GEOTECHNICAL INVESTIGATION

50,000-Gallon Water Tank
Well 4 Site
Bishop, California

PREPARED FOR:
RESOURCE CONCEPTS, INC.
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GEOCON

GEOCON PROJECT NO. R8802-06-01

MARCH 2012

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. R8802-06-01
March 14, 2012

Mr. Tim Russell
Resource Concepts, Inc.
340 N. Minnesota Street
Carson City, NV 89703

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
50,000-GALLON WATER TANK
BISHOP, CALIFORNIA

Dear Mr. Russell:

We are pleased to submit the results of our geotechnical engineering investigation for a 50,000 gallon water tank project. The accompanying report presents the findings and conclusions from our geotechnical investigation. In our opinion, the primary geotechnical consideration in the design and construction of the proposed water tank is the presence of oversized materials consisting of cobbles and possibly small boulders. This constraint will not preclude the proposed construction provided the recommendations of this report are incorporated into the design of the portion of the project addressed herein.

We would like to thank you for selecting Geocon Consultants as your geotechnical consultant on this project. If you have any questions or comments regarding any of the information contained within this report, please do not hesitate to contact our office at (775) 888-9900, or any of the undersigned at any time for assistance.

Sincerely,

Geocon Consultants, Inc.

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1.0 PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for a 50,000-gallon water storage tank to be constructed at the City of Bishop Well 4 Site, Inyo County, California. Two possible locations for the tank are being considered by the designer, Resource Concepts, and therefore our preliminary investigation addresses both sites. The two proposed tank sites are located approximately 70 feet apart. The location of the Well 4 site and proposed 50,000 gallon tank locations are illustrated on the project Vicinity Map, Figure1.

Well 4 is the primary production well for the City of Bishop and is situated approximately one-half mile west of and upslope of an existing one million gallon, welded steel water storage tank. The well is connected to the tank via a 12-inch diameter ductile iron pipe. The proposed 50,000-gallon tank will provide additional reserve capacity for the City of Bishop water system during peak consumer usage periods in order to maintain adequate system pressure during high demand days. Future expansion of the overall water system for the city may include construction of an additional 12-inch diameter pipe from the Well 4 site to the 1 million gallon storage tank and installation of another production well at the Well 4 site.

In preparation for our preliminary geotechnical investigation and report preparation we reviewed the following documents:

- Project drawings including a preliminary site map of two proposed water tank locations. The drawings were provided to our office by Resource Concepts, Inc.
- *City of Bishop Water Master Plan, Prepared for the City of Bishop Department of Public Works*, by Nolte Associates, Inc., February 2008.
- Review of regional geologic, soils and fault maps pertinent to the project vicinity.

The purpose of our preliminary geotechnical investigation was to observe the prevailing soil conditions in the project area, and based on conditions encountered; provide site-specific recommendations relative to the geotechnical aspects of designing and constructing the water tank and related improvements.

To aid in preparing this report, we performed the following scope of services:

- Reviewed the documents cited above.
- Reviewed the preliminary project plans prepared by Resource Concepts, Inc. to determine the locations of our exploratory excavations and Refraction Micro Tremor (ReMi) geophysical soundings.

- Performed two ReMi soundings (ReMi 1 and ReMi 2) in the vicinity of the proposed water tank locations.
- Observed the excavation of two exploratory backhoe test pits (TP-1 and TP-2) to a depth of 3.5 feet below the existing grade. Upon completion, the test pits were backfilled with excavated soil.
- Logged the test pits in accordance with the Unified Soil Classification System (USCS).
- Obtained representative samples from the soil test pits. The selected soil samples were transported to the Geocon Consultants geotechnical laboratory for testing and additional analysis.
- Performed laboratory tests on selected samples to evaluate pertinent geotechnical parameters.
- Performed analysis of the ReMi data to evaluate the appropriate Site Class as defined by the International Building Code (IBC).
- Prepared this report summarizing our findings, conclusions, and recommendations regarding the geotechnical aspects of constructing the project as presently proposed.

Details of our field exploration program including test pit logs are presented in Appendix A. Approximate locations of the exploratory test pits and ReMi lines are shown on Figure 2, the Site Plan. Details of our laboratory-testing program, including test results, are summarized in Appendix B. Our report is intended for the sole and exclusive use of the client, their designated representatives and agents.

2.0 SITE AND PROJECT DESCRIPTION

The project is located entirely within the existing City of Bishop Well 4 site located on West Line Street in Bishop, California. Existing onsite features include an inactive, concrete-lined open reservoir, the Well 4 pump house, a water disinfection system (chlorinator) for water pumped from Well 4, and underground utilities including a 12-inch diameter ductile iron pipe and an underground power line.

The proposed 50,000-gallon water tank will be of steel construction, 12 to 16 feet high and 25-feet in diameter with concrete floor placed on a compacted fill pad. The water tank will be located on one of two sites that are both within the area west of the Well 4 Site, adjacent to the pump house and south of the inactive water reservoir (full at the time of our investigation but not in use as a potable water source). The water tank will supply the existing 12-inch water line and provide backup capacity for the overall City of Bishop water system.

The site is a gently sloping, graded lot with elevations ranging from approximately 4,370 to 4,360 feet above mean sea level. The site slopes gently to the east toward the Owens River.

3.0 SOIL, GEOLOGY AND GROUNDWATER CONDITIONS

3.1 General

The soil conditions observed during our investigation were generally consistent with the published Natural Resource Conservation Service (NRCS) data and with geologic map descriptions. The following soil descriptions include the USCS symbol where applicable. Please refer to test pit logs and ReMi sounding logs included in Appendix A for vertical extents and relative density of the materials encountered at each location.

3.2 Soil Conditions

A map depicting the soil units in the site vicinity is presented as Figure 3. According to the *Soil Map for Benton-Owens Valley Area Parts of Inyo and Mono Counties, California* (US Department of Agriculture: January 2008), the proposed water tank will be constructed on one soil unit: Muranch family, 0 to 2 percent slopes (Soil Unit 271). According to the referenced report, this soil family is located on alluvial fan terraces with a parent material of alluvium derived from mixed sources. The typical soil profile is described as sandy loam (SC) to a depth approximately 10 inches, underlain by cobbly sandy loam to very cobbly loam (SC and GC) to a depth of 40 inches. Very cobbly coarse sand (SP to GP) is present from approximately 40 to 60 inches. The soil unit is further described as well drained.

Our site exploration consisted of two test pits and two Refraction Micro Tremor (ReMi) lines at the locations shown on the Site Plan, Figure 2. The test pits were excavated with a rubber-tire backhoe to depths of approximately 3.5 feet below the existing surface. Soils logged were generally consistent with the Soil Map being alluvial soils but without any clayey sand being encountered. The soils observed in our test pits consisted of a few inches to one foot of sand with gravel fill over approximately one foot of silty sand (SM) underlain by sandy poorly graded cobble gravel (GP) with occasional small boulders to the depth explored (3.5 feet). The soils ranged from medium dense to dense and excavation for the backhoe increased in difficulty with depth.

ReMi geophysical testing method uses refraction microtremor (low frequency) recordings from standard geophysical refraction equipment to estimate up to 100 feet of depth average shear wave velocities. The advantage of the ReMi geophysical method is that the system records ambient seismic waves which are unaffected by dynamic vibrations (noise such as traffic and construction equipment) in the area of the instrument. Where dynamic vibrations are located near conventional refraction seismic recorders they hinder the data acquisition and interpretation

The ReMi sounding profiles, test pit logs, and Site Photographs are included in Appendix A.

3.3 Geologic Conditions

The project site is situated within the northwestern portion of Owens Valley. The Owens Valley is a north-south trending structural basin bounded by the Sierra Nevada to the west, the White-Inyo Mountains to the east, and volcanic table lands associated with the Long Valley Caldera to the north. This fault-bounded basin has a local topographic relief of more than 9,000 feet between the valley floor and the crests of the nearby mountain ranges. Geologic mapping of the site is published on the United States Geologic Survey (USGS) *Geologic Map of the Bishop 15-Minute Quadrangle, California*, (1964). According the USGS map, the site is entirely situated on Holocene age (less than 12,000 years) alluvial fan deposits. A portion of this map is reproduced in Figure 4, the Site Geology Map.

3.4 Groundwater and Surface Water Courses

The proposed project is located within a portion of Owens Valley with shallow groundwater. The North Fork of Bishop Creek is within 150 feet of the proposed tank locations to the west. The creek flows to the east from near the crest of the Sierra Nevada Mountain Range into Bishop and joins the Owens River approximately 0.5 miles northeast of the project site. The creek is a significant influence of shallow groundwater elevations within the project area.

According to City of Bishop Public Works employees (verbal communication, January 30, 2012), groundwater was encountered at a depth of approximately 7.5 feet during the installation of a power pole adjacent to the Well 4 pump house during the fall of 2011. Additionally, a well driller's log for Well 4, provided by Resource Concepts, Inc, indicates that groundwater was encountered at a depth of approximately two feet below the ground surface during construction of the well in 1969. Groundwater depths may vary due to fluctuations in rainfall, spring runoff volumes, temperature, and other local factors.

3.5 Laboratory Testing

The results of laboratory test results on selected samples are presented in Appendix B. Moisture content, dry density and fines content (% passing #200 sieve) data are also presented on the logs.

3.6 Corrosion Potential

Based on our experience in this area, and published NRCS data, the soils within the project area contain low to moderate amounts of sulfates and chlorides and have a low potential for concrete attack for dense concrete especially containing fly ash or pozzolan. These same soils are moderately to highly aggressive to uncoated steel.

It is recommended that site-specific chemical testing be performed and recommendations be obtained from an experienced corrosion engineer for critical improvements that will be in direct contact with the on-site soils to determine the appropriate coating or corrosion protection method.

4.0 GEOLOGIC HAZARDS

4.1 Faulting

Owens Valley is bounded by faults which are considered capable of producing significant ground motions due to seismic events. Based on the U.S. Geological Survey, 2006, Quaternary fault and fold database for the United States (http//earthquake.usgs.gov/regional/qfaults), the nearest Holocene-aged (less than 12,000 years) faults are mapped approximately 8.75 miles east of the site (White Mountain Fault Zone, Central Section) and 12.15 miles south of the site (Owens Valley Fault Zone, 1872 Rupture Section). The White Mountain fault zone is a major Basin and Range active fault zone that is located along the western front of the White Mountains and northern Inyo Mountains. The Owens Valley Fault Zone includes an historic (1872) surface rupture that extended approximately 100 kilometers from near Bishop to south of Owens Lake near Lone Pine, California along the base of eastern front of the Sierra Nevada Mountain Range. According to information on the USGS Quaternary Fault and Fold Database webpage (http://earthquake.usgs.gov/hazards/qfaults/), various magnitudes (ranging from Mw 7.5 to 7.7) have been assigned to the 1872 earthquake with vertical displacements between 6.1 and 11 meters along the rupture section. A Fault Map showing regional faults is presented as Figure 5.

Seismic activity and significant ground-shaking are anticipated to occur during the design life of the proposed water tank. Ground shaking intensities for design considerations should be governed by seismic events occurring on the Owens Valley Fault Zone and White Mountain Fault Zone which follows the base of the Sierra Nevada Mountain Range and White Mountains, respectively.

The seismic risk along this site is not considered significantly greater than that of the surrounding areas.

4.2 Liquefaction and Related Hazards

Liquefaction of granular soils can be caused by strong vibratory motion due to earthquakes. Soils that are highly susceptible to liquefaction are loose, granular and saturated. Surface manifestations such as sand boils and ground fissures can occur in association with liquefaction. Liquefaction of soils may cause surface distress, loss of bearing capacity, and settlement of structures. Confining pressures usually preclude the occurrence of liquefaction at depths over 50 feet. With respect to pipelines, liquefaction often results in disruption of buried pipelines due to buoyant forces or lateral spreading.

Lateral spreading is a ground-failure phenomenon that can occur in association with liquefaction, whereby lateral displacements occur at the ground surface. Conditions required for lateral spreading include laterally continuous liquefiable layer(s) and gently sloping terrain. In particular, geometries with a “free-face”, such as a river or creek bank, with liquefiable layers near the toe of river bank are susceptible to lateral spreading. These conditions are not present in the vicinity of

the project site. Based on the exploration of the site, the risk of liquefaction and related lateral spreading for the project location is considered to be low.

4.3 Flood Hazards

There are no published FEMA Flood Insurance Rate Maps (FIRM) that encompasses the proposed water main alignment and Booster Pump Station site. The site is situated between the North Fork and the South Fork of Bishop Creek as shown on the Geologic Map, Figure 4. The presence of the North Fork of Bishop Creek within 150 feet of the project site suggests that there is the potential for flood related hazards.

The impact of flood hazards is primarily of concern during construction, particularly during wet seasons, and due to sheet flows that may cross the project site.

We recommend that the project Engineer determine the hydrologic conditions at the site as they relate to sheet flows and the potential for flooding from the North Fork of Bishop Creek during design level events.

4.4 Landslides and Slope Stability

The topography in the site vicinity is relatively flat-lying to gently sloping. We do not consider the potential for landsliding to be a hazard to this project.

4.5 Expansive Soil

Based on the field observations and laboratory data, we do not consider expansive soil to be a hazard to the proposed construction.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

5.1.1 Based on the results of our investigation, the proposed tank locations are suitable for the proposed 50,000-gallon tank, related utilities and associated road and landscape improvements, provided the recommendations presented herein are implemented in the design and construction of the project.

5.1.2 Our field investigation indicates the proposed project is underlain predominantly by alluvial soils. Soils are characterized by alluvial, and flood deposits, of variable age and texture. Please refer to the test pit logs and ReMi sounding profiles presented in Appendix A.

5.1.3 Groundwater was not encountered in our test pits but is inferred to be less than 10 feet below the surface based on review of the well driller's log for Well 4, the adjacent North

Fork of Bishop Creek, and verbal communication with City of Bishop Department of Public Works employees (January 2012).

5.1.4 FEMA flood hazard mapping is not available for the project site. Based on the proximity of the North Fork of Bishop Creek, we recommend that a hydrologic assessment be conducted by the project Civil Engineer to evaluate flood hazard potential.

5.1.5 Low to moderate concentrations of sulfates and chlorides are present in the project vicinity and therefore Type I or Type II cement will be suitable for construction. However, fly ash mixes are recommended to reduce reaction potential as is customary in mixes in use in the Bishop area. Recommendations from an experienced corrosion engineer should be obtained for critical improvements consisting of ferrous metals that will be in direct contact with the on-site soil.

5.2 Seismic Design Criteria

The site is located near faults capable of generating strong seismic shaking during the life of the project. Site-specific IBC geotechnical seismic design parameters were obtained utilizing the Java Ground Motion Parameter Calculator – Version 5.0.8 available on the USGS Earthquake Hazards Program website. With a site's latitude, longitude, and soil class, the calculation of spectral response accelerations may be accomplished with better precision than that with large-scale maps provided in the 2010 CBC or ASCE 7-05. The values correspond to the maximum considered earthquake ground motion having a 2% probability of exceedance in a 50 year period (with 5% critical damping). The CBC Site Class was determined to be "B" based on the ReMi data acquired on site. The ReMi data is presented in Appendix A. Seismic information for 2010 CBC design is provided in the following table.

TABLE 5.2
2010 CBC SEISMIC DESIGN PARAMETERS

Parameter	Factors	IBC Reference
Site Class	B	Table 1613.5.2
Spectral Acceleration	S_s = 1.389 S_1= 0.502	Figure 1613.5(3) Figure 1613.5(4)
Seismic Coefficient, F_a	F_a = 1.000	Table 1613.5.3(1)
Seismic Coefficient, F_v	F_v = 1.000	Table 1613.5.3(2)
Adjusted Spectral Response S_{MS}, S_{Ml}	S_{MS} = 1.390 S_{Ml} = 0.502	Equation 16-37 Equation 16-38
Design Spectral Acceleration S_{DS}, S_{D1}	S_{DS} = 0.926 S_{D1} = 0.334	Equation 16-39 Equation 16-40

5.3 Grading – General, Site Preparation

5.3.1 Earthwork and backfill operations should be observed and compacted fill tested by a Geocon Consultants representative.

5.3.2 All references to relative compaction and optimum moisture content in this report are based on the latest edition of ASTM D1557 Test Method, entitled *Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort*.

5.3.3 A preconstruction conference should be held at the site prior to the beginning of trenching operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Soil handling and grading requirements can be discussed at that time. In addition, the depth to groundwater should be measured and any special mitigation requirements and procedures discussed and agreed upon.

5.3.4 Site preparation should begin with the removal of vegetation, asphalt, and debris (if any). Material generated during stripping is not suitable for use as backfill and should be disposed of in an area designated by the Engineer or disposed of offsite. The location of on-site disposal areas should be identified on the drawings or in the specifications.

5.3.5 Native soils are not anticipated to be suitable for trench bedding. Importation of bedding should be planned for.

5.3.6 Most native soils will meet the requirements for trench backfill after removal of any oversize materials (>4 inch diameter).

5.3.7 Wet soils may deflect or pump under heavy equipment loads. Yielding soil conditions in access areas can typically be stabilized using one of the methods listed below. However, soil conditions and mitigation methods should be reviewed and approved by Geocon Consultants when encountered.

- Wet soils may be stabilized by simply allowing the soil to dry, or by replacing wet soils with approved imported backfill.
- Overexcavation of from one to three feet and the placement of reinforcing fabrics over the subgrade followed by the placement of 8 to 18 inch angular cobble rock fill. A filter fabric would then be placed over the stabilization layer of cobbles prior to the placement of a leveling course or fill as necessary. The depth of overexcavation and cobble rock fill sizes should be reviewed by Geocon for the specific application.
- Other alternative stabilization methods, as proposed by the contractor, should be reviewed and approved by the Geocon prior to implementation.

5.4 Soil Handling, Excavation Characteristics, Bedding and Backfill Criteria

5.4.1 In our opinion, excavation will require conventional grading/excavation equipment suitable for excavation of loose to dense, sand and sandy gravels with cobbles and possibly small boulders.

5.4.2 Prior to importation of bedding, aggregate base or other engineered earth materials the contractor should submit to the Geotechnical Engineer laboratory test results indicating conformance of the proposed import material to a specific application or backfill specification.

5.4.3 Temporary excavations, such as utility trench sidewalls excavated within undisturbed native soils should remain near-vertical to depths of at least three feet except where boulder removal may undermine the trench sidewall. Native soils should be considered Type C by OSHA Standards. It is the contractor's responsibility to provide sufficient and safe excavation support per OSHA standards as well as to protect nearby utilities, structures, and other improvements, which may be damaged by earth movements.

5.4.4 For the purposes of this report, structural areas are defined as those areas that may receive future structural improvements such as tanks, buildings, future roadways, existing roadways, parking lots or flatwork areas. Non-structural areas are defined as areas to remain unimproved such as undeveloped right of way, or landscape areas. The project engineer and/or owner should identify such areas in the project documents.

5.4.5 Within <u>non-structural areas</u>, the excavated native soils may be utilized as backfill if properly dried or moisture-conditioned (as applicable). Rocks in excess of 12 inches in diameter and organic material or debris should be removed from such soils prior to use as backfill. Nesting of large rocks (6-inch to 12-inch diameter) should be avoided.

5.4.6 Bedding and pipe zone backfill should extend from the bottom of the trench excavation to a minimum of 12 inches above the crown of the pipe in conformance with the details on the approved plans. Within areas of dry construction, pipe bedding material should consist of clean granular fill materials as specified on the project plans.

5.4.7 Within areas of wet construction (if any) and with the approval of the engineer, pipe bedding material should consist of gravel backfill material as defined by the project Engineer. The granular gravel material should be surrounded entirely with filter fabric (Mirafi 140N or equivalent) to prevent the piping of fines into the bedding material.

5.4.8 Within structural areas granular soils (SM, SW, SP, GM or combinations thereof) should be suitable for use as backfill after removal of any oversize material. Bedding and backfill criteria is provided in the following tables:

TABLE 5.4.8a – PIPE BEDDING CRITERIA

Sieve Size	Percentage by Weight Passing Sieve
3/8 inch	100
No.4	90-100
No 50	10 – 40
No. 100	3 – 20
No. 200	0-15
Plasticity Index	0-10

TABLE 5.4.8b – TRENCH BACKFILL CRITERIA

Sieve Size	Percentage by Weight Passing Sieve
4 inch	100
¾ inch	70-100
No. 40	10 – 50
No. 200	0 – 35
Liquid Limit	40 maximum
Plasticity Index	15 maximum

5.4.9 Trench bottoms may expose wet, soft, or loose soils that are unsuitable for direct support of the pipe. If soft or loose conditions are present as determined by yielding of the subgrade, it is recommended that the contractor over-excavate 12 to 24 inches below the bottom of the trench and place half-inch minus gravel (drain rock) with little or no fines (<5% minus 200) encapsulated by filter fabric (Mirafi 180N or equivalent) to the bottom of pipe elevation. Greater overexcavation and replacement with the gravel may be warranted based on the conditions encountered. Areas to receive such mitigation measures should be reviewed in the field by the Engineer or their representative prior to overexcavation to confirm the need for such mitigation measures.

5.5 Compaction Criteria for Bedding and Backfill

5.5.1 Bedding material within dry trenches should be compacted in lifts not exceeding six inches in loose thickness. The lifts should be compacted to a minimum of 90% relative

compaction at or near optimum moisture content. Gravel materials placed should be tamped or rodded to provide for consolidation prior to placement of additional backfill above the water surface.

5.5.2 Utility trench backfill within structural areas should be placed in loose lifts not exceeding eight inches in thickness. The lifts should be compacted to a minimum of 90% relative compaction at or near optimum moisture content. We anticipate that some of the excavated soil, depending on the time of year, may require time and effort to dry sufficiently so as to achieve 90% relative compaction.

5.5.3 Utility trench backfill within non-structural areas should be placed in loose lifts not exceeding eight inches in thickness. We recommend that the lifts should be compacted to a minimum of 85% relative compaction at or near optimum moisture content.

5.6 Structure Grading

5.6.1 As currently proposed, the proposed tank(s) are planned to be situated in a shallow fill area. Fill depths are anticipated to be from six-inches to two feet in maximum vertical dimension to provide positive drainage from around the tank.

5.6.2 It is recommended that the entire tank pad be overexcavated to a minimum of twelve inches below footing elevation and the native soils replaced with compacted structural fill or Caltrans Class 2 aggregate base. The overexcavation should extend a minimum of two feet outside of the footing exterior edge. The intent of the overexcavation is to minimize the risk of differential settlements. The intent of this recommendation is to provide a uniform bearing surface for foundational support via a uniform thickness of compacted fill beneath load-bearing elements.

5.6.3 The bottom of the removal, should be scarified 8 to 10 inches in depth, moisture-conditioned, and compacted to at least 90% relative compaction prior to placement of fill.

5.6.4 All bottoms of excavations in structural areas should be observed by a representative of our office to verify that all loose native soils have been removed and to confirm adequate depth for the structural fill thickness requirement. The excavation should be backfilled with structural fill or Caltrans Class 2 aggregate base, moistened to near optimum moisture content and compacted to at least 90% relative compaction. Any imported engineered fill should meet the minimum requirements for structural fill or as specified within the approved construction documents. Native soils will require screening to meet the structural fill criteria recommended below in Table 5.6.

TABLE 5.6 - STRUCTURAL FILL CRITERIA

Sieve Size	Percent by Weight Passing Sieve
4 inch	100
¾ inch	70 – 100
No. 40	15 – 70
No. 200	5 – 30
Liquid Limit	40 maximum
Plasticity Index	12 maximum

5.6.5 Fill should be placed in level eight inches (or less) loose lifts. Greater lift thickness may be considered by Geocon in the field, depending on the soil type, compaction equipment, and number of passes. Each lift should be moisture conditioned at or near optimum moisture content and then compacted to a minimum of 90% relative compaction, prior to placement of the next lift of structural fill.

5.6.6 The floor slab of the tank should be underlain by a minimum thickness of eight inches of Caltrans Class 2 aggregate base compacted to at least 95% relative compaction.

5.6.7 Utility trenches beneath and adjacent to the tank footprint should be minimized. Utility penetrations into the tank footprint should be made perpendicular to the foundation. Utilities outside the perimeter of the tank pad should be designed outside of the zone of influence of the foundation. The zone of influence may be taken to be the area beneath the foundation and within a 1:1 plane extending out and down from the bottom of the footing.

5.6.8 Bedding and pipe zone backfill should extend from the bottom of the trench excavation to a minimum of 12 inches above the crown of the pipe. Greater thickness may be required by the tank designer. Pipe bedding material should consist of clean granular materials meeting the minimum specifications shown in Table 5.5.8a above or as indicated in the approved construction documents. Bedding and pipe zone material should be hand compacted in 6-inch maximum lifts.

5.6.9 The top 8-inches of utility trench backfill for trenches located beneath and directly adjacent to the tank or buildings should consist of, Caltrans Class 2 aggregate base, moistened to near optimum moisture content and compacted to at least 95% relative compaction.

5.7 Foundation Design Criteria

5.7.1 Provided the tank pad is graded in accordance with the recommendations of this report, the proposed tank structure may be supported on a conventional shallow foundation bearing entirely on engineered fill.

5.7.2 Foundations should consist of a perimeter strip footing, mat footings or combinations thereof. The bottom of footings should extend at least 24 inches below lowest adjacent pad subgrade. Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom of the footing.

5.7.3 Tank foundations may be designed for an allowable bearing capacity of 2,500 pounds per square foot (psf) for dead plus live loads with a one-third increase for transient loads, including wind and seismic. Steel reinforcing for footings should be provided as required by the structural engineer.

5.7.4 Allowable passive pressure used to resist lateral movement of footings may be assumed to be equal to a fluid weighing 350 pounds per cubic foot (pcf). The allowable coefficient of friction to resist sliding of footings is 0.35 for concrete against soil. Combined passive resistance and friction may be utilized for footing design provided that the frictional resistance is reduced by 50%.

5.7.5 A Geocon representative should observe foundation excavations prior to placing reinforcing steel or concrete to observe that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

5.8 Retaining Wall Design and Lateral Earth Pressures for Retaining Walls and Footings

Retaining structures, although currently not shown on project plans, may be planned in association with the water tank during final design. It is assumed that only short retaining walls would be necessary on the site if at all. If retaining structures are considered in excess of five feet in vertical height, we should be contacted to review our recommendations for appropriateness to the specific conditions.

5.8.1 For design purposes, the soil pressure exerted against a wall may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of this equivalent fluid would depend on the actual design conditions.

5.8.2 The following values are presented for the design of retaining walls or structures with relatively level backfill conditions (overall) within 15 feet of the wall. Should different

backfill configurations or surcharges be anticipated, our office should be contacted for supplemental recommendations.

5.8.3 Passive, active and at rest earth pressures of 350, 45 and 60 pounds per square foot of surface per foot of depth are recommended respectively. A coefficient of friction of 0.35 is recommended for native soils or structural fill beneath or adjacent to concrete structures. A coefficient of friction of 0.40 may be used where at least six-inches of Class 2 aggregate base is in contact beneath structural elements.

5.9 Grading – Pavement

5.9.1 For pavement and flatwork areas, the subgrade should be scarified to a depth of 8 to 10 inches and moisture conditioned at or near optimum moisture content. The upper six inches of pavement subgrade soils should be compacted to a minimum of 90% relative compaction at or near optimum moisture content.

5.9.2 The subgrade soils for pavements should be finished to a compacted smooth unyielding surface. We recommend proof-rolling the subgrade with a loaded water truck (or similar equipment) to verify the stability of the subgrade prior to placing aggregate base.

5.9.3 Aggregate base used to support pedestrian and vehicular pavements should be compacted to a minimum of 95% relative compaction.

5.9.4 Asphalt pavement and aggregate base thicknesses should conform to City of Bishop requirements as presented on the project plans and in the accompanying construction specifications.

5.10 Preliminary Pavement Design Recommendations

The following preliminary pavement sections are intended for on-site parking and driveway areas. Pavement sections are based on Asphalt Institute recommendations for parking areas subject to automobile and truck traffic. The following preliminary Asphalt Concrete pavement sections are recommended for design to establish subgrade elevations for parking and driveways.

TABLE 5.11
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

	AC Thickness (inches)	AB Thickness (inches)
Automobile Parking Areas and Driveways	3.0	6.0
Service Truck or Fork Lift Areas	4.0	6.0

The preliminary pavement section is based on the following assumptions:

- That the pavements will be constructed on at least six-inches of Class 2 aggregate base. It is assumed that subgrade soil has an R-Value of 35 or higher.
- The Class 2 Aggregate Base (AB) has a minimum R-Value of 78 and meets the requirements of Caltrans as adopted by the City of Bishop.
- The aggregate base is compacted to 95% or higher relative compaction at or near optimum moisture content.
- Asphalt concrete should conform to Caltrans specifications.

It is recommended that the use of polymerized asphalt oil or equivalent be considered as we have found that it substantially reduces cracking due to thermal stresses prevalent in the freeze thaw environment of this area. The savings in long term maintenance of the pavement including crack sealing is, in our opinion, worth the extra expense. However, this recommendation is optional in that it is relative to frequency of maintenance only and does not affect structural calculations.

5.12 Slabs-on-Grade

5.12.1 Conventional concrete slab-on-grade floors are suitable for the tank pad prepared as recommended in this report. Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading.

5.12.2 If the soil surface has become dry, then it should be re-moistened prior to placing concrete. The building pad should be moistened by soaking or sprinkling such that the upper 12 inches of soil is near optimum moisture, as determined by our representative at least 48 hours before concrete placement.

5.12.4 Crack control spacing, where specified should be determined by the project structural engineer based on slab thickness and intended usage.

5.12.5 All exterior concrete should be air entrained with from 4.5% to 7.0% air content. The water cement ratio for all exterior concrete should be 0.45 or less. The use of mid-range plasticizer is recommended to facilitate the finishing process while maintaining the desired water cement ratio.

5.12.6 Exterior concrete should be placed and finished in accordance with American Concrete Institute (ACI) recommendations for concrete placed in areas subject to freeze-thaw environments.

5.12.7 Recommendations presented herein are intended to reduce the potential for cracking of slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still exhibit some cracking. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of concrete, the use of crack control joints and proper concrete placing and curing.

Adherence to ACI and Portland Concrete Association (PCA) recommendations including those for low humidity and wind, if applicable, should be incorporated into project construction practices.

5.13 Erosion Control

5.13.1 Erosion control mitigation measures should be as designed by the project civil engineer. Appropriate protection of surface waters will be necessary. Storm water pollution prevention plans and permits should be developed and obtained by the contractor per California State requirements.

5.13.2 Permitting including dewatering, groundwater discharge, storm water, and wetland issues is not included in our scope of work. Should these services be required, Geocon Consultants should be contacted to provide a proposal for the additional scope of work.

6.0 FURTHER GEOTECHNICAL SERVICES

6.1 Plan and Specification Review

6.1.1 Geocon Consultants should review the improvement plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if any additional analysis and/or recommendations are required.

6.2 Testing and Observation Services

6.2.1 The recommendations provided in this report are based on the assumption that Geocon Consultants will continue as Geotechnical Engineer-of-Record throughout the construction phase. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, we should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials or environmental contamination, or environmental permitting was not part of our scope of services.

This report is issued with the understanding that it is the responsibility of the owner or their representative to ensure that the information and recommendations contained herein are brought

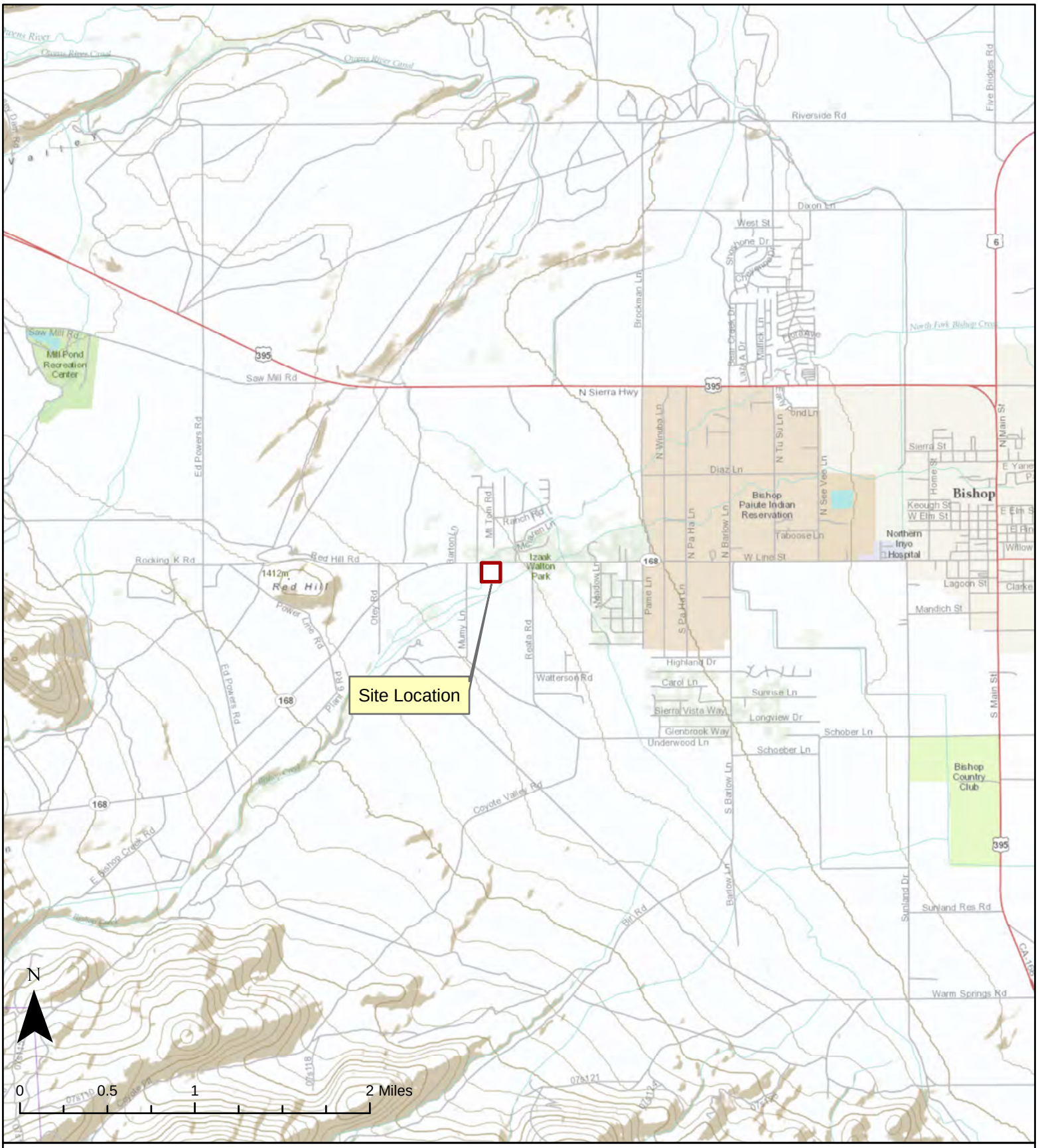
to the attention of the design team for the project and incorporated into the plans and specifications, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field. It is recommended that language in the contract documents clarifies the order of reliance the contractor should place on the plans, specifications and our geotechnical report.

The recommendations contained in this report are preliminary until verified during construction by representatives of Geocon Consultants. Changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. Most importantly, ground water levels are anticipated to fluctuate between the data of exploration and construction. Additionally, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated partially or wholly by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geocon Consultant's professional services were performed, findings obtained, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the project area at this time. No warranty as to the continuity of subsurface conditions is provided, express or implied.

8.0 REFERENCES

1. *Probabilistic Seismic Hazard Assessment for the State of California*, California Division of Mines and Geology, Open File Report 96-08, 1996.
2. *City of Bishop Water Master Plan, Prepared for the City of Bishop Department of Public Works*, by Nolte Associates, Inc., February 2008.
3. *Conceptual site drawings*, provided by Resource Concepts, Inc., January 2012.
4. *Geologic Map of the Bishop 15-Minute Quadrangle, California*, United State Geologic Survey, 1964.
5. *Soil Survey Geographic (SSURGO) database for Benton Owens Valley and Parts of Inyo and Mono Counties*, California, US Department of Agriculture, NRCS, January 2008.
6. *Quaternary Fault and Fold Database for the United States*, accessed January 2012, from USGS website: http://earthquake.usgs.gov/regional/qfaults/



Vicinity Map
50,000-Gallon Water Tank - City of Bishop Well 4 Site
Bishop, California

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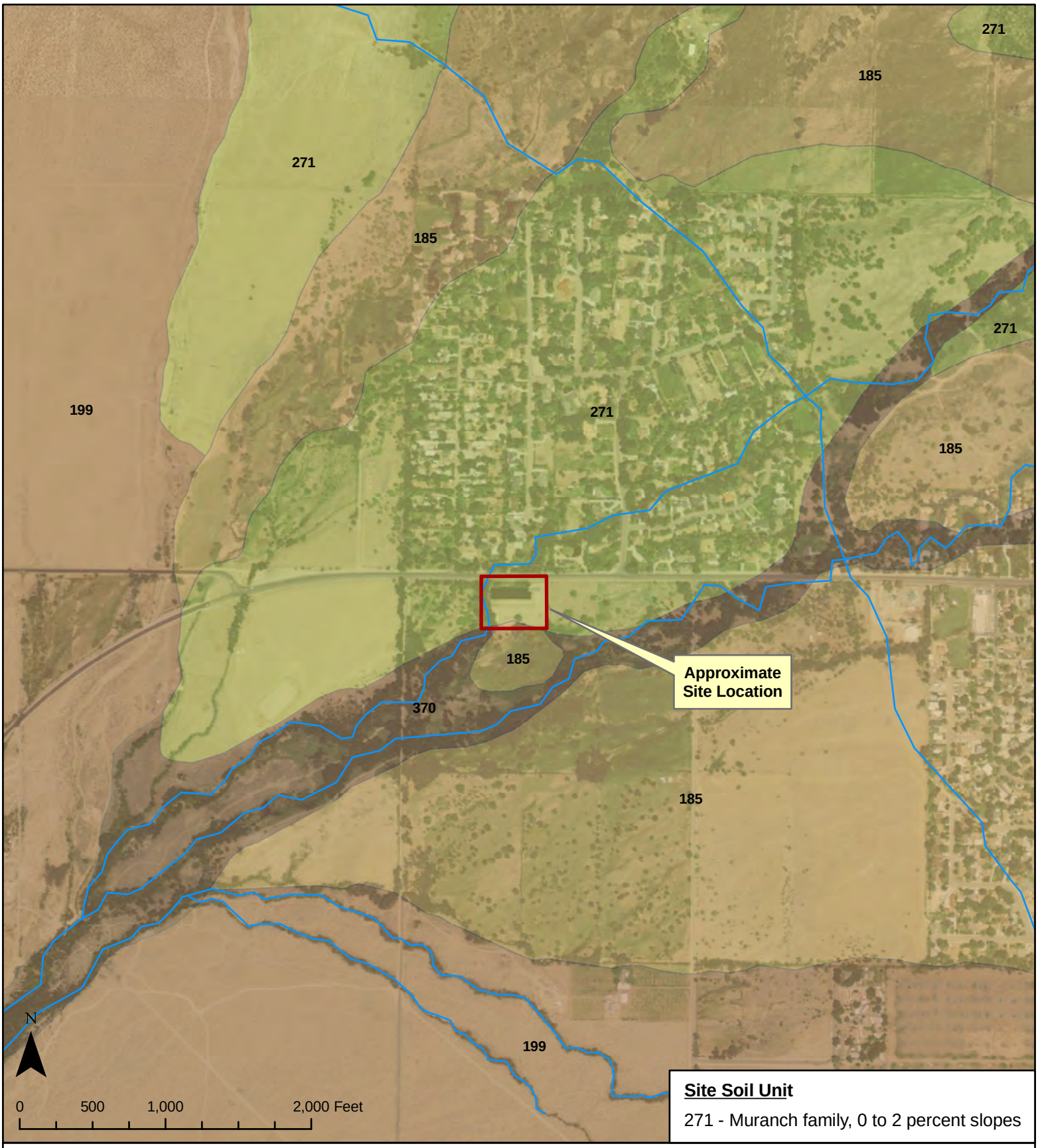
FIGURE 1
MARCH 2012
PROJECT NO.
R8802-06-01

Site Plan
50,000-Gallon Water Tank - Well 4 Site
Bishop, California

GEOCON

Figure 2
Projcet No.
R8802-06-01
March 2012



Soils Map
50,000-Gallon Water Tank - Well 4 Site
Bishop, California

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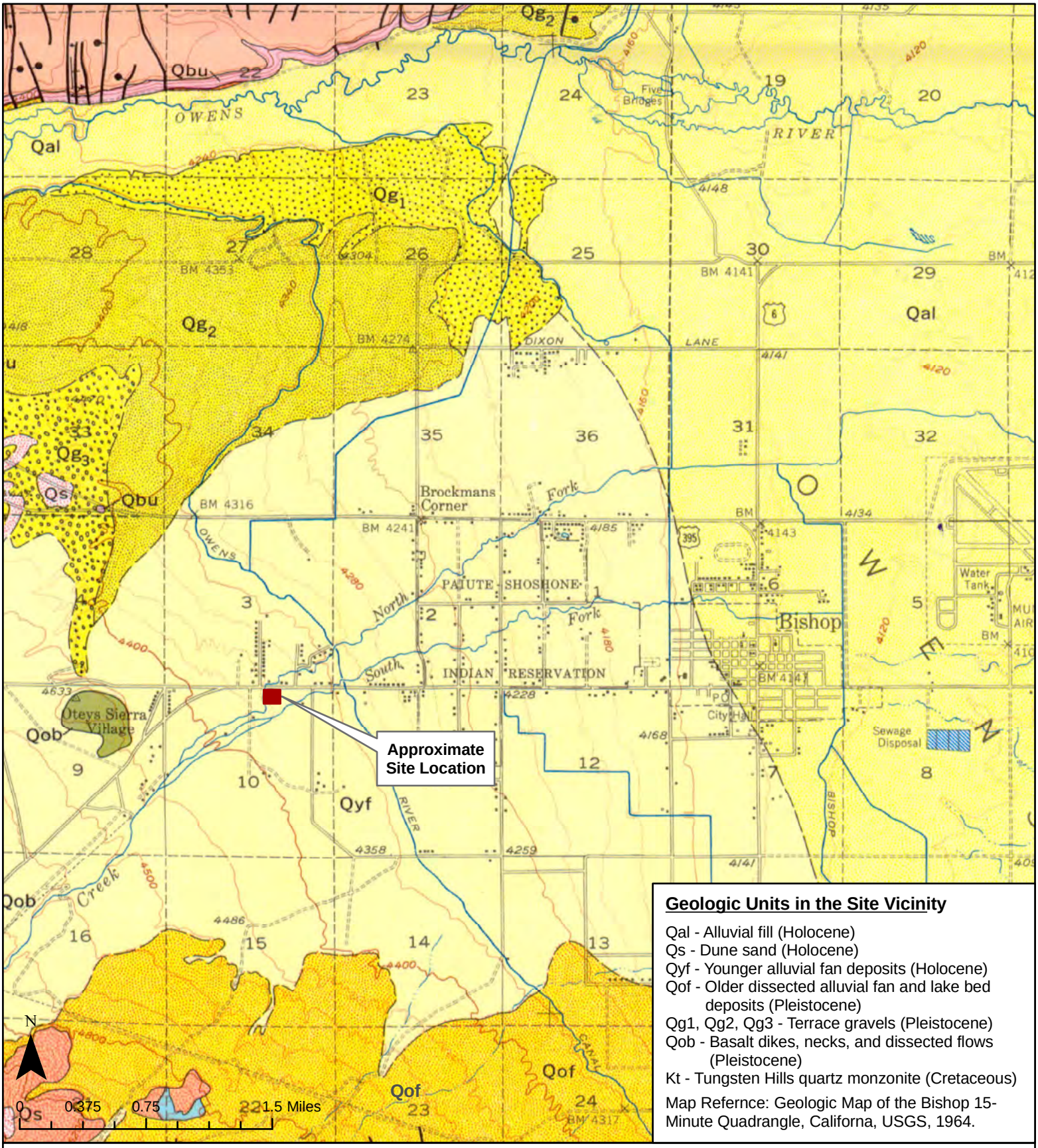
Figure 3
March 2012
Project No.
R8802-06-01



Geologic Units in the Site Vicinity

Qal - Alluvial fill (Holocene)
Qs - Dune sand (Holocene)
Qyf - Younger alluvial fan deposits (Holocene)
Qof - Older dissected alluvial fan and lake bed deposits (Pleistocene)
Qg1, Qg2, Qg3 - Terrace gravels (Pleistocene)
Qob - Basalt dikes, necks, and dissected flows (Pleistocene)
Kt - Tungsten Hills quartz monzonite (Cretaceous)

Map Refernce: Geologic Map of the Bishop 15-Minute Quadrangle, Californa, USGS, 1964.

Geologic Map
50,000-Gallon Water Tank - Well 4 Site
Bishop, California

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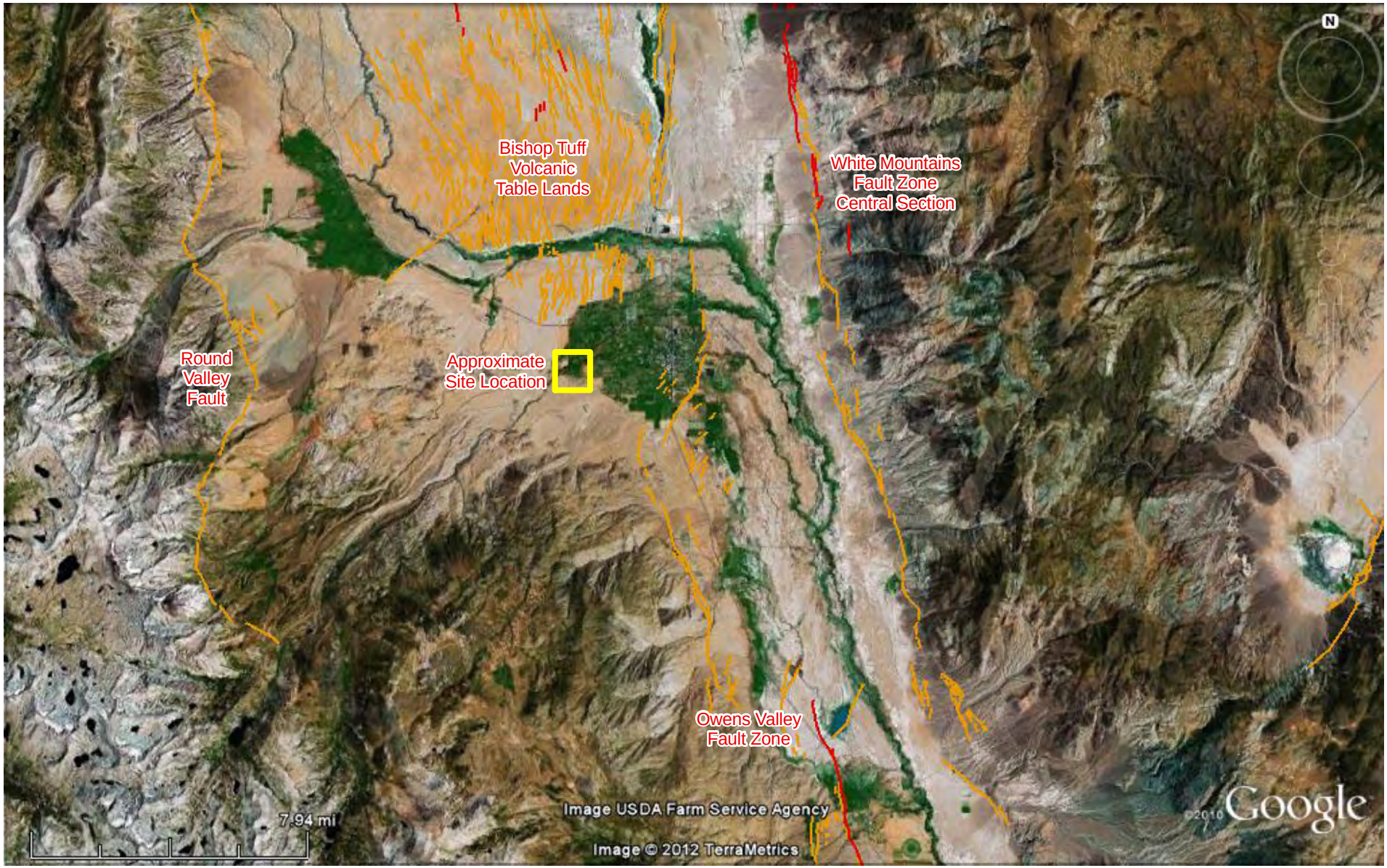
Figure 4
March 2012
Project No.
R8802-06-01



Age of Youngest Fault Displacement

- Historic (less than 150 years)
- Holocene (less than 11,000 years)
- Late Quaternary (130,000 to 15,000 years)
- Middle to Late Quaternary (750,000 to 130,000 years)
- Quaternary (1,600,000 to 750,000 years)

Map Reference: US Geological Survey and Nevada Bureau of Mines and Geology, 2006 Quaternary fault and fold database for the United States, accessed January 2012, from USGS web site: http//earthquake.usgs.gov/regional/qfaults/

REGIONAL FAULT MAP
50,000-GALLON WATER TANK
WELL 4 SITE
BISHOP, NEVADA

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FIGURE 5
MARCH 2012
PROJECT NO.
R8802-06-01

COMMENTS:

LOG OF TEST PIT No. TP-1

LOGGED BY: K. Briggs DATE: 1/30/12

EQUIPMENT: Backhoe

TOTAL DEPTH: 3.5 WATER DEPTH:

MISC. TESTS	R VALUE	UNIT DRY WEIGHT, PCF	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	PASSING NO. 200 SIEVE, %	DEPTH, FT	SYMBOL	SAMPLE	DESCRIPTION / CLASSIFICATION	LAYER ELEV./ DEPTH
										SURFACE ELEVATION:	
			9			21			B	Fill - Grayish brown sand with gravel.	0.2
										Alluvium: Silty Gravelly Sand [SM] - Medium dense to dense, dark brown, moist. Gravel and rounded cobbles up to ~8 inch diameter.	1.5
			3			3			B	Sandy Cobble Gravel [GP] - Dense, slightly moist, reddish yellow. Sand is coarse grained. Rounded cobbles up to ~14 inch diameter.	3.5

COMMENTS:

LOG OF TEST PIT No. TP-2

LOGGED BY: K. Briggs DATE: 1/30/12

EQUIPMENT: Backhoe

TOTAL DEPTH: 3.5 WATER DEPTH:

MISC. TESTS	R VALUE	UNIT DRY WEIGHT, PCF	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	PASSING NO. 200 SIEVE, %	DEPTH, FT	SYMBOL	SAMPLE	DESCRIPTION / CLASSIFICATION	LAYER ELEV./ DEPTH
										Fill - Grayish brown sand with gravel.	
										Alluvium: Silty Sand with Cobbles [SM] - Medium dense to dense, dark brown, moist. Rounded cobbles up to ~8 inch diameter.	0.8
			3			4			B	Sandy Cobble Gravel [GP] - Dense, slightly moist, reddish yellow. Sand is coarse grained. Rounded cobbles up to ~14 inch diameter.	1.8
											3.5

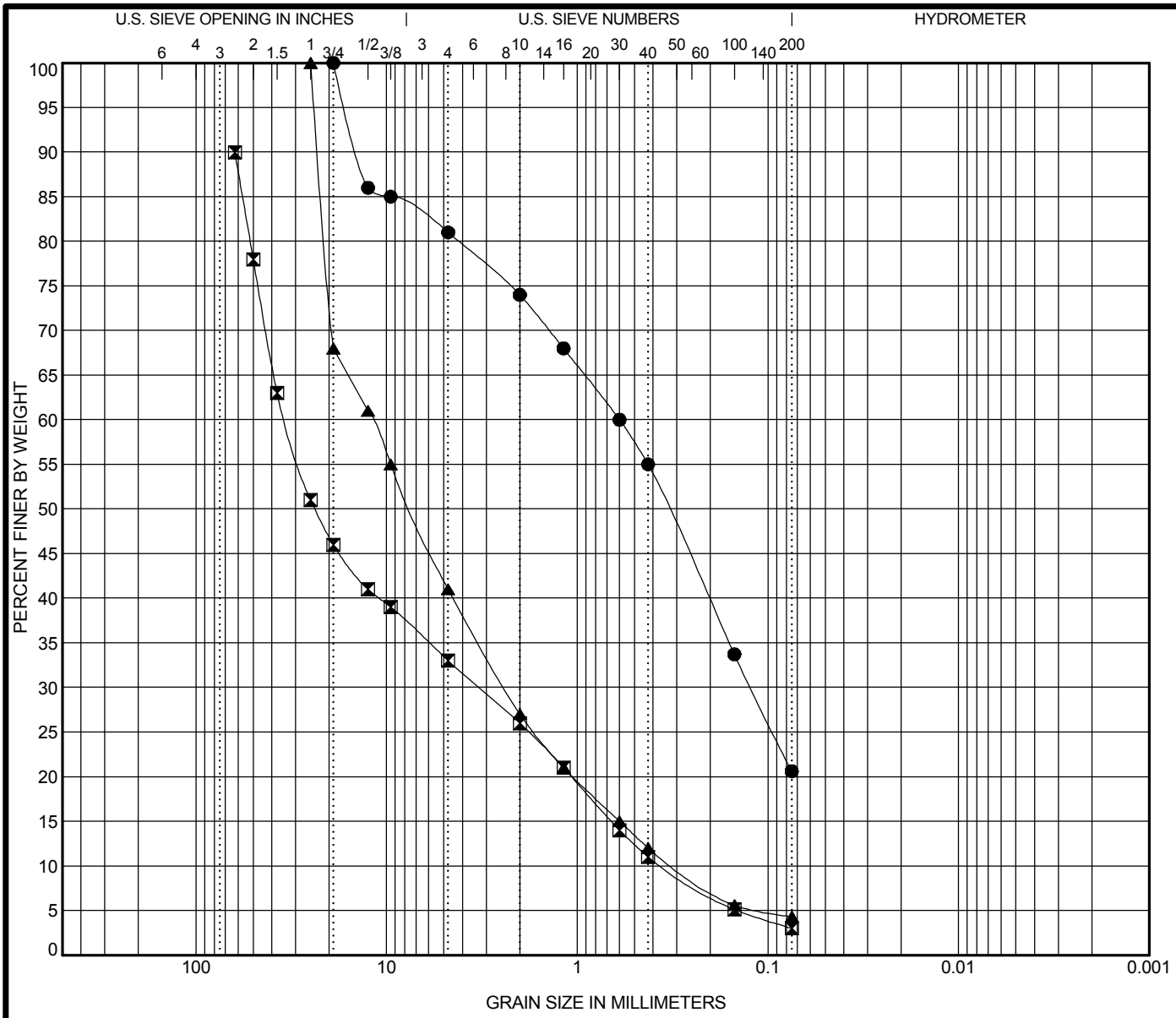
GEOCON CONSULTANTS, INC

PROJECT:

Bishop Water Tank
Bishop, California

JOB NO.: R8802-06-01 DATE: 1/30/12

Figure-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	TP-1	1.3	**Silty Gravelly Sand SM]**					
⊠	TP-1	2.5	**Sandy Cobble Gravel [GP]**				**0.89**	**95.12**
▲	TP-2	2.5	**Sandy Cobble Gravel [GP]**				**1.58**	**38.90**

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-1	1.3	**19**	**0.6**	**0.123**		**19.0**	**60.4**	**20.6**	
⊠	TP-1	2.5	**62.5**	**33.885**	**3.279**	**0.356**	**57.0**	**30.0**	**3.0**	
▲	TP-2	2.5	**25**	**11.941**	**2.407**	**0.307**	**59.0**	**36.7**	**4.3**	

GEOCON CONSULTANTS, INC
Geocon Consultants, Inc.
4010 Technology Way, Ste. D
Carson City, Nevada
775.888.9900

GRAIN SIZE DISTRIBUTION

Project: Bishop Water Tank
Location: Bishop, California
Number: R8802-06-01

Photograph #1,
View of Test Pit TP-1 looking east toward White Mountains.

Photograph #2,
View looking east of backfilled Test Pit TP-2 showing ReMi Line 2.
Insert showing fill over cobble gravel.

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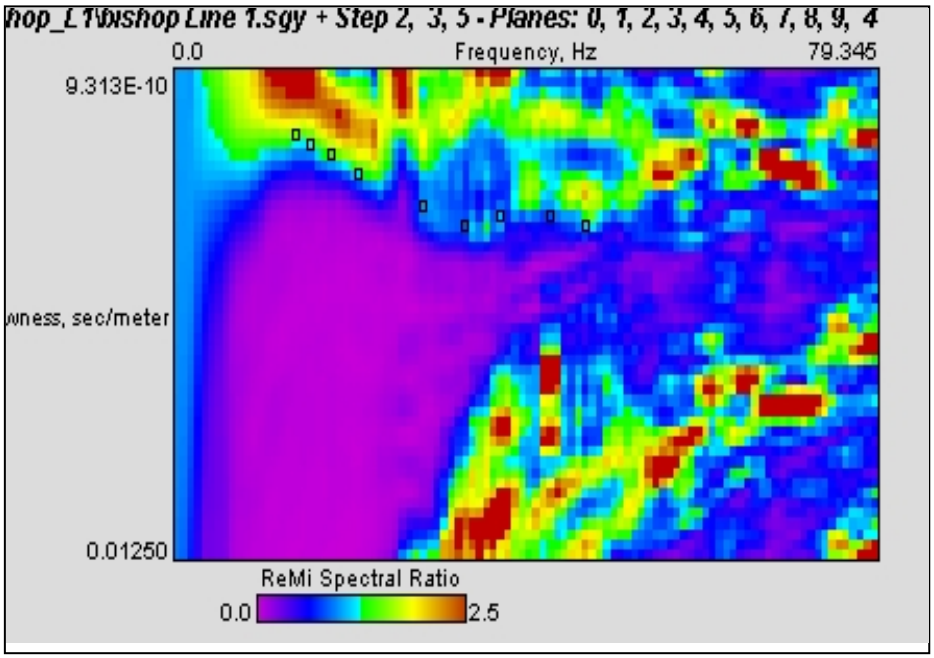
SITE PHOTOGRAPHS 1 and 2

BISHOP TANK
IMPROVEMENT PROJECT
BISHOP, CALIFORNIA

GL	MARCH 2012	PROJECT NO. R8802-06-01



Bishop Line 1

Bishop Line 1
Velocity Profile

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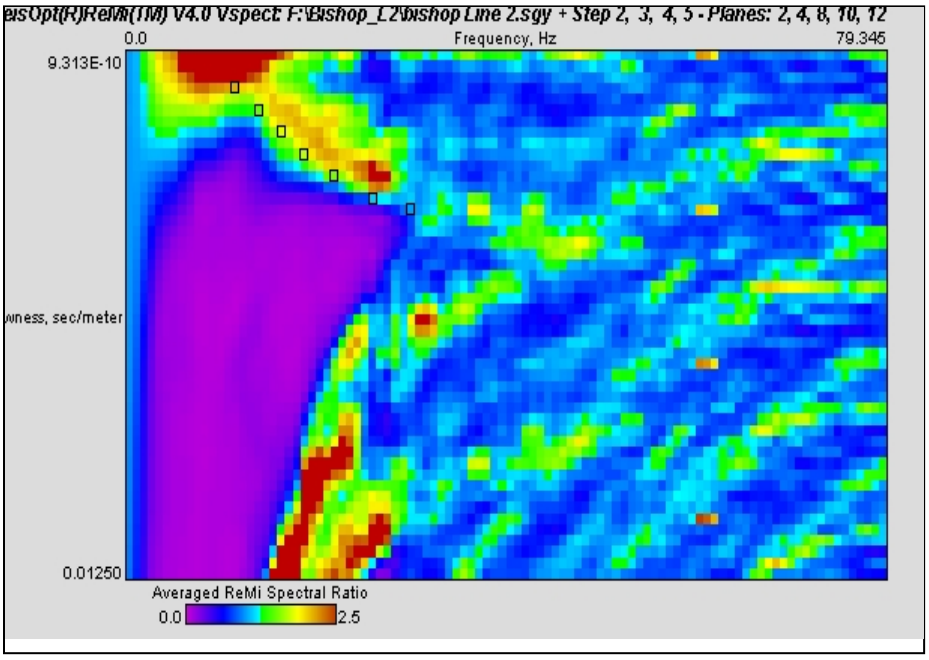
KRB

ReMi Line 1

BISHOP WATER TANK
MONO COUNTY, CALIFORNIA

PROJECT NO. R8802-06-01



Bishop Line 2

Bishop Line 2
Velocity Profile

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ReMi Line 2

BISHOP WATER TANK
MONO COUNTY, CALIFORNIA

KRB

PROJECT NO. R8802-06-01