 to 62% with an average of 11%. There were no significant slicken-sides noted within the core recovered.

Permeability values within the overlying alluvial deposits ranged from 675 to 7975 ft/yr with an average of 4604 ft/yr. In the bedrock, values ranged from 166 to 5128 ft/yr with an average of 1079 ft/yr. The groundwater was measured at a depth of 17 feet below the ground surface at the time of drilling.

**Drill Hole 15-4**  
*(Toe of Right Abutment, 65 foot upstream of dam axis, elev. 5125.1 feet)*

Due to the location of the creek and the overhead power lines, this hole was drilled about 65 feet upstream of the dam axis. The right abutment at this location consists of many limestone outcrops with bedrock dipping about 20-30 degrees upstream.

This hole was drill with a tricone bit while reaming HWT casing down to a depth of about 20 feet. Bedrock in the hole was cored with a 5 foot HQ triple tube core barrel.

Bedrock was encountered at a depth of about 12 feet. Bedrock consists predominately of limestone with some mudstone and calcareous mudstone layers. The limestone is gray and fractured to highly fractured, and appears to be average hard rock. The limestone also includes some very hard quartzite layers and some chert nodules and lenses with a hardness that ranges from hard to extremely hard rock. Many of the open fractures are coated with a rusty stain to a thin calcareous coating. A few fractures have a thin clayey coating or film. Whitish calcite and dolomite stringers and veins cut across the core and
also infill healed joints. As shown on drill logs there are some mudstone layers which range from very soft clayey to shale. Some of the mudstone layers are calcareous. Some of the softer clay may be fault gouge related.

The percent of core recovered per drilled run ranged from 60 to 100%, with an average of 96%. RQD ranged from 0 to 81% with an average of 31%. There were no significant slicken-sides noted within the core recovered.

Permeability values within the overlying alluvial deposits ranged from 423 to 4811 ft/yr with an average of 2642 ft/yr. In the bedrock, values ranged from 739 to 7972 ft/yr with an average of 3987 ft/yr. Groundwater in the hole was measured at a depth of 32 feet.

Bedrock in this hole was significantly LESS fractured and broken than the other test holes. RQD is at least three times greater than any of the other test holes. The measured permeability values in bedrock, however, are almost twice as high as observed in the drill holes with more broken up rock. One possible explanation for this discrepancy is that clay infilling was washed out during the coring of Drill Holes 15-1 through 15-3, resulting in lower per cent recovery.

Shown in Table I-1 below is a summary of the permeability values obtained in the alluvial deposits and bedrock. Table I-2 is a summary of the percent core recovery and rock quality designation (RQD) of the bedrock in each drill hole.

<table>
<thead>
<tr>
<th>DH 15-1</th>
<th>DH 15-2</th>
<th>DH 15-3</th>
<th>DH 15-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permeability</td>
<td>Overburden / Alluvium</td>
<td>Permeability</td>
<td>Bedrock</td>
</tr>
<tr>
<td>K (ft/yr)</td>
<td>K (ft/yr)</td>
<td>K (ft/yr)</td>
<td>K (ft/yr)</td>
</tr>
<tr>
<td>Average</td>
<td>440</td>
<td>Average</td>
<td>3111</td>
</tr>
<tr>
<td>MIN</td>
<td>4</td>
<td>MIN</td>
<td>741</td>
</tr>
<tr>
<td>MAX</td>
<td>2047</td>
<td>MAX</td>
<td>5388</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DH 15-1</th>
<th>DH 15-2</th>
<th>DH 15-3</th>
<th>DH 15-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permeability</td>
<td>Bedrock</td>
<td>Permeability</td>
<td>Bedrock</td>
</tr>
<tr>
<td>K (ft/yr)</td>
<td>K (ft/yr)</td>
<td>K (ft/yr)</td>
<td>K (ft/yr)</td>
</tr>
<tr>
<td>Average</td>
<td>628</td>
<td>Average</td>
<td>2071</td>
</tr>
<tr>
<td>MIN</td>
<td>473</td>
<td>MIN</td>
<td>295</td>
</tr>
<tr>
<td>MAX</td>
<td>783</td>
<td>MAX</td>
<td>7013</td>
</tr>
</tbody>
</table>
I.2.3.1 LABORATORY TESTING FOR DAM SITE DRILL HOLES

Each sample obtained in the field was 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 included in Section III, and the meaning of the various symbols, shown on the drill hole logs, can be obtained from this figure.

The results of density, moisture, classification and unconfined compressive strength tests are included on the drill hole logs. The results of all tests, with exception of the consolidation tests, are included in Table III-1, Summary of Test Data of Section III. Plots of consolidation curves are also included in Section III. The test results are summarized below as follows:

**Granular Soils**

The majority of the alluvial deposits are granular soils consisting of silty gravel with sand and cobbles and occasional boulders. Obtaining representative samples of this type of material in a drill hole is not feasible due to the diameter of the split spoon sampler, which limits particle size obtained to less than 2 inches.

Twelve classification tests were performed on split spoon samples of the granular alluvial material obtained from the drill holes. The granular soils classified as SM, SC-SM, GM, GC, GP, GP-GM and GC-GM. The percent gravel, sand and fines (minus No. 200) is shown in the following table.

<table>
<thead>
<tr>
<th>Silty Sand (SM, SC-SM)</th>
<th>Gravel w/ silt and sand (GM, GP-GM)</th>
<th>Gravel w/ clay and sand (GC, GC-GM)</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Gravel</td>
<td>% Sand</td>
<td>% Fines</td>
</tr>
<tr>
<td>19 – 42 (30)</td>
<td>35 – 42 (39)</td>
<td>16 – 46 (30)</td>
</tr>
<tr>
<td>34 – 80 (61)</td>
<td>18 – 39 (29)</td>
<td>2 – 21 (10)</td>
</tr>
<tr>
<td>27 – 64 (42)</td>
<td>11 – 33 (23)</td>
<td>25 – 48 (35)</td>
</tr>
</tbody>
</table>

**Table I-2**

(Percent Core Recovery and RQD)

<table>
<thead>
<tr>
<th>DH 15-1</th>
<th>DH 15-2</th>
<th>DH 15-3</th>
<th>DH 15-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core %</td>
<td>Recovery %</td>
<td>RQD</td>
<td>Core %</td>
</tr>
<tr>
<td>Average</td>
<td>37</td>
<td>0</td>
<td>Average</td>
</tr>
<tr>
<td>MIN</td>
<td>20</td>
<td>0</td>
<td>MIN</td>
</tr>
<tr>
<td>MAX</td>
<td>76</td>
<td>0</td>
<td>MAX</td>
</tr>
</tbody>
</table>
Cohesive Soils
A few feet of lean clay was typically encountered in the drill holes at the surface. The drill hole on the left abutment (15-1) encountered a significant clay layer between about 82 and 112 feet below the surface. Three classification tests were performed on the cohesive material. The liquid limit ranged from 25 to 30, and the plasticity index from 8 to 10.

One dimensional consolidation tests were performed on samples obtained at 80, 95 and 110 feet. The clay zone is considered to be normally consolidated with relatively low compressibility characteristics.

Bedrock
The dry unit weight, moisture content and unconfined compressive strength for core samples of the bedrock are included in Table III-1 of Section III and summarized in the following table.

<table>
<thead>
<tr>
<th>Limestone/Quartzite</th>
<th>No. of Samples Tested</th>
<th>High</th>
<th>Low</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Unit Weight (pcf)</td>
<td>20</td>
<td>167</td>
<td>153</td>
<td>162</td>
</tr>
<tr>
<td>Natural Moisture Content (%)</td>
<td>20</td>
<td>0.6</td>
<td>0.1</td>
<td>0.2</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (psi)</td>
<td>19</td>
<td>18,710</td>
<td>4,590</td>
<td>11,170</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mudstone</th>
<th>No. of Samples Tested</th>
<th>High</th>
<th>Low</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Unit Weight (pcf)</td>
<td>13</td>
<td>160</td>
<td>112</td>
<td>140</td>
</tr>
<tr>
<td>Natural Moisture Content (%)</td>
<td>13</td>
<td>16.4</td>
<td>0.7</td>
<td>5.6</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (psi)</td>
<td>12</td>
<td>9,740</td>
<td>15</td>
<td>2,781</td>
</tr>
</tbody>
</table>

1.2.4 BORROW AREAS

Shown in Figure I-7 are rough outlines of the potential borrow areas within and adjacent to the reservoir basin. Material from required foundation excavations within the dam footprint is also considered acceptable as potential borrow material for the dam embankment and cutoff. As noted previously, in addition to the four drill holes in the vicinity of the dam footprint, eight test pits were excavated in the alluvial deposits within the planned reservoir basin. The test pit logs are included in Section III – Geotechnical Data. Based upon the results of field investigations,
cohesive deposits have been identified as Zone I borrow and granular deposits have been identified as Zone II borrow on Figure I-7.

Test Pits 15-1 through 15-4 encountered 4 to 10 feet of lean clay, underlain by gravel with sand (GP) and gravel with silt and sand and occasional cobbles to the bottom of the test pits at a depth of 18 to 22 feet below the ground surface. Test Pits 15-5 through 15-7 encountered predominantly lean clay throughout the depth investigated, with the bottom of the test pits 21 to 22 feet below the surface. The locations at which bulk samples and baggie samples were obtained for laboratory classification and testing are shown on the Test Pit Logs.

As stated previously, Drill Hole 15-1 on the left abutment encountered predominantly lean clay between 82 and 112 feet. The remainder of the alluvial deposits encountered in the drill holes was predominantly granular. These materials can be used as a potential borrow source.

Potential riprap sources have been identified based on visual observation of rock outcrops in areas shown on Figure I-7.

I.2.4.1 LABORATORY TESTING
Laboratory tests were performed to define the characteristics of the proposed borrow materials. Each sample obtained in the field was classified in the laboratory according to the Unified Soil Classification System. Additional tests were performed on select samples, and the results are included on the logs and summarized in Table III-2 of Section III.

Impervious Borrow Sources
Laboratory tests completed on materials recovered from the impervious borrow source areas included (1) Atterberg Limits, (2) mechanical analysis and hydrometers, (3) soluble solids, (4) natural moisture content and in-place density, (5) moisture density relationship (proctor), (6) permeability, (7) direct shear, and (8) dispersive clay tests. Gradation, moisture density curves and direct shear plots are also included in Section III.

Atterberg Limits
Atterberg Limit tests were performed on nine samples. The liquid limit ranged from 29 to 49 with an average of 35. The plasticity index varied from 13 to 31 with an average of 18. The samples classify as CL (lean clay).
Mechanical Analyses & Hydrometers
Mechanical analyses tests were performed on nine samples with hydrometer tests on seven of the samples. The percent silt and clay ranged from 29 to 89, with an average of 65%. The percent finer than 0.005 mm ranged from 16.2 to 46, with an average of 29%.

Natural Moisture Content & In-Place Density
The dry unit weight was determined on three samples with results of 108.3, 111.3 and 100.1 pcf. The natural moisture content was determined for 15 samples, with the results varying from 7.5 to 17.8%, with an average of 12%.

Soil Moisture Density Relationship (Proctors)
Two proctor tests were performed in accordance with ASTM D 698 (standard proctor). Maximum dry densities of 109.1 and 107.2 pcf, with optimum moisture contents of 14.7 and 17.6%.

Permeability
A constant head permeability tests was performed on the sample of lean clay obtained at a depth of 5 feet in Test Pit 15-07. The sample was compacted into the permeameter mold at 95% of the maximum density and allowed to saturate prior to testing. The test resulted in a permeability of 1.12 x10^-6 cm/sec (1.15 ft/yr).

Direct Shear
Consolidated drained direct shear tests were performed in samples of the lean clay obtained from depths of 5 feet and 15 to 22 feet in Test Pit 15-07. The samples were compacted into the molds at about 95% of the maximum laboratory density. Friction angles of 33.0 and 31.9 degrees were obtained, with 1 psi cohesion.

Water Soluble Solids
Water soluble solid tests were performed in accordance with USBR 5450-89 on samples of the lean clay obtained at a depth of 3 feet in Test Pit 15-03 and at 5 feet in Test Pit 15-07. The tests resulted in 0.88 and 0.66% water soluble solids.

Pinhole (Dispersive Clay)
Pinhole tests were performed in accordance with ASTM D 4647 on three samples of the lean clay soil. All three samples were non-dispersive, with two of the
samples classifying as ND1 (Test Pit 15-01 at 6’, and Test Pit 15-06 at 6’), and one sample classifying as ND2 (Test Pit 15-07 at 5’).

**Granular Borrow Sources**

In addition to testing performed on the granular soils from the drill holes, tests were performed on five bulk samples of material obtained from the test pits.

**Mechanical Analyses**

Mechanical analyses tests were performed on five bulk samples. The percent gravel ranged from 64 to 76, with an average of 69%. The percent sand varied from 18 to 30, with an average of 23%. The percent silt ranged from 3 to 18, with an average of 8%. Four of the five samples had less than 8% fines. It was estimated that 1 to 3% of the granular borrow area contained boulders greater than 12 inches in diameter.

**Soil Moisture Density Relationship (Proctors)**

One proctor test was performed in accordance with ASTM D 698 (standard proctor) on the sample obtained from Test Pit 15-02 from 10 to 14 feet. The test resulted in a maximum dry density of 137.7 pcf, with optimum moisture content of 7.7%.

**Permeability**

A constant head permeability tests was performed on the sample obtained from Test Pit 15-02 compacted into the permeameter mold at 95% of the maximum density and saturated prior to testing. The test resulted in a permeability of $2.80 \times 10^{-5}$ cm/sec (29.0 ft/yr).

**I.3 SUMMARY AND CONCLUSIONS**

**I.3.1 SURFICIAL DEPOSITS AT DAM SITE**

The soil foundation of the proposed dam site has a complex depositional history resulting from the fluctuating levels of ancient Lake Bonneville, erosion and deposition from Peteetneet Creek, and slopewash and debris flow deposition from the canyon slopes and ravines. These fluvial/alluvial (and possibly lacustrine) deposits inter-finger across the foundation with the deposits being significantly thicker on the left or west side of the canyon than on the east side.
Data from the limited exploration program shows that the thickness of surficial deposits ranges from about 15 feet thick near the toe of the right abutment, about 90 feet thick in the valley center, and about 125 feet thick near the left abutment. See Figure I-6. Because of the limited number of drill holes and their wide spacing, there is considerable uncertainty in the interpreted bedrock surface. As mentioned in the initial Geologic Assessment, a deeply incised bedrock channel cannot be ruled out- and additional, more closely spaced drill holes are required to reduce this uncertainty.

Standard Penetration Tests (SPT) were generally performed about every 5 feet in each of the drill holes. Recovered samples included silty sand (SM), lean clay (CL), clayey gravel (GC), and gravel with silt and sand (GP-GM). Generally, liquefiable materials are loose, non-cohesive materials composed of silts, sands, and gravels deposited as described in the preceding paragraph. Ruling out liquefaction potential requires proof that a soil is unsaturated or sufficiently dense or sufficiently clayey that liquefaction would not occur with the earthquake being considered. These deposits should be considered saturated below the elevation of the creek.

Based upon soil classification only, some of these deposits would generally be considered potentially liquefiable given the expected earthquake loading. However, examination of the uncorrected blow counts for some of the silty sand layers showed blow counts greater than 25 for the 1 foot test interval indicating higher density and higher resistance to liquefaction. As expected, gravel-bearing layers generally exhibited refusal (greater than 50 blows per foot) due to the sampling tool striking gravel and the resulting increased resistance of driving the gravel through the test interval. Interpretation of SPT results can be quite complex when the test conditions differ from standard conditions (clean sand) – because results are strongly influenced by the presence of gravel. Additional testing will be required, however, to determine if the gravel is matrix (sand/silt) supported, in which case the layers could be potentially liquefiable.

Measuring/plotting of penetration per blow during SPT’s and other corroborating tests (shear-wave velocities, BPT) are required to determine the impact of gravel on the test results and to reduce the uncertainty as to how the foundation layers will behave during an earthquake. Without a large test trench, correlation of individual layers or lenses from abutment to abutment can only be inferred from the limited tests. The drill holes are too widely spaced and the SPT’s too few to reach any conclusion about the lateral extent of potentially liquefiable layers, if they exist. It should be noted that a single continuous weak layer could govern the strength of the
foundation as a whole. Extensive liquefiable layers or lenses could require treatment or removal during dam construction.

As drilling progressed through the surficial deposits, gravity-flow (no pressure) permeability tests were conducted. As expected, water loses and permeability ranged from very low (CL, GC) to high (GP-GM). A cutoff will be required to reduce water losses through the surficial deposits.

I.3.2 BEDROCK AT DAM SITE

As expected, recovered bedrock was primarily limestone with minor layers or partings of quartzite and mudstone. Rock quality across the proposed dam site is highly variable. With the exception of Drill Hole 15-4, rock was intensely fractured to brecciated and low core recovery was typical. Although the fractured nature of the rock recovered from Drill Holes 15-1 and 15-2 would indicate a highly permeable foundation, this was not always the case. Permeabilities in 15-1 ranged from 473 to 783 feet/year, from 43 to 4493 feet/year (near top of rock) in 15-2, and from 166 to 5128 feet/year in 15-3. Packer permeability tests were generally performed in 3 pressure stages to determine if increased pressure/flow changed permeability by opening fractures or by flushing out joint in-fillings. The tests indicate that this did not occur. It is our interpretation given the low core recovery and the less-than-expected permeability in Drill Holes 15-1, 15-2, 15-3 that the drilling fluid washed away the clayey matrix (fault gouge?) from the fractured rock. Although it is reasonable to assume that the brecciation of the rock in these three drill holes is the result of faulting, only infrequent, minor slickensides were identified in the core samples.

Drill Hole 15-4 recovered much higher quality rock and core recovery was frequently 100%. Frequent mineral stained, high angle joints were present and as a result, permeability was generally much higher than in the other three drill holes. Permeabilities ranged from 263 feet/year (shaley mudstone) to 7,972 feet/year (possible void). Permeability tests did not show that the bedrock became “tighter” within the depth investigated. Grout takes could potentially be high for most of the depth of the grout curtain.

It was noted during detailed examination of the right abutment outcrops that the strike and dip of bedding planes changes between Drill Holes 15-3 and 15-4. Near 15-3 bedrock dips down between 15 to 20 degrees toward the east and northeast. At 15-4 bedrock dips down between 15 to 20 degrees toward the south and southeast. At both locations joint spacing ranges from about 6 inches to 2 feet. An east-west trending ravine, downstream and parallel to the proposed dam
axis, separates these two drill holes. It is postulated that a high angle, normal fault may be present in this ravine and projects toward a ravine present on the west canyon wall. See Figure I-5. Three similar faults located down the canyon are shown on Figure I-2. The fracturing of the rock in Drill Holes 15-1, 15-2, and 15-3 is believed to be the result of the thrust fault the length of Payson Canyon and this potential cross canyon normal fault. Because Drill Hole 15-4 showed no evidence of faulting, it is our interpretation that the Payson Canyon thrust fault lies to the west or left of the drill hole. The location of the fault and the boundary between the less fractured, highly permeable rock and the fractured, less permeable rock cannot be determined with any certainty without additional, more closely spaced drill holes drilled to greater depths.

LiDAR imagery obtained since the Phase I Geologic Assessment shows a roughly north-south trending lineament or “feature” which parallels the west side of the canyon at the toe of the slope. It extends from several hundred feet downstream of the proposed axis, upstream to the gas plant. See Figures I-5 and I-5a. A segment of this feature is found immediately left (west) of Drill Hole 15-1 where the slope abruptly rises 20-25 feet.

No outcrops were identified along the left abutment alignment nor in the immediately vicinity of this feature. The closest outcrops to the left end of the proposed dam are approximately 300 feet west. See Figure I-5. Surficial talus deposits show a mixture of sandstone, limestone and quartzite. During site reconnaissance observed bedrock outcrops consisted of sandstone with some quartzite. Bedrock had a strike of about N 35-40 degrees W, with a dip of about 25-27 degrees down to the northeast. Some additional strike and dips taken further to the northwest and not shown on the map ranged from N 55 degrees E, dipping 10 degrees south, and N 10 degrees W, and dip 30 degrees south. Joint spacing varies from about 6 inches to 3 feet. Due to the limited bedrock exposures on the left abutment, it was not possible to draw meaningful conclusions based on the strike and dip of the isolated exposures.

Heavy vegetation obscures the feature making field observation or identification on aerial photographs difficult. However, this geologic feature is readily apparent on LiDAR imagery. On the imagery, the feature appears to cross or offset surficial deposits giving it the appearance of a fault scarp. A river terrace could also have a similar trend and appearance. Furthermore, individual lineament segments are at differing angles, more indicative of terrace deposits. Additionally, an alluvial fan immediately upstream of the proposed axis separates two segments of this feature and exhibits no offset. Without additional more detailed investigations which could possibly include trenching at right angles across the feature, the interpretation of this feature and the potential affect it may have on the project cannot be made with certainty.
I.3.3 SEISMIC CONSIDERATIONS AT DAM SITE

The site is located within a transition zone where there is overlapping between the Provo and Nephi Segments of the Wasatch Fault zone. The canyon in which the site is located has also been mapped as an older thrust fault, where older bedrock from the west has been pushed up and over top of younger bedrock to the east. This faulting predates the younger potential active normal faulting of the Basin and Range, with which the Wasatch Fault zone is associated. While the thrust fault has significantly fractured bedrock in this area, it is not considered a potential active fault.

The site is located nearest the Provo segment of the Wasatch Fault where the projected westerly dip of the fault would extend beneath the proposed reservoir. With the Provo segment located about 1 mile south of the dam site, the Provo segment will be the controlling seismic source for this site. The Provo segment is capable of generating a 7.3 magnitude earthquake. Further discussion of the seismic parameters for dam design are included in Section II.

I.3.4 BORROW SOURCES

Potential borrow areas evaluated during this feasibility study included materials from required excavations within the dam footprint and materials within or adjacent to the reservoir basin. Based on the results of field and laboratory investigations, it is estimated that materials from these sources include the following approximate quantities:

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Estimated Quantity (cu yd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impervious (lean clay)</td>
<td>1.1 Million</td>
</tr>
<tr>
<td>Sandy Gravel to Silty Sandy Gravel</td>
<td>3.1 Million</td>
</tr>
<tr>
<td>Riprap</td>
<td>0.2 Million</td>
</tr>
</tbody>
</table>

Daniel E. Grundvig, P.G.
Professional Geologist-Utah 553392-22500

Bradford E. Price, P.E.
Professional Engineer-Utah 162291-2203
Figure I-1  VICINITY MAP
Payson City Reservoir Feasibility Study
Payson, Utah County, Utah
Figure I-2

Geologic Map of the Spanish Fork Quadrangle
Utah County, Utah, Map 227
Utah Geological Survey, 2007

Payson Canyon Thrust Fault
Figure I-5a
LiDAR Image (2013-2014 at 0.5 meters)
Project
Payson City Reservoir, Dam Site
Location
Utah County, Utah

Approx. dam alignment
Surficial lineament