DRAFT GEOTECHNICAL ENGINEERING REPORT

PROPOSED FISH SPRINGS WATER SUPPLY PROJECT
HONEY LAKE VALLEY TO LEMMON VALLEY
WASHOE COUNTY, NEVADA

Project Number: 67055025
June 7, 2006

Prepared for:

ECO:LOGIC
ECO:LOGIC ENGINEERING
Reno, Nevada

Prepared by:

Terracon
Reno, Nevada

Terracon

June 7, 2006
Terracon Project No.: 67055025

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Re: DRAFT GEOTECHNICAL ENGINEERING REPORT
PROPOSED FISH SPRINGS WATER SUPPLY PROJECT
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WASHOE COUNTY, NEVADA

Dear Mr. Kershaw:

We are submitting, herewith, the results of our geotechnical exploration performed for the proposed Fish Springs Water Supply Project with a pipeline alignment from Honey Lake Valley to Lemmon Valley in Washoe County, Nevada. The purpose of this geotechnical exploration was to obtain information on the subsurface conditions at the proposed project site and, based on this information, to provide recommendations regarding the design and construction of the pipeline alignments and structures associated with the project. Each of the main components of the project is described separately in this draft report. As geotechnical exploration was not possible at some portions along the Well Field Collection System and Main Transmission Line alignments, some information and related recommendations have been omitted from this draft report.

Other design and construction recommendations, based upon geotechnical conditions, are presented in the report.

We appreciate being of service to you in the geotechnical engineering phase of this project, and are prepared to assist you during the construction phases as well. If you have any questions concerning this report or any of our testing, inspection, design and consulting services, please do not hesitate to contact us.

Sincerely,
Terracon

PROFESSIONAL ENGINEER - STATE OF NEVADA
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Steve D. Bowman, Ph.D., P.E., LEED® AP
Geotechnical Department Manager

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DRAFT GEOTECHNICAL ENGINEERING REPORT

PROPOSED FISH SPRINGS WATER SUPPLY PROJECT
HONEY LAKE VALLEY TO LEMMON VALLEY
WASHOE COUNTY, NEVADA

June 7, 2006

1.0 INTRODUCTION

This draft report presents the results of the subsurface exploration and geotechnical engineering services performed for the proposed Vidler Water Company (Vidler) Fish Springs Water Supply Project to be located between Honey Lake Valley and Lemmon Valley in Washoe County, Nevada. The purpose of this draft report is to describe the surface conditions observed from geotechnical mapping, subsurface conditions encountered in the exploratory borings and test pits, analyze the data obtained, and provide recommendations regarding the design and construction of the Well Field Collection System Alignment, Well House Structures, Pump Station and Pump Station Tanks, Main Transmission Line Alignment, Surge Tank, Terminal Water Storage Tank, and to provide general access road geotechnical recommendations.

Due to a variety of permitting and environmental related issues, geotechnical exploration has been performed in several phases. As field activities are currently suspended by the Bureau of Land Management (BLM), we are unable to complete the project geotechnical exploration. Planned further exploration consists of test pits and fault trenches along the Main Transmission Line alignment and a fault trench along a portion of the Well Field Collection System alignment between Fish Springs Road and Well 5.

Geologic reconnaissance and engineering geology mapping was performed along the Well Field Collection System and Main Transmission alignments. This mapping was performed to identify geotechnical soil and bedrock units and areas that may pose constraints to project design and construction and to provide detailed geologic mapping along the alignment that was not available from the Nevada Bureau of Mines and Geology (NBMG) or the U. S. Geological Survey (USGS).

A total of 8 borings and 109 test pits were drilled or excavated as part of our geotechnical exploration to date. Detailed geotechnical alignment maps, and boring and test pit logs are presented in **Appendix A.** Laboratory testing results are presented in **Appendix B**.

2.0 PROJECT DESCRIPTION

It is our understanding that the project will consist of the following integrated components:

- Approximately 10 miles of underground Well Field Collection System ranging 12 inches to 30 inches in diameter conyeying water from 6 Water Supply Wells to the Pump Station and Pump Station Tanks at the southern end of Honey Lake Valley;

- 6 Well House Structures containing one Water Supply Well each within the Fish Springs Ranch property in Honey Lake Valley;

- Pump Station with 5 high pressure water pumps, two 2,500-gallon sodium hypochlorite solution tanks, a 1,570-cubic foot (ft^2) Surge Tank, associated electrical and plumbing equipment, and an internal employee restroom in Honey Lake Valley;

- Two 500,000-gallon Pump Station Water Storage Tanks in Honey Lake Valley;

- Approximately 28 miles of 30-inch diameter underground Main Transmission Line pipe between the Pump Station in Honey Lake Valley and the Terminal Water Storage Tank between Antelope Valley and Lemmon Valley.

- A 200,000-gallon Surge Tank located near the high point along the Main Transmission Line alignment within the Fort Sage Mountains; and,

- A 1-million gallon Terminal Water Storage Tank located between Antelope Valley and Lemmon Valley.

Information related to the project was obtained from Mr. David Kershaw, ECO:LOGIC Engineering of Reno, Nevada; from a document entitled *Fish Springs Water Supply Project, Washoe County Regional Water Planning Commission, Water Facility Plan* from Vidler Water Company/ECO:LOGIC Engineering (ECO:LOGIC) of Carson City, Nevada dated September 2005; and from numerous discussions with ECO:LOGIC personnel.

Each of the main components of the project is described separately in this report for clarity. Each section is separated by a tab page for quick access. In addition, the exploration locations have been coded by the project main components for quick access as shown in Table 1.

Table 1 – Exploration Location Naming Convention

Project Component		Naming Convention
Well Field Collection System / Well Houses		WTP-xx
Pump Station	*Test Pits*	PTP-xx
	Borings	B-xx
Main Transmission Line		TP-xx
Surge Tank		STP-xx
Terminal Tank		TTTP-xx
Access Road		RTP-xx

3.0 WELL FIELD COLLECTION SYSTEM ALIGNMENT

The Well Field Collection System alignment consists of a transmission main with water laterals stubbed out to six proposed well sites with an approximate length of 10 miles from an area northeast of the intersection of Fish Springs Road and Flanigan Road to the proposed Pump Station to the southwest. It is anticipated that the Well Field Collection Pipe will have a diameter of between 12 inches and 30 inches in diameter and will have a minimum burial depth of approximately 3 feet. It is our understanding that a portion of the proposed alignment near the Honey Lake Valley playa has been moved to the east to reduce the impact to butterflies.

Geotechnical exploration has not been completed along the alignment from Test Pits WTP-06 to WTP-08 and WTP-11 to WTP-14, due to on-going environmental clearance issues. As a result, sub-surface soil, bedrock, and groundwater conditions along this portion of the alignment and the portion of the moved alignment described above is unknown.

The Well Field Collection System Alignment is located within Sections 25, 26, and 33 to 35, Township 26 North, Range 18 East, M.D.M and Sections 9, 10, 16, 17, 20, 29, and 30 Township 26 North, Range 19 East, M.D.M.

3.1 SITE EXPLORATION PROCEDURES

3.1.1 Geologic Reconnaissance

Geologic reconnaissance was performed in November, 2005 to provide alignment specific engineering geologic maps along the proposed Well Field Collection System alignment. Available geologic maps for the proposed alignment do not well indicate surficial soils and as a result, are not well suited for engineering and construction recommendations. In addition, as the best available geologic map for the portion of the proposed alignment from Well 1 to just north of the Fish Springs Ranch is mapped at a scale of 1:250,000, more detailed geologic mapping was necessary. Engineering geologic mapping of the proposed alignment is shown on Plates 1a to 1c and presented in **Appendix A**. Table 2 summarizes the estimated depth to bedrock results from our geologic reconnaissance for areas where bedrock is estimated to be less than 8 feet below the existing ground surface. Bedrock is not expected in the unexplored area near Test Pits WTP-06 to WTP-08, due to the location near the Honey Lake Valley playa and lack of encountered bedrock in nearby test pits.

Table 2 – Estimated Depth to Bedrock Reconnaissance Summary

Stationing	Estimated Depth (ft)
182+00 to 189+00	2 - 8
321+00 to 325+00	2 - 8

3.2 SITE AND SUBSURFACE CONDITIONS

3.2.1 Site Conditions

The Well Field Collection System alignment consists of a transmission main with water laterals stubbed out to six proposed well sites with an approximate length of 10 miles from an area northeast of the intersection of Fish Springs Road and Flanigan Road to the proposed Pump Station to the southwest.

At the time of our geotechnical exploration, the alignment from approximately Station 0+00 to Station 41+00 consisted of fallow center-pivot irrigated agricultural fields. From approximately Station 41+00 to Station 130+00, the alignment traverses undeveloped land consisting of sagebrush and grasses to approximately 4 feet in height. From approximately Station 130+00 to Station 224+00, the alignment follows along the western edge of the existing partially-gravel surfaced Fish Springs Road and crosses the road near the end of this alignment section. The alignment then continues south and west from approximately Station 224+00 to Station 310+00 traversing agricultural fields and other undeveloped land. A pipe lateral extends approximately from Station 234+00 south to the proposed Well 5 site and traverses the edge of existing agricultural fields. From approximately Station 310+00 to the proposed Pump Station near Station 486+15, the alignment traverses alluvial fans and undeveloped land. Several natural drainages are crossed by the alignment, including Fish Springs Creek. An additional pipe lateral extends north and east from approximately Station 319+00 to the proposed Well 6 across existing agricultural fields and undeveloped land. This alignment also crosses the existing Fish Springs Road.

The proposed Well Field Collection System alignment is accessed from Fish Springs Road, Flanigan Road, and by numerous unnamed unimproved access roads leading from Fish Springs Road.

3.2.2 General Geologic and Soil/Bedrock Conditions

The site lies within the Honey Lake Valley generally north of the Virginia Mountains and the Fort Sage Mountains within the Basin and Range geomorphic province. The Basin and Range geomorphic province is characterized by horizontal extension between the Sierra Nevada Range to the west and the Wasatch Front to the east and is bounded by the Colorado Plateau to the southeast and the Columbia Plateau to the north. This portion of the Honey Lake Valley near the base of the Virginia and Fort Sage Mountains is generally characterized by alluvial fan deposits from the Virginia and Fort Sage Mountains. Most of the coarse-grained sediments were deposited quickly during the post-glacial period during flooding events. As a milder climate developed in the region, the volume and size of the sediments were greatly reduced, and geomorphic processes generally changed into reworking of earlier deposited materials.

Geologic reconnaissance performed by Terracon mapped the proposed alignment based on a literature review of existing geologic maps and reports and from field reconnaissance along the proposed alignment.

3.1.2 Field Exploration

The proposed Well Field Collection System alignment was explored on December 7-9 and 22, 2005 and April 11-12, 2006 by excavating 30 test pits and drilling one boring at the approximate locations indicated on the Site Plans presented in **Appendix A**. Terracon established the test pit and boring locations in the field by taping from the available reference features and by the use of a hand-held WAAS enabled GPS unit. The test pit and boring locations should be considered accurate only to the degree implied by the methods used to define them. Thirty test pits were excavated along the proposed well field pipe alignment including 6 test pits that also coincide with proposed Well House Structures and one boring was drilled along the proposed Well Field Collection System alignment for subsequent liquefaction analysis. The test pits were excavated with a either a Cat 314C or Case 330C trackhoe. The boring was advanced with a balloon-tire CME-750 drill rig, utilizing 6-inch outside-diameter, continuous-flight hollow-stem augers.

During test pit excavation, each major geotechnical soil unit was sampled with samples placed in appropriate sealed containers.

During drilling, the subsurface soils were sampled by using a standard 2-inch outside diameter split-spoon sampler or a Modified California 3-inch outside diameter split-spoon sampler driven by a cathead-type 140-pound hammer with a 30-inch drop using Standard Penetration Testing (SPT). The SPT test is an indication of the density (coarse-grained soils) or consistency (fine-grained soils) of the subsurface soil materials and is defined as the number of blows required to drive the sampler the final 12 inches of an 18-inch total penetration which may be correlated to other soil properties. Penetration values recorded from the Modified California Sampler have not been corrected to the standard SPT N-values.

An engineering geologist classified the site soils in the field in general accordance with ASTM D2488. Samples were then returned to our Sparks, Nevada laboratory, and classifications were confirmed or revised based on laboratory testing of selected samples. The descriptions of the soils indicated on the boring and test pit logs are in accordance with the enclosed General Notes and the Unified Soil Classification System (USCS). Classification in this manner provides indications of soil properties and can be correlated to other properties using published charts (Bowles, 1995). Estimated group symbols according to the USCS are given on the test pit and boring logs, and a brief description of USCS classification system is shown in **Appendix C**.

3.2.2.1 Nevada Bureau of Mines and Geology (NBMG) Geologic Mapping

The Nevada Bureau of Mines and Geology (NBMG) has mapped the proposed alignment from the Test Pit WTP-01 and extending to approximately Test Pit WTP-13 as Quaternary lake deposits. The Quaternary lake deposits are described as *lake deposits, clay, silt, sand, gravel and calcareous tufa. Includes some areas thinly veneered as eolian sand* (Bonham, 1969).

From approximately Test Pit WTP-13 and extending through approximately Test Pit WTP-24, the alignment crosses an area mapped as Quaternary fine gravel and sand alluvium. The Quaternary fine gravel and sand alluvium is described as *coalescing alluvial fans form bajadas that merge with sand and clay deposits of the floor of Honey Lake Valley. The alluvium is composed of gravel- and sand-sized clasts of quartz, feldspar, and volcanic rocks derived from the Fort Sage and Virginia Mountains to the south* (Grose, 1984).

From approximately Test Pit WTP-24 and continuing southwest towards the Pump Station, the proposed alignment crosses an area mapped as Quaternary Lahontan Lake beds. The Quaternary Lahontan Lake beds are described as *exposed near-shore deposits of the late Pleistocene pluvial Lake Lahontan include sand, silt, clay, ash, and tufa* (Grose, 1984). Immediately east of the Pump Station, the proposed alignment crosses back into an area mapped as Quaternary fine gravel and sand alluvium.

3.2.2.2 Geologic Reconnaissance Mapping

Geologic reconnaissance performed by Terracon mapped the proposed alignment based on a literature review of existing geologic maps and reports and from field reconnaissance along the proposed alignment.

The portion of the alignment starting at Test Pit WTP-01 and extending past Test Pit WTP-07 to approximately Station 104+00 was mapped as Quaternary alluvium. The Quaternary alluvium deposits are described as *light brown to yellowish-brown, silty sand and gravel. Generally non to weakly cemented, easily excavatable. Composed of sheet wash, stream channel and other alluvial deposits.*

The portion of the proposed alignment from approximately Station 104+00 and extending through approximately Station 110+00 was mapped as Quaternary playa deposits. The Quaternary playa deposits are described as *light-brown to brown, moderately well sorted, slightly sandy to granular mud with interbedded fine sand and silt.*

The portion of the proposed alignment from approximately Station 110+00 to approximately Station 182+00 and from approximately Station 189+00 to Station 309+00 was mapped as

Quaternary alluvium. The Quaternary alluvium deposits were described as *light brown to yellowish-brown, silty sand and gravel. Generally non to weakly cemented, easily excavatable. Composed of sheet wash, stream channel and other alluvial deposits.* A small portion of the alignment, from approximately Station 182+00 to Station 189+00 and near Station 322+00 crosses an area mapped as Quaternary tufa. Bedrock in this area is relatively shallow, and is covered by 2 to 8 feet of alluvium.

The portion of the alignment from approximately Station 309+00 to the proposed Pump Station (and excluding a small portion near Station 322+00) was mapped as Quaternary fan gravels and have described them as *brown to grayish-brown, silty, gravelly sand and sandy gravel. Similar to Quaternary Fan deposits but with an abundance of coarse sand, gravel, cobbles, and boulders; generally non to weakly cemented.*

3.2.3 Faulting and Seismicity

Nevada ranks as the fourth highest state of earthquake occurrence of Magnitude 3.5 and greater, behind Alaska, California, and Hawaii (USGS, 2005). In addition, the Warm Springs Fault Zone, generally consisting of a main fault trace and a number of secondary faults, or splays, is one of the top five most seismically active regions of Nevada (dePolo, 2006, personal communication). Additionally, dePolo states that the Warm Springs Fault Zone has had six seismic events prior to 10,000 years ago and two seismic events within the last 10,000 years (Holocene). He states that a large event can be expected approximately every 8,000 years and a small event approximately every 2,000 years. In addition, the main trace of the Warm Springs Fault Zone can be expected to move 6 to 8 feet during an event, and current geodetic information suggests that the region is overdue for an event. The last major event on the Warm Springs Fault Zone occurred 9,000 years ago.

dePolo et al., in a study of earthquake occurrence in the Reno-Carson City urban corridor, state that *overall, the probabilities of potentially damaging earthquakes within the region are relatively high and are commensurate with many parts of California, a state with a well-recognized high earthquake hazard* (dePolo et al., 1997). These probabilities are also valid for the area encompassed by this project. As a result of seismicity present, the project should be designed accordingly.

Guidelines for determining and assessing faults have been developed by the Nevada Earthquake Safety Council (NESC, 1998); however, neither the State of Nevada nor various counties have adopted the guidelines. These NESC guidelines are generally consistent with those adopted in the State of California Alquist-Priolo Act of 1972 that defines active faults as those faults with displacement within the past 11,000 years (Holocene) and potentially active faults as those faults with displacement within the past 11,000 to

2,000,000 years. Based on the geologic mapping, faults in the vicinity of the project site are considered active.

The published and available geologic maps indicate that a single fault (Quaternary or older) is mapped as crossing the pipe lateral extending to Well 5, and 3 faults are within 1 mile of the proposed alignment near the proposed Pump Station (Grose, 1984; dePolo, 2006). dePolo (2006) has mapped the three faults near the proposed Pump Station to be Late Quaternary in age and part of the Warm Springs Fault Zone. These three faults are considered to be active.

Once final permitting and other environmental issues are resolved, a fault trench along the proposed alignment between Fish Springs Road and Well 5 will be performed in an attempt to locate and possibly date the mapped fault. As the mapped fault crosses Quaternary deposits, the fault should be considered active to potentially active.

3.2.4 Flooding

To determine relative flooding hazard, Terracon referenced the available Federal Emergency Management Agency (FEMA) FIRM maps. The referenced flooding data is provided for informational purposes only. Since flooding and site drainage are not the responsibility of Terracon, we recommend that the site Civil Engineer be consulted concerning flooding and other drainage hazards.

FEMA has mapped the proposed Well Field Collection System alignment as of 1994 as lying within unshaded Zone X, and is defined as *areas determined to be outside [the] 500-year floodplain* (FEMA, 1994). However, the alignment generally lies on alluvial fans of the Virginia and Fort Sage Mountains which may experience shallow sheet flow during rain and/or snow precipitation events. In addition, several emphreally active drainage channels are present crossing the proposed alignment. Flow from overland sheet flow and within the existing, natural drainage channels will need to be adequately routed across the proposed Well Field Collection System alignment. Drainage modifications should include necessary design elements and/or surface treatments to reduce erosion hazard, due to the potentially highly erodible surface soils along the proposed alignment.

3.2.5 Subsurface Conditions

As presented on the boring and test pit logs, the subsurface soils along the proposed Well Field Collection System alignment generally consist of *Silty Sand (SM)* to *Poorly Graded Sand with Silt (SP-SM)* granular soils. Fine-grained *Elastic Silt with Sand (MH)* to *Lean Clay (CL)* soils are also present along the alignment. Subsurface conditions encountered during our geotechnical exploration along the proposed alignment are summarized in Table 3 and described in more detail below.

NBMG. This material is underlain by *Sandy Lean Clay with Gravel (CL)* and *Silty Sand (SM)* to the depth explored.

The subsurface soils from Test Pits WTP-03 through WTP-05 generally consist of medium dense *Silty Sand (SM)* and *Silty Sand with Gravel (SM)* from the existing ground surface to approximately 12 feet below the surface and generally correspond with that mapped by the NBMG.

The subsurface soils at Boring WTP-09 (WB-09) generally consist of medium dense *Silty Sand with Gravel (SM)* from the existing ground surface to approximately 7.5 feet below the surface. This material is underlain by *Elastic Silt with Sand (MH)*, *Silty Sand (SM)*, *Sandy Elastic Silt (MH)*, and *Clayey Sand (SC)* to approximately 35 feet beneath the existing ground surface. These materials are underlain by *Poorly-Graded Sand with Silt (SP-SM)*, *Clayey Sand (SC)*, and *Silty Sand with Gravel (SM)* to the depth explored.

The subsurface soils from Test Pits WTP-10 through WTP-19 generally consist of medium dense *Silty Sand (SM)*, *Silty Sand with Gravel (SM)*, and *Sandy Silt (ML)* from the existing ground surface to approximately 12 feet below the surface. This material is underlain by *Poorly Graded Sand (SP)* and *Poorly Graded Sand with Gravel (SP)* to the depths explored.

The subsurface soils from Test Pits WTP-20 through WTP-22 generally consist of medium dense to very dense *Silty Sand (SM)* and *Silty Sand with Gravel (SM)* from the existing ground surface to approximately 4.5 to 6 feet below the surface. This material is underlain by *Elastic Silt with Sand (MH)*, *Clayey Sand (SC)*, and *Clayey Sand with Gravel (SC)* to the depths explored.

The subsurface soils from Test Pits WTP-23 through WTP-28 generally consist of stiff to very stiff, medium to high plasticity *Lean Clay with Sand (CL)*, *Lean Clay (CL)*, and *Elastic Silt (MH)* and minor *Clayey Sand (SC)* from the existing ground surface to approximately 15 feet below the surface. *Silty Sand (SM)* in lenses of less than 1 foot in thickness was encountered in Test Pits WTP-24 through WTP-26.

The subsurface soils from Test Pits WTP-29 through WTP-34 generally consist of medium dense *Silty Sand (SM)* and *Poorly Graded Sand with Silt (SP-SM)* from the existing ground surface to approximately 12 feet below the surface.

The subsurface soils from Test Pit WTP-35 generally consist of stiff, high plasticity *Sandy Fat Clay with Gravel (CH)* from the ground surface to 3 feet below the surface. The clay soils encountered were generally moist from the existing grades to the refusal depths. Excavator refusal was experienced in Test Pit WTP-35 due to cobbles and/or boulders encountered at depth.

Table 3 – Well Field Collection System Subsurface Conditions Summary

Location	USCS Soil Type at Approximately 6 Feet	Exploration Depth (ft)	GWT (ft)	Excavation Refusal (ft)
WTP-01	Silty Sand (SM)	12	NE	NE
WTP-02	Poorly Graded Sand with Silt (SP-SM)	12	NE	NE
WTP-03	Silty Sand (SM)	12	NE	NE
WTP-04	Silty Sand with Gravel (SM)	12	NE	NE
WTP-05	Silty Sand (SM)	12	NE	NE
WTP-09 (B-9)	Clayey Sand (SC)	48	8	NE
WTP-10	Silty Sand (SM)	12	NE	NE
WTP-15	Silty Sand with Gravel (SM)	14	NE	NE
WTP-16	Silty Sand (SM)	12.3	NE	NE
WTP-17	Silt with Sand (ML)	13.2	NE	NE
WTP-18	Silty Sand (SM)	13	NE	NE
WTP-19	Poorly Graded Sand (SP)	13.2	NE	NE
WTP-20	Elastic Silt with Sand (MH)	10.5	10.5	NE
WTP-21	Silty Sand (SM) / Clayey Sand (SC)	12	6	NE
WTP-22	Clayey Sand (SC)	12	NE	NE
WTP-23	Clayey Sand (SC)	15.5	13	NE
WTP-24	Lean Clay (CL)	15.5	NE	NE
WTP-25	Lean Clay (CL)	15	NE	NE
WTP-26	Silt with Sand (ML)	16	14	NE
WTP-27	Lean Clay (CL)	15	11	NE
WTP-28	Elastic Silt (MH)	16	12.5	NE
WTP-29	Poorly Graded Sand with Silt (SP-SM)	12	NE	NE
WTP-30	Silty Sand (SM)	12	NE	NE
WTP-31	Silty Sand (SM)	11.5	NE	NE
WTP-32	Silty Sand (SM)	12	NE	NE
WTP-33	Silty Sand (SM)	11.5	NE	NE
WTP-34	Poorly Graded Sand with Silt (SP-SM)	12.5	NE	NE
WTP-35	---	3	NE	3 ft - Boulders
WTP-36	Silty Sand with Gravel (SM) / Boulders	6	NE	6 ft - Boulders
WTP-37	Silty Sand with Gravel (SM)	11.5	NE	NE
WTP-38	Silty Sand with Gravel (SM)	11	NE	NE

Notes: NE – Not Encountered.

The subsurface soils at Test Pit WTP-01 generally consist of medium dense *Silty Sand (SM)* from the existing ground surface to approximately 10 feet below the surface and generally correspond with that mapped by the NBMG. This material is underlain by *Poorly Graded Sand with Silt and Gravel (SP-SM)* to the depth explored.

The subsurface soils at Test Pit WTP-02 generally consist of medium dense *Silty Sand (SM)* and *Poorly Graded Sand with Silt (SP-SM)* from the existing ground surface to approximately 10 feet below the surface and generally correspond with that mapped by the

The subsurface soils from Test Pits WTP-36 through WTP-38 generally consist of medium dense *Silty Sand (SM)* and *Silty Sand with Gravel (SM)* from the existing ground surface to approximately 11.5 feet below the surface. Excavator refusal was experienced in Test Pit WTP-36 due to cobbles and/or boulders encountered at depth.

Conditions encountered at each boring and test pit location are indicated on the individual boring and test pit logs. It should be noted that inconsistencies could exist away from, or between, boring and test pit locations. Stratification boundaries on the boring and test pit logs represent the approximate location of changes in soil types; in-situ, the transition between native materials at this site is typically gradual.

3.2.7 Groundwater Conditions

Groundwater level observations made while drilling and excavating and immediately after completion of the borings and test pits are shown in the lower left corner of the boring and test pit logs. Groundwater was encountered in Test Pits WTP-09 (B-09), WTP-20 to WTP-21, WTP-23, and WTP-26 to WTP-28 at depths of approximately 6 feet to 14 feet below the existing ground surface along the alignment.

The water level observations made during our exploration provide an indication of the groundwater conditions at the time the boring was drilled and test pits were excavated. Longer monitoring in piezometers or cased holes would be required to evaluate long-term groundwater conditions. Fluctuations in the long-term groundwater levels should be expected throughout the year depending upon variations in the amount of rainfall, runoff, evaporation, and other hydrological conditions not apparent at the time the borings were drilled. In addition, perched water can develop within higher permeability soils overlying less permeable soils following heavy or prolonged precipitation. Therefore, groundwater levels during construction or at other times in the future may be higher or lower than the levels indicated on the boring and test pit logs.

3.2.8 Laboratory Testing

Soils testing performed by Terracon were conducted in general accordance with the American Society for Testing and Materials (ASTM), standards where applicable. Representative samples of major material types were tested to determine their grain size distribution (ASTM D422), plasticity index or Atterberg Limits (ASTM D4318), and moisture content (ASTM D2216). These test results are used to properly classify the soils and to adjust the field logs if necessary. Such testing is typically referred to as index testing. These index tests can be correlated to empirical studies to obtain representative values for other engineering properties. Based on index testing, we can estimate design parameters and assign additional testing, as necessary.

A consolidation test (ASTM D2435) was also performed on a representative sample of *Sandy Elastic Silt (ML)* soil from Boring WTP-09 (WB-09). These results were used to estimate consolidation characteristics of the soil. A summary of the laboratory testing is presented in **Appendix B**.

A direct shear test (ASTM D3080) was performed on a representative sample of *Elastic Silt with Sand (MH)* soil from Boring WTP-09 (WB-09). These results were used to determine the cohesion and friction angle soil strength properties of the soil.

During geotechnical exploration, we sampled each test pit completed along the proposed Well Field Collection System alignment at an approximate depth of 6 feet below the existing ground surface for subsequent corrosion testing, where possible.

Corrosion testing includes acidity (EPA 2310B), chloride (EPA 300.0), pH (EPA 150.1), redox potential (EPA 2580B), resistivity (EPA 2510B), sulfate (EPA 300.0), and sulfate reducing bacteria (SRB) (EPA BART), as requested by ECO:LOGIC. Corrosion testing was performed by Western Environmental Testing Laboratory (WETLab) of Sparks, Nevada and the results are presented in Table 4.

Table 4 – Well Field Collection System Corrosion Testing Summary

Location	Depth (ft)	Acidity (mg/kg)	Chloride (mg/kg)	pH	Redox Potential (mV)	Resistivity (ohm-cm)	Sulfate (mg/kg)	SRB
WTP-01	6	-26	<15 M	8.10	+300	2,500	16	N
WTP-02	6	-20	<15	8.20	+320	1,200	1.0	N
WTP-03	6	-8.0	<15	7.83	+340	1,100	260	N
WTP-04	6	10	190	7.87	+340	350	2,200	N
WTP-05	6	12	140	7.99	+330	720	250	N
WTP-09 (B-09)	5	12	<15	7.85	+300	1,400	43	P
WTP-10	6	-78	260	9.21	+310	430	350	N
WTP-15	6	-10	<15	8.17	+270	2,500	43	P
WTP-16	6	-60	<15	9.03	+250	890	91	N
WTP-17	6	-32	<15	9.01	+250	330	110	N
WTP-18	6	-2.0	30	7.83	+250	390	4,400	N
WTP-19	6	14	96	7.67	+240	670	160	P
WTP-20	6	-28	<15	8.19	+240	1,400	140	P
WTP-21	6.5	-56	<15	7.83	+290	1,200	<75	P
WTP-22	6	-45	<15	8.49	+300	1,400	<75	P
WTP-23	5	-33	24	9.37	+320	660	96	P
WTP-24	6	-1.0	410	7.92	+330	130	6,300	N
WTP-25	5	-66	500	9.18	+300	130	760	N
WTP-26	6	-49	160	8.88	+300	460	240	N
WTP-27	5	-40	<15	8.75	+310	660	39	N

Table 4 – Well Field Collection System Corrosion Testing Summary

Location	Depth (ft)	Acidity (mg/kg)	Chloride (mg/kg)	pH	Redox Potential (mV)	Resistivity (ohm-cm)	Sulfate (mg/kg)	SRB
WTP-28	6	-160	150	9.38	+310	210	510	N
WTP-29	4.5	-27	<15	8.25	+300	18,000	<75	N
WTP-30	5	-8.0	<15	8.04	+310	26,000	<75	N
WTP-31	4	-26	<15	7.86	+310	6,400	<75	N
WTP-33	4.5	-17	<15	8.22	+280	19,000	<75	N
WTP-34	5.5	-26	<15 M	8.29	+290	11,000	<75	N
WTP-35	1	-89	<15	8.12	+280	3,000	<75	P
WTP-36	5	4.0	<15	7.80	+280	2,700	<75	N
WTP-37	5.5	-17	<15	8.37	+280	12,000	<75	P
WTP-38	5	4.0	<15	6.90	+300	17,000	<75	N

Notes: Sulfate Reducing Bacteria (SRB) Test Codes N = Negative, P = Positive.

3.3 ENGINEERING RECOMMENDATIONS

Our recommendations are based on the assumption that the soil conditions are similar to those disclosed by the explorations. If variations are noted during construction or if changes are made in site plan, alignment or pipe configuration, structural loading, foundation type or floor level, we should be notified so we can supplement our recommendations, as applicable.

3.3.1 Geotechnical Considerations

Based on the soils encountered during our geotechnical exploration, we anticipate excavation difficulties in the form of caving sands along portions of the proposed alignment in granular soils and in cobbles and/or boulders present along the western portion of the proposed alignment. Contractors should satisfy themselves as to excavation characteristics before bidding this project.

Low to high plasticity clay soils are present along the proposed alignment. Such soils are commonly referred to as "expansive" or "swelling" soils because they expand or swell as their moisture contents increase. However, these soils also "contract" or "shrink" as their moisture contents decrease. Clay soils excavated during construction are not suitable for reuse as trench backfill material in structural areas as defined in **Section 3.3.7.4** – Trench Backfill Materials.

Groundwater was encountered at varying elevations along the proposed alignment. Dewatering may be necessary depending upon the final pipeline grades and the actual groundwater level at the time of construction.

3.3.2 Seismic Design Criteria

The subject site is located in northwestern Nevada, which is a seismically active area within the Basin and Range area. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, and on the intensity and the magnitude of the seismic event.

In order to estimate the parameters for seismic ground motions at the subject site, Terracon utilized the USGS website, "*Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2002 Data*" (USGS, 2005) in general accordance with the 2003 *International Building Code*. According to this source, the peak accelerations at the most Northeastern end of the alignment at Test Pit WTP-01 and the end of the alignment at the proposed Pump Station are shown in Table 5. We recommend that the seismic criteria values associated with the proposed Pump Station be utilized in design. These values are more conservative than the lower values associated with the start of the alignment.

Table 5 – Site Seismic Criteria

Ground Motion	10% Possibility of Exceedence In 50 Years (%g)		2% Possibility of Exceedence in 50 Years (%g)	
	WTP-01	*Pump Station*	*WTP-01*	*Pump Station*
Peak Ground Acceleration (PGA)	23.21	25.48	44.76	48.05
0.2 sec Spectral Acceleration (SA)	56.32	62.03	109.21	117.59
1.0 sec Spectral Acceleration (SA)	19.97	21.62	42.15	44.32

Based on our geotechnical exploration along the alignment, we suggest a Site Class definition "D" (Table 1615.1.5, 2003 *International Building Code*) for the portion of the alignment that has been explored. A portion of the unexplored alignment near the Honey Lake Valley playa may be a Site Class definition "E". Geotechnical exploration has not been completed along the alignment from Test Pits WTP-06 to WTP-08 and WTP-11 to WTP-14. As a result, the Site Class definition for this section of the alignment is unknown. The Site Class definition "D" corresponds to a *stiff soil profile* and an "E" corresponds with a *soft soil profile*.

3.3.3 Liquefaction

Based on the anticipated ground motion shown above, the presence of saturated, *Silty Sand (SM)* and *Poorly-Graded Sand with Silt (SP-SM)* soils encountered during geotechnical exploration, and liquefaction analyses, it is our opinion that liquefaction potential at this site is possible.

Liquefaction analyses were performed in general accordance with current geotechnical practice and recommendations provided by the Center for Earthquake Engineering Research (NCEER) 1996 and 1998 Workshops (Youd et al., 2001). Liquefaction analyses performed on these two soil units at Boring WTP-09 (WB-09) [10 to 15 feet and 35 to 40 feet] indicate an estimated total settlement during liquefaction of approximately 2 inches. This estimated total settlement from liquefaction is based upon the soil, groundwater, and anticipated seismic conditions present near Boring WTP-09 (WB-09). Soil, groundwater, and seismic conditions may be different along other portions of the proposed alignments. As such, total liquefaction settlements may be more or less than estimated here.

It is generally accepted that liquefaction settlements below approximately 50 feet will not reflect to the surface; therefore, the liquefaction boring was not extended below 50 feet.

3.3.4 Pipe Design Parameters

The following soil design parameters for the Well-Field Collection System alignment are presented in Tables and may be used where applicable for the design of the proposed alignment. Design parameters have been determined for the according to the soil properties encountered along the alignment. Areas with similar soil properties have been grouped together and assigned pipe design parameters. Pipe design parameters have been determined assuming an embedment depth of approximately 6 feet.

Table 6 – Soil Design Parameters for the Well-Field Collection System Alignment				
Alignment Section	**Coefficient of Friction [1]**	**Modulus of Soil Reaction E at 6 Feet [1,2] (psi)**	**General USCS Soil Classification at a Depth of 6 Feet**	**Unit Weight of Compacted Fill[1] (pcf)**
WTP-01 to WTP-22	0.35	700	SM, SP-SM	125
WTP-23 to WTP-28	0.20	200	CL, SC, MH	125
WTP-29 to WTP-34	0.35	700	SM, SP-SM	125
WTP-35 to WTP-36	0.20	100	SC	125
WTP-37 to WTP-38	0.35	700	SM	125

[1]Assumes AWWA Pipe Bedding Material.
[2]Bureau of Reclamation Value of E' for Iowa Formula. [Assumes a minimum of 80 percent relative compaction (ASTM D1557).]

The modulus of soil reaction values were determined using the method of Howard (1977), included in **Appendix D**.

3.3.5 Thrust Blocks

General thrust block design parameters are shown in Table 7. Thrust blocks should bear on granular material whenever possible. Where thrust blocks are planned and *Lean Clay (CL)* or *Elastic Silt (MH)* soils are encountered, overexcavation is recommended as described in **Section 3.3.7.3** – Overexcavation. Thrust blocks in these clay or silt soils should bear on at least 12 inches of aggregate base or structural fill compacted to a minimum of 90 percent relative compaction (ASTM D1557).

Lateral loads may be resisted by soil friction and by the passive resistance of the soil. A coefficient of friction of 0.35 may be used between thrust blocks and the supporting granular, native, trench backfill, or structural fill soils.

Table 7 – Thrust Block Design Parameters		
Parameter	**Granular Soils**	**Clayey Soils**
Allowable Maximum Bearing Capacity	2,500 psf	1,500 psf
Allowable Coefficient of Friction	0.35	0.24
Ultimate Passive Earth Pressure	300 pcf	200 pcf
Ultimate Active Earth Pressure	33 pcf	49 pcf
Ultimate At-Rest Pressure	50 pcf	65 pcf

The passive, active, and at-rest earth pressures assume that the backfill is properly compacted. Backfill should be mechanically compacted in layers (8-inch maximum thickness) to a minimum of 95 percent relative compaction (ASTM D1557); flooding should not be permitted. Care should be taken when placing backfill so as not to damage the footings.

Thrust block locations should be carefully inspected to ensure adequate lateral bearing. If very soft soil conditions are encountered, larger thrust blocks or overexcavation with forming thrust blocks and backfilling with structural fill may be necessary. Thrust blocks should be poured on a firm and unyielding surface.

3.3.6 Permanent Slopes

Permanent slopes are not anticipated along this alignment which are substantially different than the natural, existing topography. Should any permanent slopes be elected, they should be no steeper than 2.5H:1V (Horizontal:Vertical). At a 2.5H:1V geometry, minor sloughing of the slope surface should be expected.

Any required erosion control measures should be provided for all slopes as soon as possible after grading. Drainage from the toe of the slope should be directed around the site and away from the disturbed area. It should be noted that native, surficial soils along the proposed alignment are moderately erosive. Rip-rap or other erosion control measures will likely be necessary at existing drainage channel and creek crossings.

3.3.7 Earthwork

3.3.7.1 Site Clearing

Existing topsoil, vegetation, and other deleterious materials are to be removed along the proposed alignment to a minimum depth of 6 inches. All materials derived from the site stripping operations should be stockpiled separately from trench backfill materials for later use in vegetation efforts. Grading work should not be performed on or with frozen soils.

3.3.7.2 Excavation

Groundwater encountered during our geotechnical exploration along the proposed well field alignment between 6 and 14 feet below the existing grades. Groundwater may pose limited constraints to installation of the proposed Well Field Collection System piping. Required excavations below the groundwater table can expect caving and will require dewatering and trench shoring.

Based on the soils encountered during the exploration, we anticipate excavation difficulties in the form of caving sands along portions of the proposed alignment in granular soils and in cobbles and/or boulders present along the western portion of the proposed alignment. Contractors should satisfy themselves as to excavation characteristics before bidding the project.

Excavations or trenches should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions. All excavations or trenches should comply with all applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that Terracon is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

3.3.7.3 Overexcavation

Minor overexcavations can be anticipated at thrust block locations in areas of *Lean Clay (CL)* and *Elastic Silt (MH)* soils. Thrust blocks in these soils should bear on a minimum of 12 inches of aggregate base or structural fill material.

Overexcavations should be cleaned of loose or deleterious material and backfilled with compacted granular structural fill or aggregate base.

Areas to receive fill should be scarified to a minimum depth of 6 inches, moisture conditioned, and compacted to a minimum of 90 percent relative compaction (ASTM D1557). Areas that are unstable or pumping should be stabilized to a firm and non-yielding condition prior to receiving fill. Native clay soils may be too wet to compact upon excavation. Moisture conditioning may be possible by scarifying the top 12 inches of subgrade and allowing it to air dry to near the optimum moisture prior to compaction. Where moisture conditioning by air drying is ineffective, mechanical stabilization may be necessary. Mechanical stabilization may be achieved by overexcavation and/or placement of an angular and well-graded 12- to 18-inch thick lift of 12-inch-minus to 3-inch-plus rock fill or Class C or D drainrock. This fill should be compacted with large equipment until no further deflection is noted. Where more than 30 percent is retained on the ¾-inch sieve, standard density testing is not valid. As will likely occur here, standard density testing is not possible, and a proof rolling program will be required to obtain an unyielding surface.

Structural fill or aggregate base material should be placed in maximum 8-inch-thick loose lifts and compacted to 95 percent relative compaction (ASTM D1557). The overexcavation and backfill procedure is shown in Figure 1 below. The guidelines for imported structural fill are presented in Table 6 – Guidelines for Imported Structural Fill presented below in **Section 3.3.7.4** – Trench Backfill Materials. It should be the contractor's responsibility to ensure that the bottom of the excavation is prepared in such a manner as to allow the specified minimum compaction of the structural fill or aggregate base to be achieved.

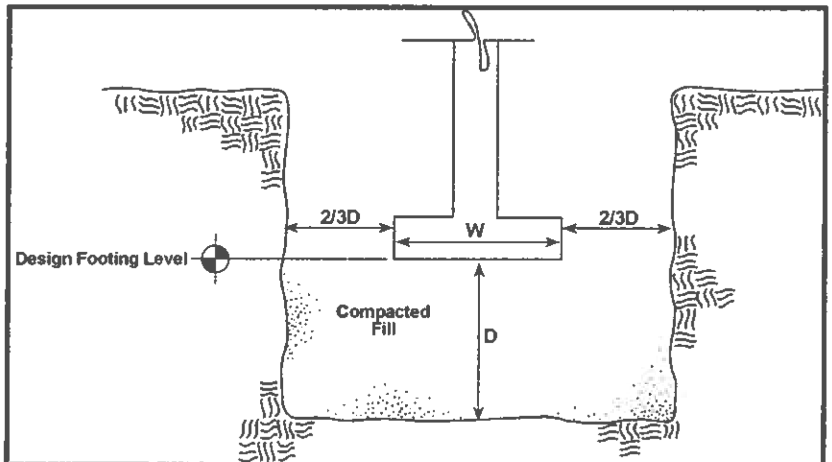

Figure 1 – Thrust Block Overexcavation and Backfill

3.3.7.4 Trench Backfill Materials

Native granular soils excavated along the proposed alignment should be considered for reuse as backfill material (as described below) above the top of pipe bedding. Structural areas are considered areas with improvements, such as the existing Fish Springs Road and other unnamed access roads, or the proposed parking/access roads associated with the Well Houses. Non-structural areas are considered other areas without improvements, such as generally along the proposed alignment.

Non-Structural Backfill

In non-structural areas, excavated material can be generally used as trench backfill provided any coarse material greater than 4 inches in largest dimension is removed prior to placement. To minimize settlement of trench backfill and subsequent ponding of water, trench backfill in non-structural areas should be placed in maximum 12-inch loose lift thicknesses and compacted to a minimum of 85 percent relative compaction (ASTM D1557).

Structural Backfill

Should structural improvements be anticipated in areas of the proposed alignment, such areas should be backfilled with approved structural fill. Native clay or silt soils will not be suitable for reuse as structural fill or within 3 feet of structural sections. All imported structural fill materials should comply with the guidelines presented in Table 8, Guidelines for Imported Structural Fill.

Table 8 – Guidelines for Imported Structural Fill (ASTM C136)	
Sieve Size	Percent Passing
4-Inch	100
¾-Inch	70 - 100
No. 40	15 - 70
No. 200	5 - 30
Maximum Allowable Liquid Limit and Plasticity Index (ASTM D4318)	
Liquid Limit (LL)	35 max
Plasticity Index (PI)	10 max
Minimum Allowable R-Value (ASTM D2844)	
R-Value	45 min
Maximum Allowable Soluble Sulfates (EPA 300)	
Sulfates	< 0.1% by Weight SO_4

Materials that deviate from these guidelines must be approved by the geotechnical engineer prior to use.

Structural fill should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557). The moisture content of structural fill at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

3.3.7.5 Pipe Bedding Materials

Pipe bedding materials should meet the minimum specifications presented by the American Water Works Association (AWWA) and the *Standard Specifications for Public Works Construction*, latest edition.

4.0 WELL HOUSE STRUCTURES

Six water supply wells and associated Well House Structures are planned for the Fish Springs Water Supply Project near the existing Fish Springs Ranch within Honey Lake Valley. The six well sites are located on Fish Springs Ranch, LLC property in Honey Lake Valley. Table 10 shows the location for each of the six well sites. An emergency generator and associated concrete pad is also planned for some of the well house sites. Existing agricultural wells are located adjacent to the proposed Well 2, 4, and 5 sites. It is our understanding that new wells will be drilled for this project to meet current Nevada Division of Environmental Protection (NDEP), Bureau of Safe Drinking Water regulations governing public water systems, as the existing agricultural wells do not meet public water system well construction regulations.

It is anticipated that the Well House Structures will be approximately 473 square feet each, will be of masonry block construction supported by a slab-on-grade, and have a finished floor elevation of 6 inches above surrounding grade. The Well House Structures will be located within a graded and fenced area, with gravel-covered compacted aggregate base surfacing. Foundation loads were not available at the time this report was prepared; however, we anticipate that loads will be low to moderate, based upon the structure use. In addition, we anticipate that pump loads will be supported by the well casing. The magnitude of structural fill that will be placed at each well house site to achieve building pad grade is currently unknown. However, we assume that less than 2 feet of structural fill will be placed.

Table 9 – Well House Structure Locations

Well Number	Section	Township	Range
Well 1	10	26 North	19 East
Well 2	9/10		
Well 3	16		
Well 4	29		
Well 5	30		
Well 6	25		

4.1 SITE EXPLORATION PROCEDURES

4.1.1 Field Exploration

The proposed Well House Structure sites were explored on December 7, 8, and 22, 2005 by excavating one test pit at each of the proposed sites at the approximate locations indicated on the Geotechnical Alignment Maps presented in **Appendix A**. Terracon established the test pit locations in the field by taping from the available reference features shown on plans provided by ECO:LOGIC and by using a hand-held WAAS enabled GPS unit. The test pit locations should be considered accurate only to the degree implied by the methods used to define them. A total of 6 test pits were excavated for geotechnical exploration of the well house sites, as summarized below in Table 10. Test pits WTP-01, WTP-02, WTP-05, WTP-15, and WTP-19 were excavated with a Cat 314C track-mounted excavator and Test Pit WTP-28 was excavated with a Case 330C track-mounted excavator.

Table 10 – Exploration and Well House Structure Summary

Exploration Number	Proposed Structure Name	Maximum Depth of Exploration (ft)	GWT (ft)
WTP-01	Well 1	12.0	NE
WTP-02	Well 2	12.0	NE
WTP-05	Well 3	12.0	NE
WTP-15	Well 4	14.0	NE
WTP-19	Well 5	13.2	NE
WTP-28	Well 6	16.0	12.5

An engineering geologist classified the site soils in the field in general accordance with ASTM D2488. Samples were then returned to our Sparks, Nevada laboratory, and classifications were confirmed or revised based on laboratory testing of selected samples. The descriptions of the soils indicated on the test pit logs are in accordance with the enclosed General Notes and the Unified Soil Classification System (USCS). Classification in this manner provides indications of soil properties and can be correlated to other properties using published charts (Bowles, 1995). Estimated group symbols according to the USCS are given on the test pit logs and a brief description of USCS classification system is shown in **Appendix C**.

4.2 SITE AND SUBSURFACE CONDITIONS

4.2.1 Site Conditions

The proposed Well House Structure sites are located along the southern margin of Honey Lake Valley in Washoe County, Nevada. At the time of exploration, existing agricultural wells were located adjacent to the proposed Well 2, 4, and 5 sites. Proposed Well 1 and 2 sites are located near the edges of existing center pivot irrigated fields, Well 3 is located in an undeveloped area west of Flanigan Road with sagebrush vegetation to 5 feet in height, Well 5 is located south of an existing field near the Fish Springs Ranch, and Well 6 is located in an undeveloped area north of Fish Springs Road with sagebrush vegetation to 5 feet in height. Proposed Wells 1, 2, and 3 are accessed from Flanigan Road and Wells 4, 5, and 6 are accessed from Fish Springs Road.

4.2.2 General Geologic and Soil/Bedrock Conditions

The well sites lie near the southern edge of Honey Lake Valley between the Fort Sage Mountains to the south and the Virginia Range to the east and within the Basin and Range geomorphic province. The Basin and Range geomorphic province is characterized by horizontal extension between the Sierra Nevada Range to the west and the Wasatch Front to the east and is bounded by the Colorado Plateau to the southeast and the Columbia Plateau to the north. This portion of the Honey Lake Valley is generally characterized by fine to coarse grained alluvial fan deposits from the adjacent mountain ranges. Most of the coarse grained sediments were deposited quickly during the post-glacial period during flooding events. As a milder climate developed in the region, the volume and size of the sediments were greatly reduced, and geomorphic processes generally changed into reworking of earlier deposited materials.

The Nevada Bureau of Mines and Geology (NBMG) has mapped the proposed Well 1, 2, and 3 sites as consisting of Quaternary stream deposits and lake deposits. The Quaternary stream and lake deposits were described as *talus, slope wash, alluvial fan and eolian deposits*. The Quaternary lake deposits were described as *clay, silt, sand, gravel and calcareous tufa ... chiefly of Pleistocene age* (Bonham, 1969). The NBMG has mapped the proposed Well 4, 5, and 6 sites as consisting of Quaternary fine gravel and sand alluvium. The Quaternary fine sand and gravel alluvium was described as *coalescing alluvial fans form bajadas that merge with sand and clay deposits of the floor of Honey Lake Valley ... the alluvium is composed of gravel- and sand-sized clasts of quartz, feldspar, and volcanic rocks derived from the Fort Sage and Virginia Mountains to the south* (Grose, 1984).

Field geologic reconnaissance performed by Terracon generally encountered similar geologic conditions to that mapped by the NBMG.

4.2.3 Faulting and Seismicity

The published and available geologic maps indicate that one fault (Quaternary or older) is mapped within 1 mile of the proposed Well 5 (Test Pit WTP-19) site (Grose, 1984). No other faults are mapped as crossing or within one mile of the other wellhouse sites. This fault is planned for fault trenching as a part of **Section 5.0** - Well Field Collection System alignment activities, once access issues have been resolved by ECO:LOGIC.

Guidelines for determining and assessing faults have been developed by the Nevada Earthquake Safety Council (NESC); however, neither the State of Nevada nor various counties have adopted the guidelines. These NESC guidelines are generally consistent with those adopted in the State of California Alquist-Priolo Act of 1972 that defines active faults as those faults with displacement within the past 11,000 years (Holocene) and those faults with displacement within the past 11,000 to 2,000,000 years as potentially active.

Based on the geologic mapping and the lack of a definitive age of faulting, the mapped fault in the vicinity of Well 5 is considered active to potentially active.

Nevada ranks as the fourth highest state of earthquake occurrence of Magnitude 3.5 and greater, behind Alaska, California, and Hawaii (USGS, 2005). In addition, the Truckee Meadows lies in one of the most seismically active regions of Nevada. dePolo et al., in a study of earthquake occurrence in the Reno-Carson City urban corridor, state that *overall, the probabilities of potentially damaging earthquakes within the region are relatively high and are commensurate with many parts of Califomia, a state with a well-recognized high earthquake hazard* (dePolo et al., 1997). These probabilities are also valid for the area encompassed by this project. As a result of seismicity present, the well house structures should be designed accordingly.

4.2.4 Flooding

To determine relative flooding hazard, Terracon referenced the available Federal Emergency Management Agency (FEMA) FIRM maps. The referenced flooding data is provided for informational purposes only. Since flooding and site drainage are not the responsibility of Terracon, we recommend that the site Civil Engineer be consulted concerning flooding and other drainage hazards.

FEMA has mapped the proposed Well House Structure sites as of 1994 as lying within unshaded Zone X, and is defined as *areas determined to be outside [the] 500-year floodplain* (FEMA, 1994).

4.2.5 Subsurface Conditions

The subsurface soils at Well 1 generally consist of medium dense to dense *Silty Sand (SM)* and *Poorly Graded Sand with Silt and Gravel (SP*); at Well 2 generally consist of medium dense to dense *Clayey Sand (SC), Silty Sand (SM)*, and *Poorly Graded Sand with Silt (SP-SM)*, with minor medium stiff *Sandy Lean Clay with Gravel (CL);* at Well 3 generally consist of loose to dense *Silty Gravel with Sand (GM)*, *Silty Sand (SM)*, and *Clayey Sand (SC)*; at Well 4 generally consist of loose to medium dense *Silty Sand with Gravel (SM)* and stiff *Sandy Silt (ML)*; at Well 5 generally consist of loose to medium dense *Silty Sand with Gravel (SM)* and *Poorly Graded Sand (SP);* and at Well 6 generally consist of stiff to hard *Lean Clay (CL), Elastic Silt (MH), and Silt with Sand (ML).*

Conditions encountered at each test pit location are indicated on the individual test pit logs. It should be noted that inconsistencies could exist away from, or between test pit locations. Stratification boundaries on the test pit logs represent the approximate location of changes in soil types; in-situ, the transition between native materials at this site is typically gradual.

4.2.6 Groundwater Conditions

Groundwater level observations made immediately after completion of the test pits are shown in the lower left corner of the test pit logs. Groundwater was only encountered in Test Pit WTP-28 (Well 6) at approximately 12.5 feet below the existing ground surface.

The water level observations made during our exploration provide an indication of the groundwater conditions at the time the test pits were excavated. Longer monitoring in piezometers or cased holes would be required to evaluate long-term groundwater conditions. Fluctuations in the long-term groundwater levels should be expected throughout the year depending upon variations in the amount of rainfall, runoff, evaporation, and other hydrological conditions not apparent at the time the test pits were excavated. In addition, perched water can develop within higher permeability soils overlying less permeable soils following heavy or prolonged precipitation. Therefore, groundwater levels during construction or at other times in the future may be higher or lower than the levels indicated on the test pit logs.

4.2.7 Laboratory Testing

Soils testing performed by Terracon were conducted in general accordance with the American Society for Testing and Materials (ASTM), standards where applicable. Representative samples of major material types were tested to determine their grain size distribution (ASTM D422), plasticity index or Atterberg Limits (ASTM D4318), and moisture content (ASTM D2216). These test results are used to properly classify the soils and to adjust the field logs if necessary. Such testing is typically referred to as index testing. These index tests can be correlated to empirical studies to obtain representative values for other engineering properties. Based on index testing, we can estimate design parameters and assign additional testing, as necessary.

4.3 ENGINEERING RECOMMENDATIONS

Our recommendations are based on the assumption that the soil conditions are similar to those disclosed by the exploration. If variations are noted during construction or if changes are made in site plan, structural loading, foundation type or floor level, we should be notified so we can supplement our recommendations, as applicable.

4.3.1 Geotechnical Considerations

The six Well House Structure sites appear to be suitable for the proposed construction based on geotechnical conditions encountered in the exploratory test pits. Based on the subsurface exploration, geotechnical analyses, and laboratory test results, we recommend that the proposed Well House Structures be supported on spread footing foundation systems.

Soil conditions in the vicinity of proposed Well Houses 1, 2, 3, 4, and 5 are generally granular soils with good bearing capacity and settlement characteristics, while soil conditions in the vicinity of proposed Well House 6 is generally *Lean Clay (CL)* and *Elastic Silt (MH)* soils that are potentially expansive and will experience significant settlement upon loading.

Expansive soils are present at the proposed Well 6 (WTP-28) site. This report provides recommendations to help mitigate the effects of soil shrinkage and expansion. However, even if these procedures are followed, some movement and at least minor cracking in the structure should be anticipated. The severity of cracking and other cosmetic damage such as uneven floor slabs will probably increase if any modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and cosmetic distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. Some of these measures include completely overexcavating the expansive clay soils and replacement with non-expansive structural fill material. We would be pleased to discuss other construction alternatives with you upon request.

If structures, concrete flatwork, pavements, utilities, or other improvements are located in the vicinity of any of the test pits, the test pit backfill should be removed and recompacted in accordance with **Section 4.3.7.3** - Overexcavation. Failure to properly compact backfill could result in excessive settlement of improvements located over test pits.

Recommendations regarding the design and construction of foundations, and the support of floor slabs, relative to the subsurface conditions encountered in the test pits, are presented in the following sections of this report.

4.3.2 Seismic Design Criteria

The proposed Well House Structure sites are located in northwestern Nevada, which is a seismically active area within the Basin and Range area. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, and on the intensity and the magnitude of the seismic event.

In order to estimate the parameters for seismic ground motions at the subject site, Terracon utilized the USGS website, *"Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2002 Data"* (USGS, 2005) in general accordance with the 2003 *International Building Code*. The peak accelerations at the proposed Well House

Structure sites, developed in accordance with the above-mentioned sources, are shown in Table 11:

Table 11 – Wells 1 to 6 *IBC* Site Seismic Criteria		
Ground Motion	**10% Possibility of Exceedence in 50 Years (%g)**	**2% Possibility of Exceedence in 50 Years (%g)**
Peak Ground Acceleration (PGA)	24.44	45.47
0.2 sec Spectral Acceleration (SA)	59.50	111.63
1.0 sec Spectral Acceleration (SA)	20.99	42.81

Based on our site exploration, we suggest a Site Class definition "D" for the proposed Well House Structure sites (Table 1615.1.5, 2003 *International Building Code*). This Site Class definition corresponds to a *stiff soil profile*.

4.3.3 Liquefaction

Based on the anticipated ground motion shown above and the lack of saturated, clean sand soils encountered during exploration, it is our opinion that liquefaction potential at the proposed sites is minimal. Should groundwater levels rise in the vicinity of the proposed Well House Structure sites in the future, due to climatic or other changes, liquefaction potential may be an issue due to the presence of clean sand soils at some locations and should be re-addressed at that time.

4.3.4 Foundation Systems

Conventional shallow foundations, such as spread footings, are appropriate for supporting the proposed well house structures. Spread footings should be embedded a minimum of 24 inches below the nearest adjacent grade for frost protection and should bear on a minimum of 12 inches of compacted granular structural fill, except for the structure at Well 6 which should bear on a minimum of 36 inches of compacted granular structural fill. Spread footings bearing on a minimum of 12 or 36 inches of properly compacted granular structural fill (as described above) can be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot (psf) (assuming a minimum embedment depth of 24 inches within the structural fill). The net allowable soil bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation.

The magnitude of settlement that will occur beneath the foundations would depend upon the variations within the subsurface soil profile, the actual structural loading conditions, the embedment depth of the footings, the actual thickness of compacted fill, and construction quality control. Assuming that spread footing construction is performed in accordance with our recommendations, it is our opinion that total settlement will be on the order of approximately 1 inch. Differential settlement up to approximately two-thirds of the total settlement value should be anticipated.

It is recommended that the building excavations be evaluated by the geotechnical engineer during excavation, but prior to structural fill or concrete placement. If, during evaluation, unsuitable materials are encountered, the geotechnical engineer may require that the footing subgrade be modified by additional overexcavation and subsequent structural fill placement.

Properly compacted structural fill in footing areas should be free of water and loose soil prior to placing concrete. Concrete should be placed as soon as possible after excavation to minimize disturbing the bearing material. Should the material at bearing level become disturbed or saturated, the affected soil should be removed and replaced prior to placing concrete. It is critical that the clay soils are not allowed to dry during construction. If the clay soils are allowed to dry, additional over-excavation will be required to remove the dried material.

4.3.5 Lateral Earth Pressures

Lateral loads may be resisted by soil friction and by the passive resistance of the soil. An allowable coefficient of friction of 0.35 may be used between foundations or floor slabs and the supporting structural fill soils. The ultimate passive resistance of properly compacted fill may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot (pcf). A one-third increase may be used for wind or seismic loads. The passive pressure and the frictional resistance of the soils may be combined without reduction in determining the total lateral resistance. An appropriate factor of safety should be applied to ultimate values.

These lateral load recommendations assume structural fill against foundations is properly placed and compacted. Backfill should be mechanically compacted in layers (8-inch maximum thickness) to a minimum of 95 percent relative compaction (ASTM D1557); flooding should not be permitted. Care should be taken when placing backfill so as not to damage the footings. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors to minimize damage.

4.3.6 Permanent Slopes

Currently, water from precipitation events flows across the six well house sites generally by sheetflow. Drainage improvements consisting of regrading and/or drainage channels may be necessary. Water must not be allowed to flow onto the constructed fill pads. In addition, drainage from the pad areas should be routed through pipes or concrete-lined drainage swales from the pad elevation to native elevations below using appropriate splash blocks or plates. Drainage near the toe of the pad slopes should be directed away from the slopes in a manner to prevent erosion of the toe area of the slopes. It should be noted that native, surficial soils at the proposed well house sites are generally moderately erosive and should not be used for slope construction without erosion protection, such as rock rip-rap.

Permanent slopes may be present at the six well house sites due to well (fill) pad construction. As grading plans for these well pads are not currently available, it is assumed that slopes will be less than 3 feet in height. Slopes should be no steeper than 2.5H:1V (Horizontal:Vertical). At a 2.5H:1V geometry, minor sloughing of the slope surface should be expected. Any required erosion control measures should be provided for on all slopes as soon as possible after grading. It is recommended that fill slope construction be monitored on a full-time basis by the geotechnical engineer during excavation and lift placement. During monitoring, the geotechnical engineer may require additional overexcavation and subsequent fill placement.

4.3.7 Earthwork

4.3.7.1 Site Clearing

Existing topsoil and other deleterious materials are to be removed from the proposed Well House Structure sites to a minimum depth of 8 inches in pad areas and non-structural areas and to a minimum depth of 12 inches within the building footprints and extending at least 3 feet in all directions from the building footprints. All materials derived from the site stripping operations should be removed from the site and not used in any fills or backfills. Grading work should not be performed on or with frozen soils.

4.3.7.2 Excavation

Groundwater was encountered during our exploration only at the proposed Well 6 site at approximately 12.5 feet below the existing grades. Groundwater may pose limited constraints to installation of deeper utilities due to the site location, topography, anticipated fill, and anticipated depth to groundwater.

Required excavations below the groundwater table can expect caving and will likely require dewatering and trench shoring. Contractors should satisfy themselves as to excavation characteristics before bidding the project.

Excavations or trenches should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions.

All excavations or trenches should comply with all applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that Terracon is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

4.3.7.3 Overexcavation

It is anticipated that fills on the order of 1 to 2 feet in thickness will be required to bring the proposed Well House Structure sites to finished grade. Additionally, it is recommended that foundations be embedded a minimum of 24 inches for frost protection and that foundations bear on a minimum of 12 (Wells 1, 2, 3, 4, and 5) inches or 36 (Well 6) inches of structural fill.

Native clay soils present at the proposed Well 6 site (WTP-28) are not directly suitable as foundation subgrade soils. Foundations at the Well 6 site should bear on a minimum of 36 inches of compacted granular structural fill. As a result, overexcavation of 5 to 6 feet can be anticipated at this site in order to achieve the required 36 inches of structural fill beneath footings. A separator geotextile, such as Geotex 451, should be placed on the bottom of the prepared overexcavation before placement of structural fill. Should soft clay soils be present below the 36 inches of compacted granular structural fill, additional overexcavation and replacement with granular structural fill may be required.

In addition, native clay soils present at the proposed Well 6 site (WTP-28) are not directly suitable for pad area subgrade soils. The proposed Well House pad area should be overexcavated a minimum of 24 inches and backfilled with compacted granular structural fill. A separator geotextile, such as Geotex 451, should be placed on the bottom of the prepared overexcavation before placement of structural fill. The pad should then be constructed over this overexcavated section to the grades required as described in **Section 4.3.9** - Pavements.

The constructed fill pads will likely intersect loose, *Silty Sand with Gravel* soils at Wells 4 (WTP-15) and 5 (WTP-19). These *Silty Sandy with Gravel* soils should be overexcavated until medium dense to dense soils are encountered to reduce the settlement potential of the constructed fill pads.

Overexcavations should be cleaned of loose or deleterious material and backfilled with compacted granular structural fill. Structural fill material should be placed in maximum 8-inch-thick loose lifts and compacted to 95 percent relative compaction (ASTM D1557). The overexcavation and backfill procedure is shown in Figure 2 below. The guidelines for imported structural fill are presented in Table 10 – Guidelines for Imported Structural Fill presented below in **Section 4.3.7.4** - Fill Materials. It should be the contractor's responsibility to ensure that the bottom of the excavation is prepared in such a manner as to allow the specified minimum compaction of the structural fill to be achieved.

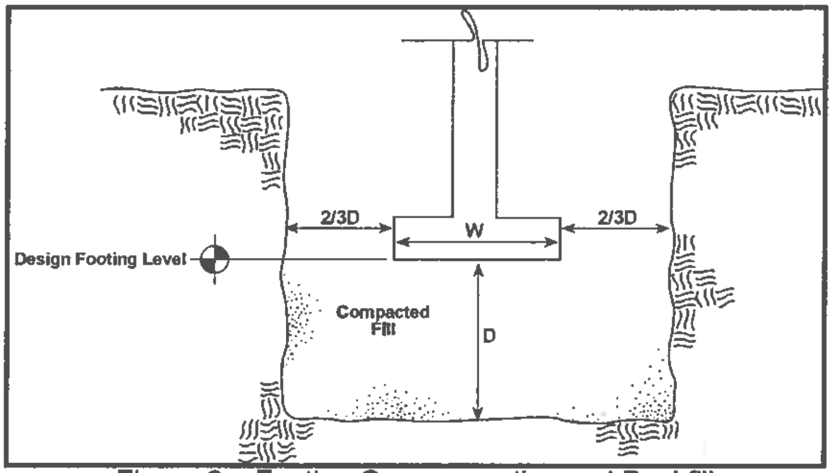

Figure 2 – Footing Overexcavation and Backfill

4.3.7.4 Fill Materials

Due to the compressible nature of native clay soils present at the proposed Well 6 site, these materials are not suitable for use as structural fill beneath structural improvements. All structural fill materials should comply with the guidelines presented in Table 12, Guidelines for Imported Structural Fill.

Table 12 – Guidelines for Imported Structural Fill (ASTM C136)	
Sieve Size	Percent Passing
4-Inch	100
¾-Inch	70 - 100
No. 40	15 - 70
No. 200	5 - 30
Maximum Allowable Liquid Limit and Plasticity Index (ASTM D4318)	
Liquid Limit (LL)	35 max
Plasticity Index (PI)	10 max
Minimum Allowable R-Value (ASTM D2844)	
R-Value	45 min
Maximum Allowable Soluble Sulfates (EPA 300)	
Sulfates	< 0.1% by Weight SO_4

Materials that deviate from these guidelines must be approved by the geotechnical engineer prior to use.

4.3.7.5 Fill Placement and Compaction

Areas to receive fill should be scarified to a minimum depth of 6 inches, moisture conditioned, and compacted to a minimum of 90 percent relative compaction (ASTM D1557). Areas that are unstable or pumping should be stabilized to a firm and non-yielding condition prior to receiving fill. Native clay soils may be too wet to compact upon excavation. Moisture conditioning may be possible by scarifying the top 12 inches of subgrade and allowing it to air dry to near the optimum moisture prior to compaction. Where moisture conditioning by air drying is ineffective, mechanical stabilization may be necessary. Mechanical stabilization may be achieved by overexcavation and/or placement of an angular and well-graded 12- to 18-inch thick lift of 12-inch-minus to 3-inch-plus rock fill or Class C or D drainrock. This fill should be compacted with large equipment until no further deflection is noted. Where more than 30 percent is retained on the ¾-inch sieve, standard density testing is not valid. As will likely occur here, standard density testing is not possible, and a proof rolling program will be required to obtain an unyielding surface.

All structural fill materials should comply with the requirements presented in Table 2 – Guidelines for Imported Structural Fill. Structural fill should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557) in non-structural areas and 95 percent relative compaction (ASTM D1557) in structural areas. The moisture content of structural fill at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

4.3.7.6 Trench Backfill

Native granular soils may be utilized as intermediate utility line backfill provided such soils are kept at a minimum depth of 12 inches below the bottom of footings (including aggregate base section). The top 12 inches of trench backfill with pad areas, respectively, should comply with the requirements presented in Table 2 – Guidelines for Imported Structural Fill. Due to the compressible nature of native clay soils present at the proposed Well 6 site, these materials are not suitable for use as intermediate utility line backfill within the pad areas.

Native materials used as intermediate trench backfill beneath Well House Structures should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557). The moisture content of native soils at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

4.3.8 Floor Slab Design and Construction

We recommend that the floor slabs and emergency generator pads be supported on a minimum of 12 inches of structural fill (including aggregate base section) to provide more uniform support and to reduce the potential for differential movement. Some differential movement of a slab-on-grade floor system is possible should the subgrade soils experience fluctuation in moisture content. Such movements are anticipated to be within general tolerance for normal slab-on-grade construction. To reduce potential slab movements, the subgrade soils should be prepared as outlined in **Section 4.3.7** - Earthwork - section of this report.

Additional floor slab design and construction recommendations are as follows:

- Positive separations and/or isolation joints should be provided between slabs and all foundations, columns or utility lines to allow independent movement.
- Control joints should be provided in slabs to control the location and extent of cracking.
- Interior trench backfill placed beneath slabs should be compacted in accordance with recommendations outlined below.
- Floor slabs should not be constructed on frozen subgrade.
- Other design and construction considerations, as outlined in the ACI *Design Manual*, Section 302.1R are recommended.

The concrete floor slab should be supported on a minimum 6-inch-thick layer of compacted aggregate base. The purpose of the aggregate base section is to help distribute concentrated loads. Note that the 6-inch layer of compacted aggregate base can account for a portion of the overexcavation.

Recommendations presented by the American Concrete Institute for slabs-on-grade should be complied with for all concrete placement and curing operations. Improper curing techniques and/or excessive slump (water-cement ratio) could cause excessive drying/shrinkage resulting in random cracking and/or slab curling. Concrete slabs should be allowed to cure adequately before placing vinyl or other moisture-sensitive floor coverings.

For design purposes, a modulus of subgrade reaction (K) equal to 250 pounds per cubic inch (pci) should be used assuming that 12 inches of properly compacted structural fill are placed below the slab-on-grade (Huang, 2004).

4.3.9 Pavements

Hard pavements (asphalt concrete or Portland cement concrete) are not anticipated for the proposed Well House Structure pad areas. It is planned that a gravel surfacing will be used over underlying aggregate base. Aggregate base used in pad areas should be a minimum of 12 inches of Type 2, Class B aggregate base (*Standard Specifications for Public Works Construction*, latest edition) and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557). Overexcavation of native clay soils will be required at the proposed Well House 6 site (WTP-28) before placement of the aggregate base section depending upon the pad thickness.

5.0 PUMP STATION AND ASSOCIATED WATER STORAGE TANKS

The proposed Pump Station site consists of a rectangular graded pad within an approximate 640 acre parcel of the Fish Springs Ranch (Washoe County Parcel APN 074-04-26) located south of Honey Lake in Washoe County, Nevada. The parcel is located entirely within Section 33, Township 26 North, Range 18 East, M.D.M. The proposed Pump Station site was relocated several times, due to system design changes by ECO:LOGIC.

It is anticipated that the Pump Station structure will be approximately 42 feet by 104 feet and will be of concrete masonry unit (CMU) construction supported by spread footings. The Pump Station structure will likely contain 5 water pumps, two 2,500-gallon sodium hypochlorite storage tanks, a 1,570-cubic foot (ft^3) surge tank, minor equipment storage, electrical room, and an employee restroom. Compacted aggregate base parking areas will be constructed around the building. Structure foundation loads were not available at the time this report was prepared; however, we anticipate that loads will be (low to moderate), based upon structure use. Table 13 summarizes the currently known loading at the proposed Pump Station as provided by ECO:LOGIC. We understand that cuts of up to 10 feet and minor fills of up to 2 feet may be required to achieve site/building pad grade at the site.

Table 13 – Known Pump Station Equipment Loadings*

Equipment	Loading (lbs)	Anticipated Foundation Size (ft)	Anticipated Bearing Pressure (psf)
Pump Assembly	17,195	6 x 6	478
Pump Maximum Downward Thrust	51,000	6 x 6	1,417

* - Provided by ECO:LOGIC Engineering of Reno, Nevada.

Two 500,000-gallon water storage tanks will be constructed at a site several hundred feet south of the proposed Pump Station site at an approximate elevation of 4,243 feet. It is anticipated that these tanks will be of welded steel construction and have a height of 24 feet and a diameter of 61 feet. We understand that cuts and fills up of to 10 feet may be required to achieve site/tank pad grade at this site.

In addition, a commercial septic system will be constructed at the site to service the pump station structure employee restroom. A septic system design has been prepared for this project as a separate report.

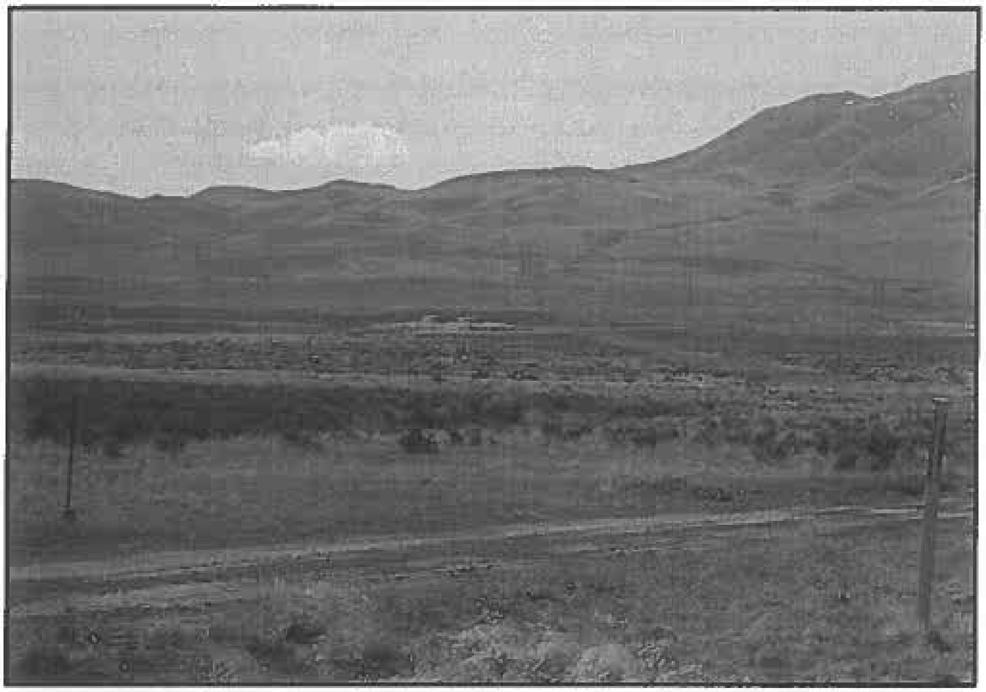

Photo 1 – Proposed Fish Springs Ranch Pump Station and Water Supply Tanks Rendering, final location to be several hundred feet downslope from location shown in rendering (from ECO:LOGIC).

5.1 SITE EXPLORATION PROCEDURES

5.1.1 Field Exploration

The proposed Pump Station and Pump Station Water Storage Tank sites were initially explored on December 7, 2005 by drilling six hollow stem auger borings at the approximate locations indicated on the Site Plan presented in **Appendix A**. Due to the presence of large cobbles and boulders, shallow refusal was encountered at each boring. Due to the shallow refusal of the hollow stem auger borings, a test pit was excavated at each previous boring location with a Cat 330 CL excavator on December 20-21, 2005. Two percolation test pits were also excavated on December 20, 2005 to perform percolation testing for future septic system design.

Due to system design changes by ECO:LOGIC, the proposed Pump Station was relocated several hundred feet downslope from the original location of the initial geotechnical exploration. As a result, two additional test pits were excavated on April 11-12, 2006 at the new pump station location with a Cat 314C excavator. In addition, two percolation test pits were also excavated on April 13, 2006 to perform percolation testing for future septic system design.

Field moisture density measurements were made in Test Pits B-02 (TP-02), B-03 (TP-03), and B-04 (TP-04) to determine in-situ soil density.

Terracon established the boring and test pit locations in the field by taping from the available reference features and by the use of a hand-held, WAAS enabled GPS unit. The boring and test pit locations should be considered accurate only to the degree implied by the methods used to define them. Two borings were drilled and two test pits were excavated within the approximate footprint of the proposed (final location) Pump Station building, four borings were drilled and four test pits were excavated within the approximate footprint of the two Pump Station Water Storage Tanks, and two percolation test pits were excavated near the site of the proposed septic system disposal field (final location). The borings were advanced with a balloon-tire CME-750 drill rig, utilizing 6-inch outside-diameter, continuous-flight hollow-stem augers.

The subsurface soils were sampled during drilling by using a standard 2-inch outside diameter split-spoon sampler driven by a cathead-type 140-pound hammer with a 30-inch drop using Standard Penetration Testing (SPT). The SPT test is an indication of the density (coarse-grained soils) or consistency (fine-grained soils) of the subsurface soil materials and is defined as the number of blows required to drive the sampler the final 12 inches of an 18-inch total penetration which may be correlated to other soil properties. During test pitting, bulk samples were collected of each major geotechnical unit.

An engineering geologist classified the site soils in the field in general accordance with ASTM D2488. Samples were then returned to our Sparks, Nevada laboratory, and classifications were confirmed or revised based on laboratory testing of selected samples. The descriptions of the soils indicated on the boring and test pit logs are in accordance with the enclosed General Notes and the Unified Soil Classification System (USCS). Classification in this manner provides indications of soil properties and can be correlated to other properties using published charts (Bowles, 1995). Estimated group symbols according to the USCS are given on the boring and test pit logs and a brief description of USCS classification system is shown in **Appendix C**.

5.2 SITE AND SUBSURFACE CONDITIONS

5.2.1 Site Conditions

The proposed Pump Station and Pump Station Water Storage Tank sites are located along the southern margin of Honey Lake Valley in Washoe County, Nevada. At the time of exploration, the site is located in an undeveloped area south of Fish Springs Road with sagebrush vegetation to 5 feet in height. Numerous small drainage channels cross the area, with water flow in drainage channels and by sheet flow.

5.2.2 General Geologic and Soil/Bedrock Conditions

The proposed Pump Station and Pump Station Water Storage Tanks lie within the Honey Lake Valley between the Virginia Mountains to the southeast and the Fort Sage Mountains to the west and south and within the Basin and Range geomorphic province. The Basin and Range geomorphic province is characterized by horizontal extension between the Sierra Nevada Range to the west and the Wasatch Front to the east and is bounded by the Colorado Plateau to the southeast and the Columbia Plateau to the north. This portion of the Honey Lake Valley near the base of the Fort Sage Mountains is generally characterized by alluvial fan deposits from the Fort Sage Mountains. Most of the coarse-grained sediments were deposited quickly during the post-glacial period during flooding events. As a milder climate developed in the region, the volume and size of the sediments were greatly reduced, and geomorphic processes generally changed into reworking of earlier deposited materials.

The Nevada Bureau of Mines and Geology (NBMG) has mapped the proposed sites as consisting of Quaternary fine gravel and sand alluvium. The Quaternary fine gravel and sand alluvium was described as *coalescing alluvial fans form[ing] bajadas that merge with sand and clay deposits of the floor of Honey Lake Valley. The alluvium is composed of gravel- and sand-sized clasts of quartz, feldspar, and volcanic rocks derived from the Fort Sage and Virginia Mountains to the south* (Grose, 1984).

5.2.3 Faulting and Seismicity

The published and available geologic maps indicate that no faults Quaternary or older are mapped as crossing the proposed Pump Station and Pump Station Water Storage Tank sites (Grose, 1984). However, the maps indicate three Quaternary faults are within 1 mile of the two sites to the south. Guidelines for determining and assessing faults have been developed by the Nevada Earthquake Safety Council (NESC, 1998); however, neither the State of Nevada nor various counties have adopted the guidelines. These NESC guidelines are generally consistent with those adopted in the State of California Alquist-Priolo Act of 1972 that defines active faults as those faults with displacement within the past 11,000 years (Holocene) and those faults with displacement within the past 11,000 to 2,000,000 years as potentially active. Based on the geologic mapping, faults in the vicinity of the project site are considered active.

Nevada ranks as the fourth highest state of earthquake occurrence of Magnitude 3.5 and greater, behind Alaska, California, and Hawaii (USGS, 2005). In addition, the Truckee Meadows lies in one of the most seismically active regions of Nevada. dePolo et al., in a study of earthquake occurrence in the Reno-Carson City urban corridor to the south of the proposed substation site, state that *overall, the probabilities of potentially damaging earthquakes within the region are relatively high and are commensurate with many parts of*

California, a state with a well-recognized high earthquake hazard (dePolo et al., 1997). These probabilities are also valid for the area encompassed by this project. As a result of seismicity present, the project should be designed accordingly.

5.2.4 Flooding

To determine relative flooding hazard, Terracon referenced the available Federal Emergency Management Agency (FEMA) FIRM maps. The referenced flooding data is provided for informational purposes only. Since flooding and site drainage are not the responsibility of Terracon, we recommend that the site Civil Engineer be consulted concerning flooding and other drainage hazards.

FEMA has mapped the proposed Pump Station, Pump Station Water Storage Tank, and Pump Station Access Road alignment sites as of 1994 as lying within unshaded Zone X, and is defined as *areas determined to be outside [the] 500-year floodplain* (FEMA, 1994). However, the site lies on an alluvial fan of the Fort Sage Mountains which may experience shallow sheet flow during rain and/or snow precipitation events. In addition, several emphreally active drainage channels are present adjacent to the proposed sites. Flow from overland sheet flow and within the existing, natural drainage channels will need to be adequately routed around the proposed sites. Drainage modifications should include necessary design elements and/or surface treatments to reduce erosion hazard, due to the potentially highly erodible surface soils at the project sites.

5.2.5 Subsurface Conditions

As presented on the test pit logs, the subsurface soils at the proposed Pump Station (final location) site generally consist of dense to very dense *Silty Sand* to *Silty Sand with Gravel (SC)* from the existing ground surface to between 9 and 11 feet below the surface. The subsurface soils at the proposed Pump Station Water Storage Tank site generally consists of medium dense *Poorly Graded Sand with Gravel (SP)* overlying medium dense to dense *Well Graded Gravel with Sand (GW)* from the existing ground surface to between 10.3 and 15 feet below the surface. Significant boulders and cobbles to approximately 4 feet in diameter were also observed within the subsurface soils. The subsurface soils encountered generally correspond with those mapped by the NBMG.

Conditions encountered at each boring or test pit location are indicated on the individual boring and test pit logs. It should be noted that inconsistencies could exist away from, or between, boring and test pit locations. Stratification boundaries on the boring and test pit logs represent the approximate location of changes in soil types; in-situ, the transition between native materials at this site is typically gradual.

5.2.6 Groundwater Conditions

Groundwater was not encountered in any of our borings or test pits at the proposed Pump Station or Pump Station Water Storage Tank sites. Due to the lack of encountered groundwater, we do not anticipate that groundwater will pose any constraints to site development.

5.2.7 Laboratory Testing

Soils testing performed by Terracon were conducted in general accordance with the American Society for Testing and Materials (ASTM), standards where applicable. Representative samples of major material types were tested to determine their grain size distribution (ASTM D422), plasticity index or Atterberg Limits (ASTM D4318), and moisture content (ASTM D2216). These test results are used to properly classify the soils and to adjust the field logs if necessary. Such testing is typically referred to as index testing. These index tests can be correlated to empirical studies to obtain representative values for other engineering properties. Based on index testing, we can estimate design parameters and assign additional testing, as necessary.

Soluble sulfate testing (EPA Method 300.0) was performed on a representative sample of foundation subgrade soil from Test Pit TP-07. These results are used to determine the reactive potential of concrete in contact with native soils. Chemical testing was performed by Western Environmental Testing Laboratory (WETLab) of Sparks, Nevada (see **Appendix B**).

5.3 ENGINEERING RECOMMENDATIONS

Our recommendations are based on the assumption that the soil conditions are similar to those disclosed by the explorations. If variations are noted during construction or if changes are made in site plan, structural loading, foundation type or floor level, we should be notified so we can supplement our recommendations, as applicable.

5.3.1 Geotechnical Considerations

The site appears suitable for the proposed construction based on geotechnical conditions encountered in the exploratory borings and test pits. Cobbles and boulders to approximately 4 feet in diameter were encountered during geotechnical exploration and may pose excavation difficulty.

Based on the subsurface exploration, geotechnical analyses, and laboratory test results, we recommend that the proposed Pump Station building be supported on a spread footing foundation system.

If structures, concrete flatwork, utilities, or other improvements are located in the vicinity of any of the test pits, the test pit backfill should be removed and recompacted in accordance with **Section 5.3.9.3** - Overexcavation. Failure to properly compact backfill could result in excessive settlement of improvements located over test pits.

Recommendations regarding the design and construction of foundations, and the support of floor slabs and pavements, relative to the subsurface conditions encountered in the test pits and borings, are presented in the following sections of this report.

5.3.2 Seismic Design Criteria

The subject Pump Station and Pump Station Water Storage Tank sites are located in northwestern Nevada, which is a seismically active area within the Basin and Range area. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, and on the intensity and the magnitude of the seismic event.

In order to estimate the parameters for seismic ground motions at the subject site, Terracon utilized the USGS website, *"Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2002 Data"* (USGS, 2005) in general accordance with the 2003 *International Building Code*. According to this source, the peak accelerations at the Pump Station and Pump Station Water Storage Tank sites are shown in Table 14.

Based on our site exploration, we suggest a Site Class definition "D" for this site (Table 1615.1.5, 2003 *International Building Code*). This site class definition corresponds to a *stiff soil profile*.

Table 14 – Pump Station and Pump Station Water Supply Tank Site Seismic Criteria

Ground Motion	10% Possibility of Exceedence in 50 Years (%g)	2% Possibility of Exceedence in 50 Years (%g)
Peak Ground Acceleration (PGA)	25.48	48.05
0.2 sec Spectral Acceleration (SA)	62.03	117.59
1.0 sec Spectral Acceleration (SA)	21.62	44.32

5.3.3 Pump Station Water Storage Tank Design Criteria

As requested, we are providing recommended design criteria for the Pump Station Water Storage Tanks as developed by the American Water Works Association (AWWA). Recommended design criteria based on the AWWA D100-96 Standard for Welded Steel Tanks for Water Storage (AWWA, 1996) are shown in Table 15 below.

Table 15 – AWWA Pump Station Water Storage Tanks Design Criteria

Design Parameter	Value
Seismic Zone (Figure 7)	4
Zone Coefficient (Table 24)	0.40
Soil Profile Type (Table 27)	B
Site Amplification Factor (Table 27)	1.2
Poisson's Ratio	0.35
Young's Modulus (Modulus of Elasticity)	1,350 ksf

5.3.4 Liquefaction

Based on the lack of saturated, clean sand soils encountered during exploration, it is our opinion that liquefaction potential at this site is minimal. Should groundwater levels rise in the vicinity of the proposed Pump Station or Pump Station Water Storage Tank sites in the future, due to climatic or other changes, liquefaction potential may be an issue due to the presence of clean sand soils and should be re-addressed at that time.

5.3.5 Pump Station Foundations

Conventional shallow foundations, such as spread footings, are appropriate for supporting the proposed Pump Station structure at this site. Spread footings should be embedded a minimum of 24 inches below the nearest adjacent grade for frost protection and should bear on a minimum of 12 inches of compacted granular structural fill. Spread footings bearing on a minimum of 12 inches of properly compacted granular structural fill can be designed for a maximum net allowable soil bearing pressure of 3,000 pounds per square foot (psf) (assuming a minimum embedment depth of 24 inches within the structural fill). The net allowable soil bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation.

The magnitude of settlement that will occur beneath the foundation would depend upon the variations within the subsurface soil profile, the actual structural loading conditions, the embedment depth of the footings, the actual thickness of compacted fill, and construction quality control. Assuming that spread footing construction is performed in accordance with our recommendations, it is our opinion that total settlement will be on the order of approximately 1 inch. Differential settlement up to approximately two-thirds of the total settlement value should be anticipated.

It is recommended that the building excavation be evaluated by the geotechnical engineer during excavation, but prior to structural fill or concrete placement. If, during evaluation, unsuitable materials are encountered, the geotechnical engineer may require that the footing subgrade be modified by additional overexcavation and subsequent structural fill placement.

Properly compacted structural fill in footing areas should be free of water and loose soil prior to placing concrete. Concrete should be placed as soon as possible after excavation to minimize disturbing the bearing material. Should the material at bearing level become disturbed or saturated, the affected soil should be removed and replaced prior to placing concrete

5.3.6 Pump Station Water Storage Tanks

Conventional shallow foundations, such as spread footings, are appropriate for supporting the Pump Station Water Storage Tanks at this site. The Pump Station Water Storage Tank foundations, including ringwall footings and interior column footings, should be embedded a minimum of 24 inches below the nearest adjacent grade for frost protection and should bear on a minimum of 12 inches of compacted granular structural fill. Spread footings bearing on a minimum of 12 inches of properly compacted granular structural fill can be designed for a maximum net allowable soil bearing pressure of 3,500 pounds per square foot (psf) (assuming a minimum embedment depth of 24 inches within the structural fill and a minimum footing width of 24 inches). The net allowable soil bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation.

The magnitude of settlement that will occur beneath the foundation would depend upon the variations within the subsurface soil profile, the actual structural loading conditions, the embedment depth of the footings, the actual thickness of compacted fill, and construction quality control. Assuming that spread footing construction is performed in accordance with our recommendations, it is our opinion that total settlement will be on the order of approximately 1 inch. Differential settlement up to approximately two-thirds of the total settlement value should be anticipated.

It is recommended that the footing excavation be evaluated by the geotechnical engineer during excavation, but prior to structural fill or concrete placement. If, during evaluation, unsuitable materials are encountered, the geotechnical engineer may require that the footing subgrade be modified by additional overexcavation and subsequent structural fill placement.

Properly compacted structural fill in footing areas should be free of water and loose soil prior to placing concrete. Concrete should be placed as soon as possible after excavation to minimize disturbing the bearing material. Should the material at bearing level become disturbed or saturated, the affected soil should be removed and replaced prior to placing concrete.

5.3.7 Lateral Earth Pressures

Lateral loads may be resisted by soil friction and by the passive resistance of the soil. An allowable coefficient of friction of 0.35 may be used between foundations or floor slabs and the supporting structural fill soils. The ultimate passive resistance of properly compacted fill may be assumed to be equal to the pressure developed by a fluid with a density of 290 pounds per cubic foot (pcf). The ultimate active resistance of properly compacted fill may be assumed to be equal to the pressure developed by a fluid with a density of 38 pcf. A one-third increase may be used for wind or seismic loads. The passive pressure and the frictional resistance of the soils may be combined without reduction in determining the total lateral resistance. Appropriate factors of safety should be applied to ultimate values.

These lateral load recommendations assume structural fill against foundations is properly placed and compacted. Backfill should be mechanically compacted in layers (8-inch maximum thickness) to a minimum of 95 percent relative compaction (ASTM D1557); flooding should not be permitted. Care should be taken when placing backfill so as not to damage the footings. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors to minimize damage.

5.3.8 Permanent Slopes

Permanent slopes are anticipated along the southem and eastern boundaries of the proposed Pump Station pad area and along most of the exterior of the Pump Station Water Storage Tank pad area. These slopes should be no steeper than 2.5H:1V (Horizontal:Vertical). At a 2.5H:1V geometry, minor sloughing of the slope surface should be expected. Any required erosion control measures should be provided for all slopes as soon as possible after grading. Drainage from the toe of the slope should be directed around the site and away from the structure. Flow from overland sheet flow and within the existing, natural drainage channels will need to be adequately routed around the proposed sites. Drainage modifications should include necessary design elements and/or surface treatments to reduce erosion hazard, due to the potentially highly erodible surface soils at the project site.

5.3.9 Earthwork

5.3.9.1 Site Clearing

Existing topsoil and other deleterious materials are to be removed from the proposed sites to a minimum depth of 8 inches in driveway, parking, and non-structural areas and to a minimum depth of 12 inches within the building and tank footprints and extending at least 3 feet in all directions from the building and tank footprints. All materials derived from the site stripping operations should be removed from the site and not used in any fills or backfills. Grading work should not be performed on or with frozen soils.

5.3.9.2 Excavation

Based on the soils encountered during the exploration, we anticipate excavation difficulties in the form of cobbles and/or boulders. Contractors should satisfy themselves as to excavation characteristics before bidding the project.

Excavations or trenches should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions. All excavations or trenches should comply with all applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that Terracon is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

5.3.9.3 Overexcavation

It is anticipated that minor fills on the order of 2 feet in thickness will be required to bring the proposed Pump Station site to finished grade. In addition, fills are also anticipated at the proposed Pump Station Water Storage Tank site. Additionally, it is recommended that foundations be embedded a minimum of 24 inches for frost protection and that foundations bear on a minimum of 12 inches of structural fill. Therefore, minor overexcavations can be anticipated in footing areas where import fills are less than 3 feet in thickness in order to achieve the required 12 inches of structural fill beneath footings. In addition, overexcavations are anticipated below the five proposed Pump Station water pumps to allow installation of 30-inch-long suction cans.

Overexcavations should be cleaned of loose or deleterious material and backfilled with compacted granular structural fill. Structural fill material below footings and the proposed

Pump Station Water Storage Tanks should be placed in maximum 8-inch-thick loose lifts and compacted to 95 percent relative compaction (ASTM D1557). The overexcavation and backfill procedure is shown in Figure 3 below. The guidelines for imported structural fill are presented in Table 2 – Guidelines for Imported Structural Fill presented below in **Section 5.3.9.4** - Fill Materials. It should be the contractor's responsibility to ensure that the bottom of the excavation is prepared in such a manner as to allow the specified minimum compaction of the structural fill to be achieved.

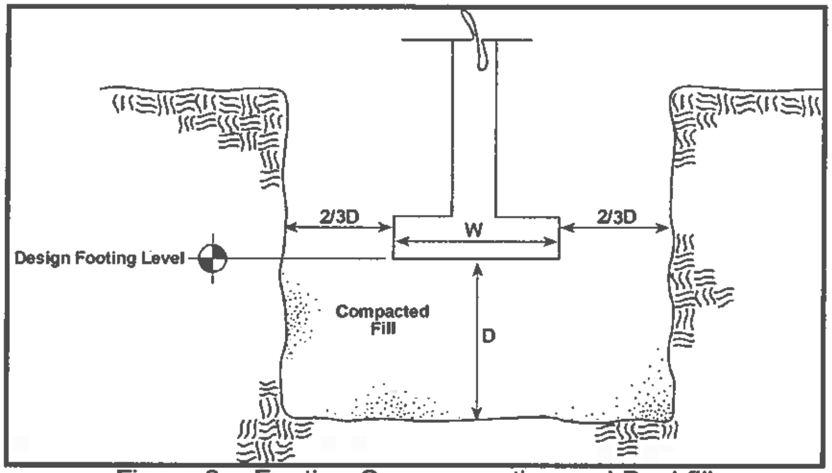

Figure 3 – Footing Overexcavation and Backfill

5.3.9.4 Fill Materials

Native granular soils are suitable for re-use as structural fill beneath structural improvements given that particles greater than 6 inches are removed. Oversized material (cobbles and boulders) removed from the native soils may be suitable as rip-rap. All imported structural fill materials should comply with the guidelines presented in Table 16, Guidelines for Imported Structural Fill.

Table 16 – Guidelines for Imported Structural Fill (ASTM C136)	
Sieve Size	Percent Passing
4-Inch	100
¾-Inch	70 - 100
No. 40	15 - 70
No. 200	5 - 30
Maximum Allowable Liquid Limit and Plasticity Index (ASTM D4318)	
Liquid Limit (LL)	35 max
Plasticity Index (PI)	10 max
Minimum Allowable R-Value (ASTM D2844)	
R-Value	45 min
Maximum Allowable Soluble Sulfates (EPA 300)	
Sulfates	< 0.1% by Weight SO_4

Materials that deviate from these guidelines must be approved by the geotechnical engineer prior to use.

5.3.9.5 Fill Placement and Compaction

Areas to receive fill should be scarified to a minimum depth of 6-inches, moisture conditioned, and compacted to a minimum of 90 percent relative compaction (ASTM D1557). Areas that are unstable or pumping should be stabilized to a firm and non-yielding condition prior to receiving fill.

All structural fill materials should comply with the requirements presented in Table 14 – Guidelines for Imported Structural Fill. Structural fill should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557) at the proposed Pump Station site and a minimum of 95 percent at the proposed Pump Station Water Storage Tanks. The moisture content of structural fill at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

5.3.9.6 Trench Backfill

Native granular soils may be utilized as intermediate utility line backfill. It is anticipated that native cobbles and/or boulders will be too coarse for use as trench backfill and should be screened and removed. Particles larger than 4 inches in diameter should be removed.

Native materials used as intermediate trench backfill should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557). The moisture content of native soils at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

5.3.10 Floor Slab Design and Construction

We recommend that the proposed Pump Station floor slab and concrete entryways be supported on a minimum of 12 inches of structural fill (including aggregate base section) to provide more uniform support and to reduce the potential for differential movement. Some differential movement of a slab-on-grade floor system is possible should the subgrade soils experience fluctuation in moisture content. Such movements are anticipated to be within general tolerance for normal slab-on-grade construction. To reduce potential slab movements, the subgrade soils should be prepared as outlined in **Section 5.3.9** - Earthwork - section of this report.

Additional floor slab design and construction recommendations are as follows:

- Positive separations and/or isolation joints should be provided between slabs and all foundations, columns or utility lines to allow independent movement.

- Control joints should be provided in slabs to control the location and extent of cracking.

- Interior trench backfill placed beneath slabs should be compacted in accordance with recommendations outlined below.

- Floor slabs should not be constructed on frozen subgrade.

- Other design and construction considerations, as outlined in the ACI *Design Manual*, Section 302.1R are recommended.

The concrete floor slab may be supported on a 6-inch-thick layer of compacted aggregate base. The purpose of the aggregate base section is to help distribute concentrated loads. Note that the 6-inch layer of compacted aggregate base can account for a portion of the overexcavation.

Recommendations presented by the American Concrete Institute for slabs-on-grade should be complied with for all concrete placement and curing operations. Improper curing techniques and/or excessive slump (water-cement ratio) could cause excessive drying/shrinkage resulting in random cracking and/or slab curling. Concrete slabs should be allowed to cure adequately before placing vinyl or other moisture-sensitive floor coverings.

For design purposes, a modulus of subgrade reaction (*K*) equal to 250 pounds per cubic inch (pci) should be used assuming that 12 inches of properly compacted structural fill are placed below the slab-on-grade (Huang, 2004).

5.3.11 Pavements

We have calculated a section design based on post-construction traffic only. For this design, we have assumed one truck trip and one pickup truck (equivalent to car loading) will occur per week at the proposed Pump Station for chlorine delivery and maintenance activities. We have utilized a conservative one trip per day for our analysis. Assuming an approximate 20-year design life, an equivalent single axle load (ESAL) of 179 was estimated for section design corresponding to a Traffic Index (TI) of 3.2.

Using the estimated traffic ESAL data described above, native subgrade soils with a minimum R-value of 45, and the CalTrans low volume road pavement design, we recommend that the post-construction option road section consist of a minimum of 6 inches of Type 2, Class B (*Standard Specifications for Public Works Construction*, latest edition) aggregate base.

The placement and compaction practices should conform to the *Standard Specifications for Public Works Construction*, latest edition. The aggregate base section should be compacted to a minimum of 95 percent relative compaction (ASTM D1557) in a maximum 6-inch lift.

In addition, overexcavation of surficial clay soils will be necessary to a depth of approximately 1 to 1.5 feet along the northern portion of the proposed realigned Pump Station access road. It is anticipated that the initial 1,500 to 2,000 lineal feet of roadway south of the existing Fish Springs Road will require overexcavation.

Design and construction of the proposed realigned Pump Station Access Road should also note flooding and erosion potential as described in **Section 5.2.4** – Flooding.

The minimum pavement section outlined above was determined based on nearby test pits, laboratory test results, and post-construction traffic loading conditions for this type of development. These pavement sections do not account for heavy construction traffic during the early stages of the development. A partially constructed structural section may be subjected to heavy construction traffic that can result in pavement deterioration and premature failure. Our experience indicates that this pavement construction practice can result in pavements that will not perform as intended. Considering this information, several alternatives are available to mitigate the impact of heavy construction traffic on the pavement construction. These include using thicker sections to account for the construction traffic, using some method of soil stabilization to improve the support characteristics of the pavement subgrade, or by routing heavy construction traffic around constructed roadways. We are available to discuss these alternatives with you.

6.0 MAIN TRANSMISSION LINE ALIGNMENT

The Main Transmission Line alignment consists of an approximate 28-mile-long transmission main from the proposed Pump Station in the southern end of Honey Lake Valley to the Terminal Tank, located between Antelope Valley and Lemmon Valley. It is anticipated that the Main Transmission Line pipe will have a diameter of 30 inches, an average invert elevation of 6 feet below the existing ground surface, and be constructed of Ductile Iron Pipe (DIP).

Geotechnical exploration has not been completed along the proposed alignment from Test Pits TP-35 to TP-37, TP-39 to TP-41, TP-43 to TP-45, TP-47, TP-49, TP-51, TP-53 to TP-54, TP-57/59 to TP-64, and TP-78 due to on-going environmental clearance issues and from a request from Vidler Water Company/ECO:LOGIC to reduce the geotechnical exploration component of the project. As a result, sub-surface soil, bedrock, and groundwater conditions along these portions of the proposed alignment are unknown.

6.1 SITE EXPLORATION PROCEDURES

6.1.1 Geologic Reconnaissance

Geologic reconnaissance was performed in November, 2005 to provide alignment specific engineering geologic maps along the proposed Main Transmission Line alignment. Available geologic maps for the proposed alignment do not well indicate surficial soils and as a result, are not well suited for engineering and construction recommendations. In addition, as the best available geologic map for the portion of the proposed alignment is mapped at a scale of 1:250,000, more detailed geologic mapping was necessary. Engineering geologic mapping of the proposed alignment is shown on Plates 2a to 2f and presented in **Appendix A**. Table 17 below summarizes the estimated depth to bedrock results from our geologic reconnaissance for areas where bedrock is estimated to be less than 8 feet below the existing ground surface. <u>Bedrock is expected in portions of the unexplored area due to the presence of encountered bedrock in nearby test pits</u>.

Table 17 – Estimated Depth to Bedrock Reconnaissance Summary	
Stationing	**Estimated Depth (ft)**
45+00 to 58+00	2 – 8
85+00 to 91+00	2 – 8
102+00 to 105+00	2 – 8
178+00 to 197+00	2 – 8
209+00 to 221+00	2 – 8
221+00 to 224+00	< 2
224+00 to 229+00	2 – 8
275+00 to 370+00	2 – 8
520+00 to 530+00	2 - 8
635+00 to 644+00	2 – 8
1033+00 to 1055+00	2 – 8
1064+00 to 1075+00	2 – 8
1160+00 to 1170+00	2 – 8
1261+00 to 1270+00	2 - 8
1270+00 to 1278+00	<2
1278+00 to 1290+00	2 – 8
1474+00 to 1477+00	2 – 8

Geologic reconnaissance performed by Terracon mapped the proposed alignment based on a literature review of existing geologic maps and reports and from field reconnaissance along the proposed alignment.

6.1.2 Field Exploration

The proposed Main Transmission Line alignment was explored on December 9 and 12-14, 2005, March 24 and 27-28, 2006, April 13, 2006, and May 2-3, 2006 by excavating 57 test pits and drilling one boring at the approximate locations indicated on the Site Plans presented in **Appendix A**. Terracon established the test pit and boring locations in the field by taping from the available reference features and by the use of a hand-held WAAS enabled GPS unit. The test pit and boring locations should be considered accurate only to the degree implied by the methods used to define them. One boring was drilled along the proposed Main Transmission Line alignment for subsequent liquefaction analysis. The test pits were excavated with a either a Case 580 rubber-tired backhoe or Cat 314C trackhoe. The boring was advanced with a balloon-tire CME-750 drill rig, utilizing 6-inch outside-diameter, continuous-flight hollow-stem augers.

During test pit excavation, each major geotechnical soil unit was sampled with samples placed in appropriate sealed containers.

During drilling, the subsurface soils were sampled by using a standard 2-inch outside diameter split-spoon sampler or a Modified California 3-inch outside diameter split-spoon sampler driven by a cathead-type 140-pound hammer with a 30-inch drop using Standard Penetration Testing (SPT). The SPT test is an indication of the density (coarse-grained soils) or consistency (fine-grained soils) of the subsurface soil materials and is defined as the number of blows required to drive the sampler the final 12 inches of an 18-inch total penetration which may be correlated to other soil properties. Penetration values recorded from the Modified California Sampler have not been corrected to the standard SPT N-values.

An engineering geologist classified the site soils in the field in general accordance with ASTM D2488. Samples were then returned to our Sparks, Nevada laboratory, and classifications were confirmed or revised based on laboratory testing of selected samples. The descriptions of the soils indicated on the boring and test pit logs are in accordance with the enclosed General Notes and the Unified Soil Classification System (USCS). Classification in this manner provides indications of soil properties and can be correlated to other properties using published charts (Bowles, 1995). Estimated group symbols according to the USCS are given on the test pit and boring logs and a brief description of USCS classification system is shown in **Appendix C**.

6.2 SITE AND SUBSURFACE CONDITIONS

6.2.1 Site Conditions

The proposed Main Transmission Line alignment consists of a 30 inch diameter ductile iron pipe (DIP) transmission main from the proposed Pump Station at the southern end of Honey Lake Valley to the proposed Terminal Water Storage Tank between Antelope Valley and Lemmon Valley.

At the time of our geotechnical exploration, the portion of the alignment from the Pump Station at Station 0+00 to the end of Antelope Valley Road at approximately Station 1198+00 was undeveloped. The remainder of the alignment follows Antelope Valley Road and Matterhorn Drive through a generally rural developed area. The proposed alignment generally follows the existing Tuscarora gas line alignment from near the proposed Pump Station to approximately Station 965+00.

6.2.2 General Geologic and Soil/Bedrock Conditions

The proposed Main Transmission Line alignment lies within the Basin and Range geomorphic province. The Basin and Range geomorphic province is characterized by horizontal extension between the Sierra Nevada Range to the west and the Wasatch Front to the east and is bounded by the Colorado Plateau to the southeast and the Columbia Plateau to the north. The proposed alignment crosses several mapped geologic units which are described starting at the proposed Pump Station and ending at the Terminal Water Storage Tank. Most of the proposed alignment traverses coarse-grain alluvial fan or alluvial fan derived sediments. Most of the coarse-grained sediments were deposited quickly during the post-glacial period during flooding events. As a milder climate developed in the region, the volume and size of the sediments was greatly reduced, and geomorphic processes generally changed into reworking of earlier deposited materials.

6.2.2.1 Nevada Bureau of Mines and Geology (NBMG) Geologic Mapping

From the proposed Pump Station to approximately 1.5 miles southeast, the alignment lies within the Honey Lake Valley between the Virginia Mountains to the southeast and the Fort Sage Mountains to the west and south. This portion of the Honey Lake Valley near the base of the Fort Sage Mountains is generally characterized by alluvial fan deposits from the Fort Sage Mountains. The Nevada Bureau of Mines and Geology (NBMG) has mapped this portion of the alignment as consisting of Quaternary fine gravel and sand alluvium and Quaternary boulder and sand colluvium. The Quaternary fine gravel and sand alluvium was described as *coalescing alluvial fans form[ing] bajadas that merge with sand and clay deposits of the floor of Honey Lake Valley. The alluvium is composed of gravel- and sand-sized clasts of quartz, feldspar, and volcanic rocks derived from the Fort Sage and Virginia Mountains to the south* (Grose, 1984). The Quaternary boulder and sand colluvium was described as *course detritus, with granitic boulders up to 10 feet in diameter* (Grose, 1984).

From approximately 1.5 miles southeast of the Pump Station, extending southeast to approximately 1 mile south of the proposed Surge Tank, the proposed alignment crosses a series of related Tertiary volcanics. The NBMG has described these volcanics as Tertiary crystal vitric rhyolite tuff and welded tuff. The Tertiary crystal vitric rhyolite tuff was described as *white, gray, pink, tan; fine to coarse vitroclase matrix with broken crystals of sanidine, sodic oligoclase to sodic andesine, less quartz, and minor biotite; some bedded ash-fall sections with pumice lapilli and minor dark lithic felsite fragments; mostly nonbedded and nonwelded, uniform, and massive ash-flow deposit.* The Tertiary welded tuff was described as *moderately to strongly welded portions of the crystal vitric rhyolite tuff* (Grose, 1984).

Approximately 1 mile south of the Surge Tank, the proposed alignment crosses Dry Valley. Dry Valley is located between the Fort Sage Mountains to the northwest and the Virginia Mountains to the east. The NBMG has mapped Dry Valley as Quaternary Alluvium and has described it as *Stream deposits, talus, slope wash, alluvial fan and eolian deposits* (Bonham, 1969).

South of Dry Valley, the proposed alignment crosses Bedell Flat. Bedell Flat is a northwest trending valley located between Petersen Mountain to the west and the Dogskin Mountains to the east. The NBMG has mapped the proposed alignment in Bedell Flat as Quaternary alluvial plain and undifferentiated alluvial deposits and has described them as *unconsolidated sand and gravelly sand, predominantly arkosic; sheetwash, sidestream, and wash alluvium in predominantly Holocene to modern channels or as broad, low-gradient alluvial plains* (Garside, 1993).

The Main Alignment then crosses Bedell Flat into Antelope Valley through an area mapped by the NBMG as Mesozoic granodiorite of Golden Valley. The Mesozoic granodiorite of Golden Valley was described as *[g]ray hornblende-biotite granodiorite, equigranular to porphyritic, medium grained. Resistant to weathering, forming blocky to spheroidal corestone outcrops surrounded by grus* (Garside, 1993).

The proposed alignment then crosses Antelope Valley, which is a between Fred's Mountain to the west and the Pah Rah Range to the east. The NBMG has mapped this portion of the proposed alignment through Antelope Valley as Quaternary lake deposits and Quatemary alluvial plain and undifferentiated alluvial deposits. The Quaternary lake deposits was described as *lacustrine silt and clay, and beach bar and forebeach deposits*, and the Quaternary alluvial plain and undifferentiated alluvial deposits are decrribed as *unconsolidated sand and gravelly sand, predominantly arkosic; sheetwash, sidestream, and wash alluvium in predominantly Holocene to modern channels or as broad, low-gradient alluvial plains* (Garside, 1993).

The proposed alignment ends at the proposed Terminal Water Storage Tank that lies to the north of Lemmon Valley on the southern flank of Fred's Mountain, to the west of the Pah Rah Range. The site of the proposed Terminal Water Storage Tank is generally characterized by quartz monzonite. The quartz monzonite is described as *pink to pale-gray, massive, medium to coarse-grained, equigranular to porphyritic quartz monzonite to granite. Includes extensive aplite, graphic granite, quartz veins, and pegmatite dikes. Generally deeply weathered; forms low, rounded outcrops* (Cordy, 1985).

6.2.2.2 Geologic Reconnaissance Mapping

Geologic reconnaissance performed by Terracon mapped the proposed alignment based on a literature review of existing geologic maps and reports and from field reconnaissance along the proposed alignment.

The portion of the alignment from the Pump Station to approximately Station 210+00 was mapped as Quaternary fan gravels. The Quaternary fan gravels are described as *Brown to grayish-brown, silty, gravelly sand and sandy gravel. Similar to Quaternary Fan deposits* *but with an abundance of coarse sand, gravel, cobbles, and boulders; generally non to weakly cemented; excavatable with little to moderate effort.*

The portion of the alignment from approximately Station 210+00 to Station 250+00 was mapped as Quaternary alluvium and was described as *light brown to yellowish-brown, silty sand and gravel; Generally non to weakly cemented and easily excavatable; Composed of sheet wash, stream channel and other alluvial deposits.*

The portion of the alignment from approximately Station 250+00 to Station 340+00 was mapped as Quaternary older alluvium and was described as *Light brown to brown, clayey and silty sand with some gravel. Generally weak to moderately cemented. Excavatable with little to moderate effort.*

The portion of the alignment from approximately Station 340+00 to 405+00 was mapped as Quaternary fan deposits and was described as *brown, to grayish-brown, silty sand with some gravel. Generally non to weakly cemented. Easily excavatable. May consist of eolian and beach deposits.*

From approximately Station 405+00 to Station 580+00, the alignment was mapped as Quaternary older alluvium. From approximately Station 580+00 to Station 1000+00, the alignment was mapped as Quaternary fan deposits. From approximately Station 1000+00 to Station 1025+00, the alignment was mapped as Quaternary alluvium. From approximately Station 1025+00 to Station 1140+00, the alignment was mapped as Quaternary fan deposits.

From approximately Station 1140+00 to Station 1175+00, the alignment was mapped as Quaternary fan gravels. From approximately Station 1175+00 to Station 1200+00, the alignment was mapped as Quaternary fan deposits. From approximately Station 1200+00 to Station 1255+00, the alignment was mapped as Quaternary alluvium. From approximately Station 1255+00 to Station 1260+00, the alignment was mapped as Quaternary fan gravels.

From approximately Station 1260+00 to Station 1285+00, the alignment was mapped as Mesozoic granitic rocks and was described as *light gray to pinkish-gray. Ranges in composition from granodiorite to quartz monzonite. Variably wethered from fresh to very severe. Excavatable with moderate to significant effort. Blasting may be required in some areas.*

From approximately Station 1285+00 to Station 1295+00, the alignment was mapped as Quaternary fan gravels. From approximately Station 1295+00 to Station 1320+00, the alignment was mapped as Quaternary fan deposits. From approximately Station 1320+00 to Station 1340+00, the alignment was mapped as Quaternary alluvium.

From approximately Station 1340+00 to Station 1370+00, the alignment was mapped as Quaternary lake deposits and was described as *pale yellow to gray, silty sand, easily excavatable.*

From approximately Station 1370+00 to Station 1375+00, the alignment was mapped as Quaternary beach bar deposits. These are described as *gray to grayish brown, poorly-graded sand and silty sand. Very thinly bedded, weakly to moderately cemented, easily excavatable.*

From approximately Station 1375+00 to Station 1400+00, the alignment was mapped as Quaternary sands-undifferentiated. These are described as *Light brown to yellowish brown, poorly graded sand and silty fine sand. Generally non-cemented, easily excavatable. May consist of eolian and beach deposits.*

From approximately Station 1400+00 to Station 1420+00, the alignment was mapped as Quaternary alluvium. From approximately Station 1420+00 to Station 1430+00, the alignment was mapped as Quaternary sands-undifferentiated. From approximately Station 1430+00 to Station 1480+00, the alignment was mapped as Quaternary fans. And from approximately Station 1480+00 to the end of the alignment at the Terminal Water Storage Tank, the alignment was mapped as Quaternary fan gravels.

6.2.3 Faulting and Seismicity

Nevada ranks as the fourth highest state of earthquake occurrence of Magnitude 3.5 and greater, behind Alaska, California, and Hawaii (USGS, 2005). In addition, the Warm Springs Fault Zone, consisting of a main fault trace and a number of secondary faults, or splays, is one of the top 5 most seismically active regions of Nevada (dePolo, 2006). It should be noted that the Main Alignment closely follows the trend of the Warm Springs Fault Zone.

dePolo states that the Warm Springs Fault Zone has had six seismic events prior to 10,000 years ago and two seismic events within the last 10,000 years (Holocene). He states that a large event can be expected approximately every 8,000 years and a small event approximately every 2,000 years. According to dePolo, the main trace of the Warm Springs Fault can be expected to move 6 to 8 feet during an event, and current geodetic information suggests that the region is overdue for an event.

The last major event in the region occurred approximately 9,000 years ago. dePolo et al., in a study of earthquake occurrence in the Reno-Carson City urban corridor, state that *overall, the probabilities of potentially damaging earthquakes within the region are relatively high and are commensurate with many parts of California, a state with a well-recognized high* *earthquake hazard* (dePolo et al, 1997). As a result of seismicity present, the project structure should be designed accordingly.

Guidelines for determining and assessing faults have been developed by the Nevada Earthquake Safety Council (NESC, 1998); however, neither the State of Nevada nor various counties have adopted the guidelines. These NESC guidelines are generally consistent with those adopted in the State of California Alquist-Priolo Act of 1972 that defines active faults as those faults with displacement within the past 11,000 years (Holocene) and potentially active faults as those faults with displacement within the past 11,000 to 2,000,000 years. Based on the geologic mapping, faults in the vicinity of the project site are considered active.

The published and available geologic maps indicate that the Warm Spring Fault Zone and 24 faults (Quaternary or older) are mapped as crossing or within 1 mile of the proposed alignment, according to the NBMG geologic maps (Grose, 1984; Bonham, 1969). The proposed alignment continues south through Bedell Flat, where it crosses 2 faults (Quaternary or older) (Garside, 1993) and terminates at the Terminal Water Storage Tank between Antelope Valley and Lemmon Valley, where it crosses or is within 1 mile of 5 faults (Quaternary or older) (Cordy, 1985). Antelope Valley is a linear geomorphic feature bounded on the west by rangefront faulting. Cordy (1985) describes these five faults as *indeterminate age; predominantly bedrock faults of probable pre-Pleistocene* age *and bedrock-alluvial faults of probable pre-Pleistocene to possible late Pleistocene age. Recent movements are not precluded on faults in this category.*

Once final permitting and other environmental issues are resolved, several fault trenches along the proposed alignment in the Warm Springs Fault Zone will be performed in an attempt to locate and possibly date the mapped faults.

6.2.4 Flooding

To determine relative flooding hazard, Terracon referenced the available Federal Emergency Management Agency (FEMA) FIRM maps. The referenced flooding data is provided for informational purposes only. Since flooding and site drainage are not the responsibility of Terracon, we recommend that the site Civil Engineer be consulted concerning flooding and other drainage hazards.

FEMA has mapped the proposed alignment as of 1994 as lying within unshaded Zone X, and is defined as *areas determined to be outside [the] 500-year floodplain* except where the alignment crosses Dry Valley Creek and the North Fork of Dry Valley Creek. In these areas, FEMA has mapped the parcel as of 1994 as lying within Zone A, and is defined as *special flood hazard areas inundated by 100-year-flood and no base elevation determined.* (FEMA, 1994).

However, the proposed alignment generally lies on alluvial fans of the nearby mountains which may experience shallow sheet flow during rain and/or snow precipitation events. In addition, the proposed alignment crosses the Main, North, and South Forks of Dry Valley Creek and numerous emphreally active drainage channels. Drainage modifications should include necessary design elements and/or surface treatments to reduce erosion hazard, due to the potentially highly erodible surface soils along the proposed alignment.

In addition, water was generally observed flowing in the main, North, and South Forks of Dry Valley Creek during our geotechnical reconnaissance and exploration activities along the proposed alignment.

6.2.6 Subsurface Conditions

As presented on the boring and test pit logs, the subsurface soils along the proposed alignment generally consist of granular *Poorly Graded Sand with Silt (SP-SM)* and *Silty Sand (SM)* soils. Refusal was experienced in several test pits due to caving conditions or shallow bedrock. A summary of subsurface conditions along the proposed alignment is shown in Table 18.

Table 18 – Main Transmission Line Alignment Subsurface Conditions Summary

Location	USCS Soil Type at Approximately 6 Feet	Exploration Depth (ft)	GWT (ft)	Excavation Refusal (ft)
TP-01	Poorly Graded Sand with Silt (SP-SM)	12	NE	NE
TP-02	Poorly Graded Sand with Silt (SP-SM)	10.5	NE	10.5*
TP-03	Silty Sand (SM)	9.5	NE	9.5*
TP-04	Silty Sand (SM)	10	NE	10*
TP-05	Silty Sand (SM)	12	NE	NE
TP-06	Silty Sand (SM)	12	NE	NE
TP-07	Poorly Graded Sand with Silt (SP-SM)	12	NE	NE
TP-08	Silty Sand (SM)	12	NE	NE
TP-09	Silty Sand (SM)	12	NE	NE
TP-10 (B-10)	Poorly Graded Sand (SP)	51.5	20	NE
TP-11	Silty Sand with Gravel (SM)	12	NE	NE
TP-12	Silty Sand (SM)	9.5	NE	9.5*
TP-13	Silty Sand (SM)	11	NE	11*
TP-14	Poorly Graded Sand with Gravel (SP)	12	NE	NE
TP-15	Weathered Bedrock	6	NE	6 – Bedrock
TP-16	---	3	NE	3 – Bedrock
TP-17	---	4	NE	4 – Bedrock
TP-18	Silty, Clayey Sand (SC-SM)	12	NE	NE
TP-19	Silty Sand (SM)	12	NE	NE
TP-20	Poorly Graded Sand with Silt (SP-SM)	12	NE	NE
TP-21	Silty Sand with Gravel (SM)	13.5	NE	NE

Table 18 – Main Transmission Line Alignment Subsurface Conditions Summary				
Location	**USCS Soil Type at Approximately 6 Feet**	**Exploration Depth (ft)**	**GWT (ft)**	**Excavation Refusal (ft)**
TP-22	Silty Sand (SM)	12	NE	NE
TP-23	Silty Sand with Gravel (SM)	5.5	NE	5.5 – Bedrock
TP-24	Silty Sand (SM)	10	NE	NE
TP-25	Silty Sand with Gravel (SM)	11.5	NE	NE
TP-26	Silty Sand (SM)	11.5	NE	NE
TP-27	Silty Sand with Gravel (SM)	12	NE	NE
TP-28	Silty Sand with Gravel (SM)	10	NE	10 – Bedrock
TP-29	Poorly Graded Sand with Silt (SP-SM)	7	NE	7 – Bedrock
TP-30	Silty Sand (SM)	9	NE	9 – Bedrock
TP-31	---	3.5	NE	3.5 – Bedrock
TP-32	Silty Sand with Gravel (SM)	12	NE	NE
TP-33	Silty Sand (SM)	12	NE	NE
TP-34	Silty Sand (SM)	13	NE	NE
TP-38	Silty Sand (SM)	10	NE	10*
TP-42	Silty Sand with Gravel (SM)	12	NE	NE
TP-46	Silty Sand with Gravel (SM)	7.5	NE	7.5 – Boulders
TP-48	Silty Sand with Gravel (SM)	8.5	NE	8.5 – Boulders
TP-50	Silty Sand (SM)	12	NE	NE
TP-52	Silty Sand with Gravel (SM)	12	NE	NE
TP-55	Silty Sand with Gravel (SM)	7	NE	7 – Bedrock
TP-65	---	2	NE	2 – Bedrock
TP-66	---	5	NE	5 – Bedrock
TP-67	Silty Sand (SM)	6.5	NE	6.5 – Bedrock
TP-68	---	4	NE	4 – Bedrock
TP-69	---	4	NE	4 – Bedrock
TP-70	Clayey Sand with Gravel (SC)	12	NE	NE
TP-71	---	3.5	NE	3.5 – Bedrock
TP-72	Clayey Sand with Gravel (SC)	6	NE	6 – Bedrock
TP-73	Poorly Graded Sand with Gravel (SP)	8.5	NE	8.5 - Bedrock
TP-74	---	3	NE	3 – Cemented
TP-75	Poorly Graded Sand with Silt and Gravel (SP-SM)	6	NE	6 – Bedrock
TP-76	Silty Sand (SM)	7	NE	7 – Bedrock
TP-77	Clayey Sand (SC)	11	NE	NE
TP-79	---	5.5	NE	5.5 – Boulders
TP-80	---	5.5	NE	5.5 – Boulders
TP-81	Clayey Sand with Gravel (SC)	11.5	NE	NE
TP-82	Silty Sand with Gravel (SM)	13	NE	NE

Notes: * - Excavation refusal depth represents depth at which trench caving prevented further excavation.

The subsurface soils from Test Pit TP-01 through TP-09 generally consist of loose to medium dense *Silty Sand (SM)* and *Poorly Graded Sand with Silt (SP-SM)*, and *Poorly*

Graded Sand (SP) from the existing ground surface to approximately 12 feet below the surface.

The subsurface soils at Boring TP-10 (B-10) generally consist of medium dense to very dense, *Poorly Graded Sand with Silt (SP-SM)*, *Poorly Graded Sand (SP)* and *Silty Sand (SM)* from the existing ground surface to approximately 21.5 feet below the surface. This material is underlain by *Clayey Sand (SC)* from approximately 21.5 to 24.5 feet below the ground surface. This material is underlain by variable *Silty Sand (SM)* from approximately 24.5 feet to 51.5 feet below the existing ground surface.

The subsurface soils from Test Pit TP-11 through TP-14 generally consist of medium dense to very dense *Silty Sand (SM)* and *Silty Sand with Gravel (SM)* from the existing ground surface to approximately 5.5 to 9 feet below the surface. This material is underlain by *Poorly Graded Sand with Silt (SP-SM)* and *Poorly Graded Sand with Silt and Gravel (SP-SM).*

The subsurface soils from Test Pit TP-15 and TP-16 generally consist of medium dense to very dense *Poorly Graded Gravel with Sand (GP-GM)* from the existing ground surface to approximately 3 to 6 feet below the surface. This material is underlain by bedrock with shallow excavation refusal.

The subsurface soils from Test Pit TP-17 through TP-27 generally consist of medium dense *Silty Sand (SM)*, *Silty Sand with Gravel (SM)*, and *Poorly Graded Sand with Silt (SP-SM)* from the existing ground surface to approximately 3.5 to 12.5 feet below the surface. Shallow refusal was experienced at Test Pit TP-17 due to bedrock.

The subsurface soils at Test Pit TP-28 generally consist of dense *Silty Sand (SM)* from the existing ground surface to approximately 3 feet below the surface. This material is underlain *Clayey Sand (SC)* to 4.5 feet. This material is underlain by *Silty Sand with Gravel (SM)* to approximately 10 feet.

The subsurface soils from Test Pit TP-29 through TP-30 generally consist of dense *Silty Sand (SM)*, *Silty Sand with Gravel (SM)*, and *Poorly Graded Sand with Silt (SP-SM)* from the existing ground surface to approximately 7 to 9 feet below the surface. Shallow refusal was experienced due to cobbles and boulders.

The subsurface soils at Test Pit TP-31 generally consist of dense *Clayey Sand (SC)* and *Clayey Sand with Gravel (SC)* from the existing ground surface to approximately 3.5 feet below the surface. Shallow refusal was experienced due to bedrock.

The subsurface soils from Test Pit TP-32 through TP-33 generally consist of dense *Silty Sand (SM)* and *Silty Sand with Gravel (SM)* from the existing ground surface to approximately 12 feet below the surface.

The subsurface soils at Test Pit TP-34 generally consist of dense *Clayey Sand (SC)* from the existing ground surface to approximately 1.5 feet below the surface. This material is underlain by *Silty Sand (SM)* and *Silty Sand with Gravel (SM)* from 1.5 feet to 13 feet below the existing ground surface.

The subsurface soils from Test Pit TP-38 and TP-42 generally consist of dense *Silty Sand (SM)* and *Silty Sand with Gravel (SM)*, from the existing ground surface to approximately 10 to 12 feet below the surface.

The subsurface soils at Test Pit TP-46 generally consist of dense *Clayey Sand (SC)* from the existing ground surface to approximately 1.5 feet below the surface. This material is underlain by *Silty Sand with Gravel (SM)* from 1.5 feet to 7.5 feet below the existing ground surface. Shallow refusal was experienced on cobbles and boulders at 7.5 feet.

The subsurface soils from Test Pit TP-48 and TP-50 generally consist of *dense Silty Sand (SM), Silty Sand with Gravel (SM)*, and *Poorly Graded Gravel with Silt (GP-GM)* from the existing ground surface to approximately 8.5 to 12 feet below the surface.

The subsurface soils at Test Pit TP-52 and TP-55 generally consist of dense *Clayey Sand (SC)* and stiff, high plasticity *Fat Clay (CH)* from the existing ground surface to approximately 1 foot below the surface. This material is underlain by *Silty Sand with Gravel (SM)* from approximately 7 to 12 feet below the existing ground surface. Refusal was experienced on bedrock at 7 feet in Test Pit TP-55.

The subsurface soils at Test Pits TP-65 through TP-69 generally consist of dense *Silty Sand (SM), Silty Sand with Gravel (SM)*, and *Poorly Graded Sand with Silt and Gravel (SP-SM)* from the existing ground surface to approximately 2 to 6.5 feet below the surface. Refusal was experienced in these test pits due to shallow bedrock.

The subsurface soils at Test Pit TP-70 generally consist of dense *Clayey Sand with Gravel (SC)* from the existing ground surface to approximately 6.5 feet below the surface. This material is underlain by *Poorly Graded Sand with Gravel (SP)* from 6.5 feet to 12 feet below the existing ground surface.

The subsurface soils at Test Pit TP-71 generally consist of very dense *Poorly Graded Sand with Silt and Gravel (SP-SM)* from the existing ground surface to approximately 3.5 feet below the surface. Refusal was experienced at 3.5 feet due to shallow bedrock.

The subsurface soils at Test Pit TP-72 and TP-73 generally consist of *Clayey Sand with Gravel (SC)* and *Fat Clay (CH)* from the existing ground surface to approximately 2 to 6 feet below the surface. The *Fat Clay (CH)* in Test Pit TP-73 was underlain by *Poorly Graded Sand with Gravel (SP)* at 2 feet. Refusal was experienced at both test pits due to shallow bedrock.

The subsurface soils at Test Pit TP-74 through TP-76 generally consist of dense to very dense *Silty Sand (SM)*, *Silty Sand with Gravel (SM)*, and *Poorly Graded Sand with Silt and Gravel (SP-SM)* from the existing ground surface to approximately 7 feet below the surface. Refusal was experienced in these test pits due to shallow bedrock.

The subsurface soils at Test Pit TP-77 and TP-79 through TP-81 generally consist of mediurm dense to dense *Clayey Sand (SC)* and *Clayey Sand with Gravel (SC)* from the existing ground surface to approximately 2 to 11 feet below the surface. This material is underlain by *Silty Sand (SM)* and *Silty Sand with Gravel (SM)*. Refusal was experienced in Test Pit TP-79 and TP-80 at 5.5 feet due to bedrock.

The subsurface soils at Test Pit TP-82 generally consist of medium dense to dense *Silty Sand (SM)* and *Silty Sand with Gravel (SM)* from the existing ground surface to approximately 13 feet below the surface.

Conditions encountered at each boring and test pit location are indicated on the individual boring and test pit logs. It should be noted that inconsistencies could exist away from, or between, boring and test pit locations. Stratification boundaries on the boring and test pit logs represent the approximate location of changes in soil types; in-situ, the transition between native materials at this site is typically gradual.

6.2.7 Groundwater Conditions

Groundwater level observations made while drilling and excavating and immediately after completion of the borings test pits are shown in the lower left corner of the boring test pit logs. Groundwater was encountered at approximately 20 feet below the existing ground surface in Boring TP-10 (B-10) in Antelope Valley. We do not anticipate that groundwater will pose significant constraints to construction along the proposed alignment except at natural creek crossings. As water was generally observed flowing in the main, North, and South Forks of Dry Valley Creek during our geotechnical reconnaissance and exploration activities along the proposed alignment, we anticipate shallow ground water in the vicinity of proposed alignment crossing the main, North, and South Forks of Dry Valley Creek and in other large, natural drainages.

The water level observations made during our exploration provide an indication of the groundwater conditions at the time the borings were drilled and test pits were excavated.

Longer monitoring in piezometers or cased holes would be required to evaluate long-term groundwater conditions. Fluctuations in the long-term groundwater levels should be expected throughout the year depending upon variations in the amount of rainfall, runoff, evaporation, and other hydrological conditions not apparent at the time the borings were drilled. In addition, perched water can develop within higher permeability soils overlying less permeable soils following heavy or prolonged precipitation. Therefore, groundwater levels during construction or at other times in the future may be higher or lower than the levels indicated on the boring and test pit logs.

6.2.8 Laboratory Testing

Soils testing performed by Terracon were conducted in general accordance with the American Society for Testing and Materials (ASTM), standards where applicable. Representative samples of major material types were tested to determine their grain size distribution (ASTM D422), plasticity index or Atterberg Limits (ASTM D4318), and moisture content (ASTM D2216). These test results are used to properly classify the soils and to adjust the field logs if necessary. Such testing is typically referred to as index testing. These index tests can be correlated to empirical studies to obtain representative values for other engineering properties. Based on index testing, we can estimate design parameters and assign additional testing, as necessary.

A direct shear test (ASTM D3080) was performed on a representative sample of soil from Boring TP-10 (B-10). These results were used to determine the cohesion and friction angle soil strength properties of the soil.

During geotechnical exploration, we sampled each test pit completed along the proposed Main Transmission Line alignment at an approximate depth of 6 feet below the existing ground surface for subsequent corrosion testing, where possible.

Corrosion testing includes acidity (EPA 2310B), chloride (EPA 300.0), pH (EPA 150.1), redox potential (EPA 2580B), resistivity (EPA 2510B), sulfate (EPA 300.0), and sulfate reducing bacteria (SRB) (EPA BART), as requested by ECO:LOGIC. Corrosion testing was performed by Western Environmental Testing Laboratory (WETLab) of Sparks, Nevada and the results are presented in Table 19.

Table 19 – Main Transmission Line Corrosion Testing Summary

Location	Depth (ft)	Acidity (mg/kg)	Chloride (mg/kg)	pH	Redox Potential (mV)	Resistivity (ohm-cm)	Sulfate (mg/kg)	SRB
TP-01	6	14	<15	6.87	+300	6,600	18	N
TP-02	6	14	<15	6.61	+280	12,000	<15	N
TP-03	6	7.0	<15	7.14	+350	10,000	<15	N
TP-04	6	11	<15	7.58	+300	5,600	<15	N

Table 19 – Main Transmission Line Corrosion Testing Summary								
Location	Depth (ft)	Acidity (mg/kg)	Chloride (mg/kg)	pH	Redox Potential (mV)	Resistivity (ohm-cm)	Sulfate (mg/kg)	SRB
TP-05	6	12	<15	7.29	+310	14,000	<15	N
TP-06	6	8.0	24	7.85	+300	3,800	<15	N
TP-07	6	9.0	<15	8.24	+290	4,100	20	N
TP-08	6	3.0	<15	8.21	+300	5,600	<15	N
TP-09	6	-20	<15	8.61	+330	2,600	<15	N
TP-11	6	3.0	<15	7.50	+340	13,000	<15	N
TP-12	6	5.0	<15	8.06	+360	8,800	<15	P
TP-13	6	2.0	<15	7.19	+300	20,000	18	N
TP-14	6	6.0	<15	7.91	+280	11,000	<15	N
TP-15	6	5.0	<15	8.13	+310	4,200	16	N
TP-16	1	7.0	<15	6.56	+300	20,000	<15	N
TP-17	2.5	3.0	<15	6.41	+300	10,000	<15	N
TP-18	6	-2.0	<15	7.91	+330	2,400	<15	N
TP-19	6	-8.0	<15	8.15	+350	1,900	31	N
TP-20	6	3.0	<15	6.66	+370	8,500	<15	N
TP-21	6	0	<15	7.03	+310	3,500	<75	N
TP-22	6	1.0	<15	7.17	+320	10,000	<75	N
TP-23	4	2.0	<15	7.22	+320	12,000	<75	N
TP-24	4	2.0	<15	6.44	+340	27,000	<75	N
TP-25	4	0	<15	6.90	+340	13,000	<75	N
TP-26	6	-1.0	<15	7.51	+330	22,000	<75	N
TP-27	6	-13	<15	8.16	+340	5,700	<75	N
TP-28	4	1.0	<15	6.83	+340	10,000	<75	N
TP-29	4	-3.0	<15	6.96	+340	14,000	<75	N
TP-30	5	5.0	<15	7.10	+280	7,600	<75	N
TP-31	2.5	6.0	<15	6.95	+300	11,000	<75	N
TP-32	5	-11	<15	8.03	+300	3,100	<75	N
TP-33	3.5	0	<15	7.00	+300	29,000	<75	N
TP-34	7	-3.0	<15	8.29	+300	5,600	<76	N
TP-38	7	0	<15	7.05	+310	28,000	<75	N
TP-42	6	-12	<15	8.32	+300	5,100	<75	N
TP-46	6	-4.0	<15	7.91	+310	3,800	<75	N
TP-48	4	-33	<15	8.33	+300	3,200	<75	P
TP-50	6	-2.0	<15	6.93	+310	5,500	<75	N
TP-52	6	-39	18	8.03	+320	1,900	<75	N
TP-55	6	0	220	7.57	+360	510	<75 M	N
TP-80	2	2.0	19	8.21	+350	1,500	<75	N
TP-81	6	2.0	<15	6.77	+370	13,000	<75	N
TP-82	5.5	2.0	<15	6.97	+370	12,000	<75	N

6.3 ENGINEERING RECOMMENDATIONS

Our recommendations are based on the assumption that the soil conditions are similar to those disclosed by the explorations. If variations are noted during construction or if changes are made in site plan, alignment, or pipe configuration, we should be notified so we can supplement our recommendations, as applicable.

6.3.1 Geotechnical Considerations

Based on the soils encountered during our geotechnical exploration, we anticipate excavation difficulties in the form of caving sands along portions of the proposed alignment in granular soils, shallow bedrock, and in cobbles and/or boulders present along the proposed alignment. Blasting may be required in some sections of the proposed alignment in areas of shallow bedrock. Contractors should satisfy themselves as to excavation characteristics before bidding this project.

Surficial low to high plasticity clay soils are present along the proposed alignment, mainly along the northern portion of the alignment. Such soils are commonly referred to as "expansive" or "swelling" soils because they expand or swell as their moisture contents increase. However, these soils also "contract" or "shrink" as their moisture contents decrease. Clay soils excavated during construction are not suitable for reuse as trench backfill material in structural areas as defined in **Section 6.3.7.3** – Trench Backfill Materials.

Ground water was encountered at only one exploration location along the proposed alignment in Antelope Valley. It is anticipated that ground water will be present during construction of the proposed alignment at several creek crossings which were not explored. Dewatering may be necessary depending upon the final pipeline grades and the actual ground water level at the time of construction.

6.3.2 Faulting and Seismic Design Criteria

The proposed alignment is located in northwestern Nevada, which is a seismically active area within the Basin and Range area. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, and on the intensity and the magnitude of the seismic event.

In order to estimate the parameters for seismic ground motions at the subject site, Terracon utilized the USGS website, "*Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2002 Data*" (USGS, 2005) in general accordance with the 2003 *International Building Code*. According to this source, the peak accelerations at the start of the proposed alignment at the Pump Station and at the end of the alignment at the Terminal Water Storage Tank are shown in Table 20.

We recommend that the seismic criteria values associated with the proposed Terminal Water Storage Tank be utilized in design. These values are more conservative than the lower values associated with the start of the alignment at the proposed Pump Station. Recent work by dePolo (2006) and others may lead to revisions in the U.S. Geological Survey (USGS) Seismic Hazard Maps on which these data and the 2003 *IBC* Codes are based. Most likely the seismic design criteria values would be increased, resulting in increased seismic hazard.

Table 20 – Site Seismic Criteria

Ground Motion	**10% Possibility of Exceedence in 50 Years (%g)**		**2% Possibility of Exceedence in 50 Years (%g)**	
	Pump Station	***Terminal Tank***	***Pump Station***	***Terminal Tank***
Peak Ground Acceleration (PGA)	25.48	29.01	48.05	54.44
0.2 sec Spectral Acceleration (SA)	62.03	70.33	117.59	132.40
1.0 sec Spectral Acceleration (SA)	21.62	24.21	44.32	49.32

Based on our site exploration, we suggest a Site Class definition "D" for this site (Table 1615.1.5, 2003 *International Building Code*). This Site Class definition corresponds to a *stiff soil profile*.

Due to the potential for surface rupture on faults associated with the Warm Springs Fault Zone, automatic system shutdown procedures should be implemented and documented as a part of system operation. This system could employ an accelerometer or other sensor so that, when a seismic event exceeds a threshold amount, the system would automatically be shut down faster than human intervention. Should surface fault rupture occur, particularly on the main Warm Springs Fault, rupture of the proposed Main Transmission Line may occur, resulting in potentially severe flooding and triggering of a debris flow.

It is our understanding that an underground fiber optic or other communication cable will also parallel the proposed alignment from the Pump Station to the Terminal Water Storage Tank. Appropriate measures should be taken to ensure that this cable is not severed during a seismic event with surface fault rupture. Mitigation measures can include, but not be limited to, using large pull boxes with coiled loops of extra cable, flexible pipe cable encasement, shallow trench backfill, etc. Secondary communication systems should also be considered, such as utilizing the Sierra Pacific Power Company's (SPPCo) Alturas Power Line system and the proposed Ft. Sage Substation.

6.3.3 Liquefaction

Based on the anticipated ground motion shown above and the lack of saturated, clean sand soils encountered during exploration, it is our opinion that liquefaction potential at this site is minimal. Saturated, *Silty Sand (SM)* soils encountered in Boring TP-10 (B-10) are generally medium dense to very dense with generally low liquefaction potential.

Should ground water levels rise in the vicinity of the proposed alignment in the future, due to climatic or other changes, liquefaction potential may be an issue due to the presence of clean sand soils and should be re-addressed at that time.

6.3.4 Pipe Design Parameters

The following soil design parameters for the Main Transmission Line alignment are presented in Table and may be used where applicable for the design of the proposed alignment. Design parameters have been determined for the according to the soil properties encountered along the alignment. Sections with similar soil properties have been grouped together and assigned pipe design parameters. Pipe design parameters have been determined assuming an embedment depth of 6 feet.

Table 21 – Soil Design Parameters for the Main Transmission Line Alignment

Alignment Section	Coefficient of Friction (1)	Modulus of Soil Reaction E (1,2) (psi)	General USCS Soil Classification at a Depth of 6 Feet	Unit Weight of Compacted Fill(1) (pcf)
TP-01 to TP-14	0.35	700	SM, SP-SM, SP	125
TP-15 to TP-16	0.35	1000	GP-GM	125
TP-17 to TP-30	0.35	700	SM, SP-SM	125
TP-31	0.20	200	SC	125
TP-32 to TP-71	0.35	700	SM	125
TP-72 to TP-73	0.20	100	SC, CH	125
TP-74 to TP-82	0.35	700	SM	125

1Assumes AWWA Pipe Bedding Material.
2Bureau of Reclamation Value of E' for Iowa Formula. [Assumes a minimum of 80 percent relative compaction (ASTM D1557).]

The modulus of soil reaction values were determined using the method of Howard (1977), included in **Appendix D**.

6.3.5 Thrust Blocks

General thrust block design parameters are shown in Table 22. Thrust blocks should bear on granular material whenever possible. Clay soils at thrust block bearing depth are not anticipated along the proposed alignment.

Lateral loads may be resisted by soil friction and by the passive resistance of the soil. A coefficient of friction of 0.35 may be used between thrust blocks and the supporting granular, native, trench backfill, or structural fill soils.

Table 22 – Thrust Block Design Parameters

Parameter	Granular Soils
Allowable Maximum Bearing Capacity	2,500 psf
Allowable Coefficient of Friction	0.35
Ultimate Passive Earth Pressure	300 pcf
Ultimate Active Earth Pressure	33 pcf
Ultimate At-Rest Pressure	50 pcf

The passive, active, and at-rest earth pressures assume that the backfill is properly compacted. Backfill should be mechanically compacted in layers (8-inch maximum thickness) to a minimum of 95 percent relative compaction (ASTM D1557); flooding should not be permitted. Care should be taken when placing backfill so as not to damage the footings.

Thrust block locations should be carefully inspected to ensure adequate lateral bearing. If very soft soil conditions are encountered, larger thrust blocks or overexcavation with forming thrust blocks and backfilling with structural fill may be necessary. Thrust blocks should be poured on a firm and unyielding surface.

These lateral load recommendations assume structural fill against foundations is properly placed and compacted. Backfill should be mechanically compacted in layers (8-inch maximum thickness) to a minimum of 95 percent relative compaction (ASTM D1557); flooding should not be permitted. Care should be taken when placing backfill so as not to damage the footings. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors to minimize damage.

6.3.6 Permanent Slopes

Permanent slopes are not anticipated along the proposed alignment which are substantially different than the natural, existing topography. Should any permanent slopes be elected, they should be no steeper than 2.5H:1V (Horizontal:Vertical). At a 2.5H:1V geometry, minor sloughing of the slope surface should be expected.

Any required erosion control measures should be provided for all slopes as soon as possible after grading. Drainage from the toe of the slope should be directed around the site and away from the disturbed area. It should be noted that native, surficial soils along the proposed alignment are moderately erosive. Rip-rap or other erosion control measures will likely be necessary at existing drainage channel and creek crossings.

6.3.7 Earthwork

6.3.7.1 Site Clearing

Existing topsoil, vegetation, and other deleterious materials are to be removed from along the proposed alignment minimum depth of 6 inches. All materials derived from the site stripping operations should be stockpiled separately from trench backfill materials for later use in revegetation efforts. Grading work should not be performed on or with frozen soils.

6.3.7.2 Excavation

Groundwater was encountered during our geotechnical exploration along the proposed alignment in Antelope Valley at approximately 20 feet below the existing grade. Groundwater may pose limited constraints to installation of the proposed Main Transmission Line pipe. Required excavations below the groundwater table can expect caving and will likely require dewatering and trench shoring.

Based on the soils encountered during the exploration, we anticipate excavation difficulties in the form of caving sands in granular soils, cobbles and/or boulders, and shallow bedrock present along various portions of the proposed alignment. Blasting may be required in some sections of shallow bedrock along the proposed alignment. Contractors should satisfy themselves as to excavation characteristics before bidding the project.

Excavations or trenches should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions. All excavations or trenches should comply with all applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that Terracon is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

6.3.7.3 Trench Backfill Materials

Native granular soils excavated along the proposed alignment should be considered for reuse as backfill material (as described below) above the top of pipe bedding. Structural areas are considered areas with improvements, such as the existing Fish Springs Road and other unnamed access roads, or the proposed parking/access roads associated with the Well Houses. Non-structural areas are considered other areas without improvements, such as generally along the proposed alignment.

Non-Structural Backfill

In non-structural areas, excavated material can be generally used as trench backfill provided any coarse material greater than 4 inches in largest dimension is removed prior to placement. To minimize settlement of trench backfill and subsequent ponding of water, trench backfill in non-structural areas should be placed in maximum 12-inch loose lift thicknesses and compacted to a minimum of 85 percent relative compaction (ASTM D1557).

Structural Backfill

Should structural improvements be anticipated in areas of the proposed alignment, such areas should be backfilled with approved structural fill. Native clay or silt soils will not be suitable for reuse as structural fill or within 3 feet of structural sections. All imported structural fill materials should comply with the guidelines presented in Table 23, Guidelines for Imported Structural Fill.

Table 23 – Guidelines for Imported Structural Fill (ASTM C136)	
Sieve Size	Percent Passing
4-Inch	100
¾-Inch	70 - 100
No. 40	15 - 70
No. 200	5 - 30
Maximum Allowable Liquid Limit and Plasticity Index (ASTM D4318)	
Liquid Limit (LL)	35 max
Plasticity Index (PI)	10 max
Minimum Allowable R-Value (ASTM D2844)	
R-Value	45 min
Maximum Allowable Soluble Sulfates (EPA 300)	
Sulfates	< 0.1% by Weight SO_4

Materials that deviate from these guidelines must be approved by the geotechnical engineer prior to use.

Structural fill should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557). The moisture content of structural fill at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

6.3.7.4 Pipe Bedding Materials

Pipe bedding materials should meet the minimum specifications presented by the American Water Works Association (AWWA) and the *Standard Specifi ations for Public Works Construction*, latest edition.

7.0 SURGE AND TERMINAL WATER STORAGE TANKS

The proposed Surge Tank (200,000 gallons) site is located near the high point of the Main Transmission Line alignment between Honey Lake Valley and Dry Valley and is located entirely within Section 14, Township 25 North, Range 18 East, M.D.M. The proposed Terminal Water Storage Tank (1 million gallons) is located near the divide between Antelope Valley and Lemmon Valley at the current terminus of the project. It is anticipated that the Surge Tank and Terminal Water Storage Tank will be of welded steel construction and be supported by ring and central footings and/or a compacted gravel foundation. Compacted aggregate base parking and/or access areas will be constructed around the tanks. A grading plan was not available at the time of this draft report; however, we anticipate cuts and fills up of up 8 feet will be required to achieve pad grade at the two proposed tank sites.

7.1 SITE EXPLORATION PROCEDURES

7.1.1 Field Exploration

The proposed Surge Tank and Terminal Water Storage Tank sites were explored on May 1, 2006 by excavating six test pits at the approximate locations indicated on Plates 4 and 5 presented in **Appendix A**. Terracon established the test pit locations in the field by taping from the available reference features and by the use of a hand-held WAAS enabled GPS unit. The test pit locations should be considered accurate only to the degree implied by the methods used to define them. The test pits were excavated with a Cat 314C trackhoe. During test pitting, bulk samples were collected of each major geotechnical unit.

An engineering geologist classified the site soils in the field in general accordance with ASTM D2488. Samples were then returned to our Sparks, Nevada laboratory, and classifications were confirmed or revised based on laboratory testing of selected samples. The descriptions of the soils indicated on the test pit logs are in accordance with the enclosed General Notes and the Unified Soil Classification System (USCS). Classification in this manner provides indications of soil properties and can be correlated to other properties using published charts (Bowles, 1995). Estimated group symbols according to the USCS are given on the test pit logs and a brief description of USCS classification system is shown in **Appendix C**.

7.2 SITE AND SUBSURFACE CONDITIONS

7.2.1 Site Conditions

The proposed Surge Tank site is located near the high point of the Main Transmission Line alignment between Honey Lake Valley to the north and Dry Valley in the south in Washoe

County, Nevada. The proposed Terminal Water Storage Tank is located between Antelope Valley to the north and Lemmon Valley to the south in Washoe County, Nevada. At the time of exploration, the two sites were undeveloped and contained low pine trees, brush, and grasses. The Surge Tank is accessed from an unimproved road (Tuscarora Gas Line access road) and the Terminal Water Storage Tank is accessed from Matterhorn Boulevard.

7.2.2 General Geologic and Soil/Bedrock Conditions

The proposed Surge Tank site lies between the Fort Sage Mountains to the west and the Virginia Range to the east and within the Basin and Range geomorphic province. The Basin and Range geomorphic province is characterized by horizontal extension between the Sierra Nevada Range to the west and the Wasatch Front to the east and is bounded by the Colorado Plateau to the southeast and the Columbia Plateau to the north. The site of the proposed Surge Tank is generally characterized by Tertiary volcanics consisting of vitric tuffs, welded tuffs, and tuff breccias.

The Nevada Bureau of Mines and Geology (NBMG) has mapped the proposed site of the Surge Tank as Tertiary crystal vitric rhyolite tuff and welded tuff. The Tertiary crystal vitric rhyolite tuff was described as *white, gray, pink, tan; fine to coarse vitroclase matrix with broken crystals of sanidine, sodic oligoclase to sodic andesine, less quartz, and minor biotite; some bedded ash-fall sections with pumice lapilli and minor dark lithic felsite fragments; mostly nonbedded and nonwelded, uniform, and massive ash-flow deposit.* The welded tuff was described as *moderately to strongly welded portions of the crystal vitric rhyolite tuff* (Grose, 1984).

The proposed Terminal Water Storage Tank site lies to the north of Lemmon Valley on the southern flank of Fred's Mountain, to the west of the Pah Rah Range, and within the Basin and Range geomorphic province. The Basin and Range geomorphic province is characterized by horizontal extension between the Sierra Nevada Range to the west and the Wasatch Front to the east and is bounded by the Colorado Plateau to the southeast and the Columbia Plateau to the north. The site of the Terminal Water Storage Tank is generally characterized by Cretaceous intrusive rocks consisting of gabbro to granite (Bonham, 1969).

The Nevada Bureau of Mines and Geology (NBMG) has mapped the proposed site of the Terminal Tank as quartz monzonite. The quartz monzonite is described as *pink to pale-gray, massive, medium to coarse-grained, equigranular to porphyritic quartz monzonite to granite. Includes extensive aplite, graphic granite, quartz veins, and pegmatite dikes. Generally deeply weathered; forms low, rounded outcrops* (Cordy, 1985).

7.2.3 Faulting and Seismicity

Nevada ranks as the fourth highest state of earthquake occurrence of Magnitude 3.5 and greater, behind Alaska, California, and Hawaii (USGS, 2005). In addition, the Warm Springs Fault Zone, generally consisting of a main fault trace and a number of secondary faults, or splays, is one of the top 5 most seismically active regions of Nevada (dePolo, 2006). Additionally, dePolo states that the Warm Springs Fault Zone has had 6 seismic events prior to 10,000 years ago and 2 seismic events within the last 10,000 years (Holocene). He states that a large event can be expected approximately every 8,000 years and a small event approximately every 2,000 years. In addition, the main trace of the Warm Springs Fault Zone can be expected to move 6 to 8 feet during an event, and current geodetic information suggests that the region is overdue for an event. The last major event on the Warm Springs Fault Zone occurred 9,000 years ago.

dePolo et al., in a study of earthquake occurrence in the Reno-Carson City urban corridor, state that *overall, the probabilities of potentially damaging earthquakes within the region are relatively high and are commensurate with many parts of California, a state with a well-recognized high earthquake hazard* (dePolo et al., 1997). These probabilities are also valid for the area encompassed by this project. As a result of seismicity present, the project structures should be designed accordingly.

Guidelines for determining and assessing faults have been developed by the Nevada Earthquake Safety Council (NESC, 1998); however, neither the State of Nevada nor various counties have adopted the guidelines. These NESC guidelines are generally consistent with those adopted in the State of California Alquist-Priolo Act of 1972 that defines active faults as those faults with displacement within the past 11,000 years (Holocene) and potentially active faults as those faults with displacement within the past 11,000 to 2,000,000 years. Based on the geologic mapping, faults in the vicinity of the project site are considered active.

The proposed Surge Tank site is located in the Warm Springs Fault zone and according to the NBMG published and available geologic maps, 14 faults (Quaternary or older) are mapped as within 1 mile of the site (Grose, 1984). Of these, dePolo has mapped 6 faults as being Late Quaternary (active) in age (2006).

The published and available geologic map indicate that 5 faults (Quaternary or older) are mapped as within 1 mile of the proposed Terminal Water Storage Tank site (Cordy, 1985). The Terminal Tank is located between Antelope Valley and Lemmon Valley. Antelope Valley is a linear geomorphic feature generally bounded on the west by rangefront faulting. Cordy (1985) describes these five faults as *indeterminate age; predominantly bedrock faults of probable pre-Pleistocene age and bedrock-alluvial faults of probable pre-Pleistocene to* *possible late Pleistocene age. Recent movements are not precluded on faults in this category.* These faults are most likely considered potentially active.

7.2.4 Flooding

The Federal Emergency Management Agency (FEMA) has mapped the proposed Surge Tank and Terminal Water Storage Tank sites as of 1994 as lying within shaded Zone X, and is defined as *areas of the 500-year flood or areas of the 100-year flood with average depths of less than 1 foot* (FEMA, 1994). As a result, the project should be designed accordingly.

7.2.5 Subsurface Conditions

As presented on the test pit logs, the subsurface soils at the proposed Surge Tank site generally consist of dense to very dense *Silty Sand with Gravel (SM)* and *Poorly Graded Sand with Silt and Gravel (SP-SM)* from the existing ground surface to 5 feet below the surface and generally correspond with that mapped by the NBMG. This material is underlain by bedrock. Excavator refusal was experienced in Test Pit STTP-01 and STTP-02 due to shallow bedrock.

As presented on the test pit logs, the subsurface soils at the proposed Terminal Water Storage Tank site generally consist of medium dense to dense *Silty Sand (SM)*, *Poorly Graded Gravel with Sand (GP)*, and *Poorly Graded Sand with Gravel (SP)* from the existing ground surface to approximately 3.5 to 9 feet below the surface and generally correspond with that mapped by the NBMG. This material is underlain by granite bedrock. Excavator refusal was experienced in Test Pits TTTP-02 to TTTP-04 due to shallow bedrock.

Conditions encountered at each test pit location are indicated on the individual test pit logs. It should be noted that inconsistencies could exist away from, or between, test pit locations. Stratification boundaries on the test pit logs represent the approximate location of changes in soil types; in-situ, the transition between native materials at this site is typically gradual.

7.2.6 Groundwater Conditions

Groundwater was not encountered in any of our test pits at the proposed Surge Tank and Terminal Water Storage Tank locations. As a result, we do not anticipate that groundwater will pose any constraints to site development.

7.2.7 Laboratory Testing

Soils testing performed by Terracon were conducted in general accordance with the American Society for Testing and Materials (ASTM), standards where applicable. Representative samples of major material types were tested to determine their grain size distribution (ASTM D422), plasticity index or Atterberg Limits (ASTM D4318), and moisture content (ASTM D2216). These test results are used to properly classify the soils and to adjust the field logs if necessary. Such testing is typically referred to as index testing. These index tests can be correlated to empirical studies to obtain representative values for other engineering properties. Based on index testing, we can estimate design parameters and assign additional testing, as necessary.

7.3 ENGINEERING RECOMMENDATIONS

Our recommendations are based on the assumption that the soil conditions are similar to those disclosed by the explorations. If variations are noted during construction or if changes are made in site plan, structural loading, or foundation type, we should be notified so we can supplement our recommendations, as applicable.

7.3.1 Geotechnical Considerations

The site appears suitable for the proposed construction based on geotechnical conditions encountered in the exploratory test pits. Shallow bedrock was encountered between 4 and 6 feet below the existing ground surface at the proposed Surge Tank site and between 3 and 6.5 feet below the existing ground surface at the proposed Terminal Tank site.

Excavation at the proposed Surge and Terminal Water Storage Tank sites will most likely be difficult, due to the hard and slightly to moderately weathered nature of the bedrock. The bedrock present at the two tank sites will be adequate for structural support of the tanks and could also be reused as structural fill, provided oversize particles are removed.

If structures or other utilities are located in the vicinity of any of the test pits, the test pit backfill should be removed and recompacted in accordance with **Section 7.3.8.3** - Overexcavation. Failure to properly compact backfill could result in excessive settlement of improvements located over test pits.

Recommendations regarding the design and construction of foundations, and the support of floor slabs and pavements, relative to the subsurface conditions encountered in the borings, are presented in the following sections of this report.

7.3.2 Seismic Design Criteria

The Surge Tank and Terminal Water Storage Tank sites are located in northwestern Nevada, which is a seismically active area within the Basin and Range area. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, and on the intensity and the magnitude of the seismic event.

In order to estimate the parameters for seismic ground motions at the subject site, Terracon utilized the USGS website, "*Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude and Longitude, 2002 Data*" (USGS, 2005) in general accordance with the 2003 *International Building Code*. According to this source, the peak accelerations at the proposed Surge and Terminal Water Storage Tank sites are as follows:

Table 24 – Site Seismic Criteria

Ground Motion	10% Possibility of Exceedence in 50 Years (%g)		2% Possibility of Exceedence in 50 Years (%g)	
	Surge Tank	Terminal Tank	Surge Tank	Terminal Tank
Peak Ground Acceleration (PGA)	26.14	29.01	49.20	54.44
0.2 sec Spectral Acceleration (SA)	63.71	70.33	120.28	132.40
1.0 sec Spectral Acceleration (SA)	22.17	24.21	45.54	49.32

Based on our geotechnical exploration, we suggest a Site Class definition "C" for the proposed Surge Tank and Terminal Water Storage Tank sites (Table 1615.1.5, 2003 *International Building Code*). This Site Class definition corresponds to a *very dense soil and soft rock* profile.

7.3.3 Pump Station Water Storage Tank Design Criteria

As requested, we are providing recommended design criteria for the proposed Surge and Terminal Water Storage Tanks as developed by the American Water Works Association (AWWA). Recommended design criteria based on the AWWA D100-96 Standard for Welded Steel Tanks for Water Storage (AWWA, 1996) are shown in Table 25 below.

Table 25 – AWWA Surge and Terminal Tank Design Criteria	
Design Parameter	Value
Seismic Zone (Figure 7)	4
Zone Coefficient (Table 24)	0.40
Soil Profile Type (Table 27)	A
Site Amplification Factor (Table 27)	1.0
Poisson's Ratio	0.35
Young's Modulus (Modulus of Elasticity)	3,000 ksf

7.3.4 Liquefaction

Based on the lack of saturated, clean sand soils and the presence of shallow bedrock encountered during exploration, it is our opinion that liquefaction potential at the proposed Surge and Terminal Water Storage Tank sites is minimal.

7.3.5 Tank Foundation Systems

Conventional shallow foundations, such as spread footings, are appropriate for supporting the proposed Surge and Terminal Water Storage Tanks at their respective sites. Spread footings should be embedded a minimum of 24 inches below the nearest adjacent grade for frost protection and should bear on a minimum of 12 inches of compacted granular structural fill. Spread footings bearing on a minimum of 12 inches of properly compacted granular structural fill can be designed for a maximum net allowable soil bearing pressure of 5,000 pounds per square foot (psf) (assuming a minimum embedment depth of 24 inches within the structural fill and a minimum width of 24 inches). The net allowable soil bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation.

The magnitude of settlement that will occur beneath the foundation would depend upon the variations within the subsurface soil profile, the actual structural loading conditions, the embedment depth of the footings, the actual thickness of compacted fill, and construction quality control. Assuming that spread footing construction is performed in accordance with our recommendations, it is our opinion that total settlement will be on the order of approximately 1 inch. Differential settlement up to approximately two-thirds of the total settlement value should be anticipated.

Tank foundations should not be placed partially on bedrock and partially on structural fill. Where tank foundations will be placed partially on bedrock and partially on structural fill due to site limitations and requirements, settlement of the structural fill may be on the order of 1 percent of the maximum fill height that would result in large differential settlements of the tank foundations. This differential settlement should be avoided, if at all possible. Mitigation measures to minimize this differential settlement may include providing a gradual transition from the bedrock to the structural fill and/or overexcavating a portion of the bedrock and backfilling with structural fill.

It is recommended that the building excavation be evaluated by the geotechnical engineer during excavation, but prior to structural fill or concrete placement. If, during evaluation, unsuitable materials are encountered, the geotechnical engineer may require that the footing subgrade be modified by additional overexcavation and subsequent structural fill placement.

Properly compacted structural fill in footing areas should be free of water and loose soil prior to placing concrete. Concrete should be placed as soon as possible after excavation to minimize disturbing the bearing material. Should the material at bearing level become disturbed or saturated, the affected soil should be removed and replaced prior to placing concrete.

7.3.6 Lateral Earth Pressures

Lateral loads may be resisted by soil friction and by the passive resistance of the soil. An allowable coefficient of friction of 0.45 may be used between tank foundations and the supporting structural fill soils. The coefficient of friction has been reduced by a factor of 1.5. The ultimate passive resistance of properly compacted fill may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot (pcf). A one-third increase may be used for wind or seismic loads. The passive pressure and the frictional resistance of the soils may be combined without reduction in determining the total lateral resistance. The ultimate active resistance of properly compaction fill may be assumed to be equal to a fluid with a density of 30 pcf. Ultimate values should have an appropriate safety factor applied before use.

These lateral load recommendations assume structural fill against foundations is properly placed and compacted. Backfill should be mechanically compacted in layers (8-inch maximum thickness) to a minimum of 95 percent relative compaction (ASTM D1557); flooding should not be permitted. Care should be taken when placing backfill so as not to damage the footings. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors to minimize damage.

7.3.7 Permanent Slopes

Permanent slopes are anticipated at the proposed Surge and Terminal Water Storage Tank sites. Slopes should be no steeper than 2.5H:1V (Horizontal:Vertical). At a 2.5H:1V geometry, minor sloughing of the slope surface should be expected. Any required erosion control measures should be provided for all slopes as soon as possible after grading.

Drainage from the toe of the slope should be directed around the site and away from the tank. Water should not be allowed to flow over the top of the constructed slopes.

7.3.8 Earthwork

7.3.8.1 Site Clearing

Existing topsoil and other deleterious materials are to be removed from the site to a minimum depth of 8 inches in parking areas and non-structural areas and to a minimum depth of 12 inches within the building footprint and extending at least 3 feet in all directions from the tank footprints. All materials derived from the site stripping operations should be removed from the site and not used in any fills or backfills. Grading work should not be performed on or with frozen soils.

7.3.8.2 Excavation

Groundwater was not encountered during our exploration and is not expected to pose construction constraints.

Based on the soils encountered during the exploration, we anticipate excavation difficulties for tank pad construction in the form of shallow bedrock below approximately 4 to 6 feet below the existing ground surface at the proposed Surge Tank site and 3 to 6.5 feet below the existing ground surface at the proposed Terminal Water Storage Tank site. Contractors should satisfy themselves as to excavation characteristics before bidding the project.

Excavations or trenches should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions. All excavations or trenches should comply with all applicable local, state and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that Terracon is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

7.3.8.3 Overexcavation

The excavated Surge and Terminal Water Storage Tank pads should be directly overlain by at least 6 inches of Type 2, Class B aggregate base or drain rock in accordance with AWWA guidelines. Depending on the characteristics of the excavated bedrock surface, a thicker section of aggregate base may be required as a leveling course. It is anticipated that cuts and fills on the order of 8 feet in thickness will be required to bring the proposed tank sites to finished grade. Additionally, it is recommended that foundations be embedded a minimum of 24 inches for frost protection and that foundations bear on a minimum of 12 inches of structural fill. Therefore, pad and/or footing overexcavations can be anticipated to achieve the required aggregate base beneath the proposed tanks and in areas to minimize differential settlements as discussed in **Section 7.3.5** – Tank Foundation Systems.

Overexcavations should be cleaned of loose or deleterious material and backfilled with compacted granular structural fill. Structural fill material should be placed in maximum 8-inch-thick loose lifts and compacted to 95 percent relative compaction (ASTM D1557). The overexcavation and backfill procedure for footings is shown in Figure 5 below. The guidelines for imported structural fill are presented in Table 26 – Guidelines for Imported Structural Fill presented below in **Section 7.3.8.4** - Fill Materials. It should be the contractor's responsibility to ensure that the bottom of the excavation is prepared in such a manner as to allow the specified minimum compaction of the structural fill to be achieved.

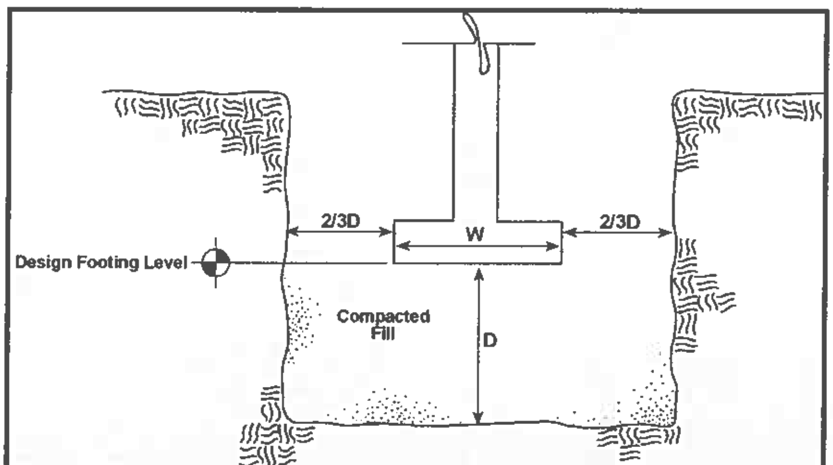

Figure 5 – Footing Overexcavation and Backfill

7.3.8.4 Fill Materials

Native granular soils may be utilized as structural fill and utility backfill provided they are kept at a minimum depth of 12-inches below the bottom of footings. All imported structural fill materials should comply with the guidelines presented in Table 28, Guidelines for Imported Structural Fill.

Table 26 – Guidelines for Imported Structural Fill (ASTM C136)	
Sieve Size	Percent Passing
4-Inch	100
¾-Inch	70 - 100
No. 40	15 - 70
No. 200	5 - 30
Maximum Allowable Liquid Limit and Plasticity Index (ASTM D4318)	
Liquid Limit (LL)	35 max
Plasticity Index (PI)	10 max
Minimum Allowable R-Value (ASTM D2844)	
R-Value	45 min
Maximum Allowable Soluble Sulfates (EPA 300)	
Sulfates	< 0.1% by Weight SO_4

Materials that deviate from these guidelines must be approved by the geotechnical engineer prior to use.

7.3.8.5 Fill Placement and Compaction

Areas to receive fill should be scarified to a minimum depth of 6-inches, moisture conditioned, and compacted to a minimum of 90 percent relative compaction (ASTM D1557). Areas that are unstable or pumping should be stabilized to a firm and non-yielding condition prior to receiving fill. Native clay soils, if present, may be too wet to compact upon excavation. Moisture conditioning may be possible by scarifying the top 12 inches of subgrade and allowing it to air dry to near the optimum moisture prior to compaction. Where moisture conditioning by air drying is ineffective, mechanical stabilization may be necessary. Mechanical stabilization may be achieved by overexcavation and/or placement of an angular and well-graded 12- to 18-inch thick lift of 12-inch-minus to 3-inch-plus rock fill or Class C or D drainrock. This fill should be compacted with large equipment until no further deflection is noted. Where more than 30 percent is retained on the ¾-inch sieve, standard density testing is not valid. As will likely occur here, standard density testing is not possible and a proof rolling program will be required to obtain an unyielding surface.

All structural fill materials should comply with the requirements presented in Table 28 – Guidelines for Imported Structural Fill. Structural fill should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 95 percent relative compaction (ASTM D1557). The moisture content of structural fill at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

7.3.8.6 Trench Backfill

Native granular soils may be utilized as intermediate utility line backfill provided that cobbles and boulders greater than 4 inches in diameter are screened and removed. It is anticipated that native cobbles and/or boulders will be too coarse for use as trench backfill and should be screened and removed.

Native materials used as trench backfill should be placed in loose lifts not exceeding 8 inches in thickness and thoroughly compacted to a minimum of 90 percent relative compaction (ASTM D1557). The moisture content of native soils at the time of compaction should be at –1 to +3 percent of the soil's optimum moisture content as determined by the modified Proctor test (ASTM D1557).

8.0 ADDITIONAL DESIGN AND CONSTRUCTION CONSIDERATIONS

8.1 Drainage and Moisture Protection

While the soils encountered during our geotechnical exploration program do not appear to be highly expansive, the importance of minimizing infiltration into the foundation bearing materials cannot be overstated. Adequate drainage should be provided at the site to minimize any increase in moisture content of the slopes or foundation bearing materials. All pavement or parking areas should be sloped away from the building to prevent ponding of water near structures.

Positive drainage away from structures should be provided during construction and maintained throughout the life of the structures. Any downspouts, roof drains, or scuppers should discharge into storm drains with sealed joint pipes, or at a minimum, splash blocks or extensions away from the structure. Proper drainage is essential for the long-term performance of foundations and other constructed structures. Backfill against footings, exterior walls, and in utility trenches should be properly compacted and free of all construction debris to reduce the possibility of moisture infiltration. If practical, surface features that could retain water should not be adjacent to the structures or pavements.

9.0 GENERAL COMMENTS

Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon also should be retained to provide testing and observation during excavation, grading, foundation and construction phases of the project to ensure conformance to the design recommendations presented herein.

The analysis and recommendations presented in this report are based on the data obtained from the borings and test pits performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings and test pits, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, and bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

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