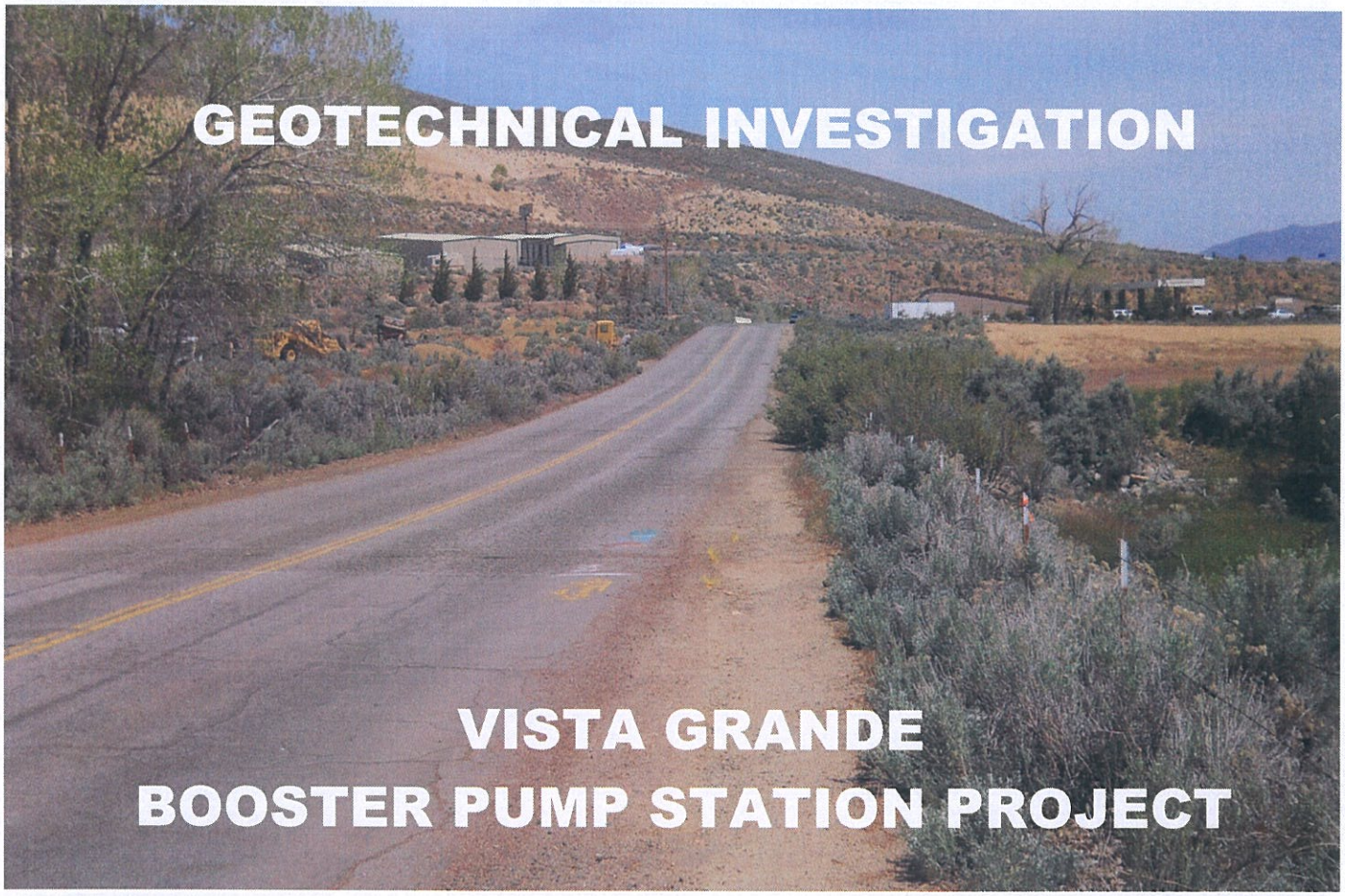
ATTACHMENT D



GEOTECHNICAL INVESTIGATION

VISTA GRANDE
BOOSTER PUMP STATION PROJECT

PREPARED FOR:

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GEOCON PROJECT NO. R8757-06-01

GEOCON

JUNE 2011

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. R8757-06-2011
June 1, 2011

Ms. Rebecca, Bernier, PE
Manhard Consulting, Ltd.
3476 Executive Pointe Way, Suite 12
Carson City, Nevada 89706

Subject: GEOTECHNICAL ENGINEERING INVESTIGATION
FOR A PORTION OF THE COSTCO INTER-TIE/VISTA GRANDE
WATER LINE IMPROVEMENT PROJECT
CARSON CITY AND DOUGLAS COUNTY, NEVADA

Dear Ms. Bernier:

We are pleased to submit the results of our geotechnical engineering investigation for a portion of the proposed Costco Inter-tie/Vista Grande Water Line Improvement Project. The accompanying report presents the findings and conclusions from our study of a portion of the overall project. In our opinion, the primary geotechnical considerations in the design and construction of the proposed Vista Grande Boulevard roadway realignment, water line and Vista Grande Booster Pump Station improvements include: (1) the drainage ditch running through the footprint of the proposed building requiring overexcavation and replacement with structural fill; (2) significant design and construction consideration includes the presence of various existing underground utilities along the existing road alignments. These constraints 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.

PROFESSIONAL ENGINEER-STATE OF NEVADA
GARY C. LUCE
CIVIL
No. 12001
EXPIRES 12-31-11

Gary Luce, PE
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Kiersten Briggs, CEM
Project Geologist

(5) Addressee

(1) File

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GEOTECHNICAL INVESTIGATION

1.0 PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for a portion of the proposed Costco Inter-tie/Vista Grande Water line Improvement Project, Carson City and Douglas County, Nevada.

The overall Costco Inter-tie/Vista Grande Waterline Improvements project will extend from the Topsy Lane and Vista Grande Boulevard intersection north to Clear Creek Road where a new intersection is proposed approximately 100 feet east of the existing intersection that will line up with an existing Costco driveway. The remainder of the project will extend approximately 1,200 feet east along Clear Creek Road. The purpose of the overall project is to provide additional capacity and connectivity to the Carson City water system.

Our scope of work is limited to the central portion of the project that includes:

1. The proposed Vista Grande Booster Pump Station building and related improvements.
2. The Vista Grande Boulevard realignment and related utilities and improvements.

The location of our portion of the project is illustrated on the project Vicinity Map, Figure1.

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

- Project drawings including a preliminary site map of the proposed booster pump station, plan and profile of the proposed Vista Grande Boulevard realignment and water line improvements in the portion of the project within our scope of work. The drawings were provided to our office by Manhard Consulting, Ltd.
- Review of regional geologic, soils and flood zone maps pertinent to our project limits.
- Traffic data supplied by Carson City Engineering for Vista Grande Boulevard dated April 2011.

The purpose of our geotechnical investigation was to observe the prevailing soil conditions in our project area, and based on conditions encountered; provide site-specific recommendations relative to the geotechnical aspects of constructing the realigned roadway, the booster pump station facilities and installing the respective portions of proposed waterlines.

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 the Manhard Consulting to determine the locations of our exploratory excavations and borings.

- Performed a site reconnaissance visit and marked the proposed exploratory locations in the field with stakes and white paint for subsequent underground utility location purposes.
- As required by law, notified local utility subscribers via Underground Service Alert (USA) at least 48 hours prior to performing subsurface excavations.
- Observed the advancement of four soil borings (BH-1 through BH-4).
- The test borings were excavated to depths between 5.5 and 20.1 feet. Upon completion, the test borings were backfilled with the excavated soil and in pavement areas cold patched placed matching the existing pavement.
- Logged the borings in accordance with the Unified Soil Classification System (USCS).
- Obtained representative samples from the soil borings. 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.
- 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 boring logs are presented in Appendix A. Approximate locations of the exploratory borings are shown on Figure 2, the Project 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 as defined for our investigation includes approximately 1,000 feet of water line and roadway construction in the existing and proposed realigned portions of Vista Grande Boulevard as well as the new Vista Grande booster pump station. In addition, the existing storm drain will be relocated in the area of the road realignment. The approximate project alignment and associated elements are shown on the Project Site Map, Figure 2.

The proposed booster pump station facility is to be located on the northwest corner of what will be the reconfigured Vista Grande Boulevard-Clear Creek intersection. The proposed facility will include a approximately 1,292 square-foot metal or masonry building to house pumping facilities, related underground pipelines, emergency generator pad, vaults and valves. Associated improvements will also include concrete flatwork and asphalt concrete paving, around the building, in parking areas and for the driveway access.

As part of the overall Costco Inter-tie/Vista Grande Water Line project, a single 14-inch diameter water line will extend from Topsy Lane located south of our project area along Vista Grande Boulevard to the booster pump station. A 12-inch diameter line will leave the pump station and extend east along Clear Creek Road. The portions of the two pipelines considered in our project limits are depicted on the Site Plan, Figure 2.

Existing utilities in our project area include, but are not limited to, overhead power lines, underground power, gas lines, water line, storm drain and sanitary sewer. The description of observed utility features was accomplished via a site visit, review of preliminary plans and visual observations only. Potholing or surveying of individual utilities was not included in our project scope.

The topography across the proposed roadway and water main alignments ranges from a moderate slope on the south to a flat low lying area along Clear Creek and a gentle slope up to Clear Creek Road. The booster pump station is located at an elevation of approximately 4,790 feet above mean seal level (AMSL). Both the roadway and water line alignments slope down to the south at 1.7 percent for approximately 500 feet to the low point and then slope upward at 6.9 percent to the end of the project.

The alignment crosses one stream, Clear Creek, which is channeled in 72-inch corrugated metal pipe beneath Vista Grande Boulevard. Additional roadside drainages will be crossed including the roadside ditches along the east side of Vista Grande Blvd. and along Clear Creek Road.

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 and borehole logs included in Appendix A for vertical extents of the materials encountered at each location.

3.2 Soil Conditions

The project site is situated primarily within the Carson City Urban Area but crosses into northern Douglas County. The following descriptions use the Carson City soil map units that have different numbers on the Douglas County soil maps. A soil map depicting the soil units in the site vicinity is presented as Figure 3. According to the *Soil Survey of Carson City Area, Nevada* (US Department of Agriculture: V.3, 2006), the proposed booster pump station, water lines, and road realignment will cross three soil types: Surprise Sandy Loam, 8-15% slopes (Soil Unit 60); Prey Gravelly Loamy Sand, 0-4% slopes (Soil Unit 712); and Jubilee Sandy Loam, 0-4% slopes (Soil Unit 37).

Surprise Sandy Loam is listed as fine silty sand to approximately 15 inches underlain by silty sand (SM) to gravelly sand (GM) to five feet.

Prey Fine Sandy Loam is listed as fine sandy loam (SM) from the surface to a depth of thirteen inches. Under the fine sandy loam is sandy loam (SM) to a depth of 30 inches, which is underlain by cemented material to a depth of 35 inches. Under the cemented layer is coarse silty sand (SM) that extends to a depth of 60 inches.

Jubilee Sandy Loam is described as a surface 12-inch thick layer of silty sand (SM) that are underlain by stratified layers of coarse to fine silty sands (SM).

Findings from our exploratory borings are in general agreement with the USDA's mapping of the site.

It should be noted that the pipeline installation depths will exceed the five feet considered by the NRCS mapping and therefore the boring data should be reviewed and the mapping considered as only a general guide.

3.3 Geologic Conditions

The project site is located in the lower portion of Clear Creek Canyon just west of the topographic divide between Eagle Valley and Carson Valley. Clear Creek is a tributary to the Carson River. Clear Creek flows eastward from near the crest of the Carson Range into Carson Valley and joins the Carson River a few miles to the southeast of the project site. Carson and Eagle Valley are fault bounded sedimentary basins with the Carson Range to the west and the Pine Nut Range to the east. Geologic mapping of the site is published on the Nevada Bureau of Mines and Geology (NBMG) *Genoa Quadrangle Geologic Map,, Nevada,* (1980). A portion of this map is reproduced in Figure 4, the Site Geology Map.

All of the geologic units are interpreted to be Quaternary in age (less than 1.65 million years) with the sediments along Clear Creek being the youngest. On the southern half of our project area are Pediment Deposits of Indian Hills (QTg). This unit corresponds with the Prey soil unit. A small portion near the center of the alignment consists of Flood-plain deposits of Clear Creek (Qvc). These soils correspond to the Jubilee soil unit. Along Clear Creek Road older Alluvial-plain deposits of Clear Creek are present. This soil generally corresponds with the Surprise soil unit.

3.4 Groundwater

The proposed project alignment is located within a portion of the Carson Valley where groundwater depths can be anticipated to vary greatly. A review of Well Driller's Log Reports available on the Nevada Department of Water Resources website and the *Genoa Quadrangle Groundwater Map,* (1`992), indicate that depth to groundwater in the vicinity of the proposed water main alignment is generally found between 10 and 20 feet in lower areas of our project

site.No groundwater was encountered in our borings at the booster pump station at approximately 15 to 20 feet below the existing surface in April of 2011. Although groundwater was generally not encountered at the depths anticipated to be impacted by construction, during wet seasons, near Clear Creek or after significant rainfall events zones of "Perched" groundwater seepage may be encountered in highly permeable soils that are hydraulically connected to surface or groundwater sources. Groundwater depths may vary due to fluctuations in rainfall, temperature, and other 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 the Soil Report for the Carson City Area, USDA, 2006, our experience in this area, and limited test results from our investigation, the soils along the project alignment 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.

Due to the variability of the soils, 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

Carson 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 and Nevada Bureau of Mines and Geology, 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) fault is mapped approximately 1,500 feet north of the Booster Pump Station site. The fault, referred to as the Carson City Fault trends northeasterly along the hill across US 50. The fault is well defined at this location but approximately a half mile north of the site separates into a distributed fault or series of short sub parallel segments as shown on the Fault Map, Figure 5.

Seismic activity and significant ground-shaking are anticipated to occur during the design life of the proposed waterline. Ground shaking intensities for design considerations should be governed by seismic events occurring on the Genoa Fault which follows the base of the Carson Range approximately 3.0 miles to the west of the site. Faulting along the Carson Range has been

evaluated by the Nevada Bureau of Mines and Geology to be capable of producing earthquake Richter Magnitudes on the order of 7.2 with peak ground accelerations as high as 1.5g. These values are equivalent to Modified Mercalli Intensities of X or greater.

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

4.2 Liquefaction and Related Hazards

4.2.1 Roadway Water Line Alignments

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 typically results in disruption of buried pipelines due to buoyant forces. Disruptions to roadways can also occur but are generally not considered for low volume roadways.

Based on the exploration of the site and Geocon's previous experience in this portion of Carson City, the risk of liquefaction along our alignment varies from very low in higher elevation areas to moderate along the lower elevation portions of the project along Clear Creek. The area along and within the banks of Clear Creek may be subject to liquefaction. Localized lateral spreading may also be possible in this area.

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.

4.2.2 Booster Pump Station Site

Soil conditions encountered in the area of the booster pump station site were found to have high blow counts as determined by Standard Penetration Tests (SPT). Blow counts below 10 feet, were documented to range from a low of approximately 20 per foot to over 100 per foot with the average near the latter number. Based on this data, the potential for liquefaction at the Booster Pump Station site is considered low to very low.

4.3 Flood Hazards

A review of the FEMA Flood Insurance Rate Maps (FIRM) that encompasses the proposed water main alignment and Booster Pump Station site (Panels: 3200010207E, and 32005C0060G)

indicate that most of the project is located within Flood Hazard Zone X and Flood Hazard Zone AE as shown on the Flood Map, Figure 6.

Zone X is defined by FEMA as:

"...areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees for 1% annual chance flood."

Zone AE is defined as:

"...areas subject to inundation by the 1% annual chance flood Base flood elevations determined for Zone AE."

The area within Zone X is the northern most portion of the project where Vista Grande Boulevard will be realigned. The site of the Booster Pump Station and new intersection of Vista Grande Blvd. appears to be outside of, but very close to the Zone X boundary according to the mapping. The Roadway portion of the project south of Clear Creek is mapped as in Zone AE

The impact of flood hazards is primarily of concern during construction, particularly during wet seasons, due to sheet flows and where small dry washes cross the project.

4.4 Landslides and Slope Stability

In the vicinity of the entire water main alignment it is noted that the topography is generally hilly to flat lying with only moderate slopes. 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 Vista Grande Booster Pump Station site and waterline alignments are suitable for the proposed 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 alluvial soils. Soils are characterized by alluvial, and flood deposits, of variable age and texture. The booster pump station site is predominantly underlain by medium dense to dense alluvial soils. Please refer to the boring logs presented in Appendix A.

5.1.3 Groundwater was not encountered in our test borings and is inferred to be approximately 20 feet below the surface based on our explorations, well data review and the Genoa Quadrangle Groundwater Map. However, the presence of shallow perched groundwater conditions during wet periods or seasons should be considered possible. The contractor should pothole the alignment if construction will commence during wet seasons or periods.

5.1.4 The majority of the project is within flood zones as defined by FEMA, therefore the contractor should monitor weather and flow conditions within the project area so that the job site, materials, and equipment can be secured in advance of any potential floods. Should flooding occur, wet trench conditions in those areas may be encountered. Protection of work during precipitation periods should be planned for.

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 Carson City. 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 Fault Rupture Mitigation and Seismic Design Criteria

5.2.1 The site is located near faults capable of generating strong seismic shaking during the life of the project. There are no known surface expressions of active Holocene or Quaternary faults underlying the proposed roadway/pipeline alignments or the booster pump station. The risk of ground rupture due to faulting is considered low throughout the project area. Potential seismic hazards at the site will likely be associated with possible moderate to strong ground shaking from an event along regional active faults.

5.2.2 The liquefaction potential is estimated to be low to moderate in the project area. In the area of the booster pump station improvements, the risk of liquefaction is considered low. No specific ground improvement measures are recommended for the project to prevent liquefaction, since the booster pump station area has a low risk and mitigation measures are not customary for non-hazardous pipelines.

5.3 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 2006 IBC 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). Seismic information for 2006 IBC design is provided in the following table.

TABLE 5.3
2006 IBC SEISMIC DESIGN PARAMETERS

Parameter	Factors	IBC Reference
Site Class	D	Table 1613.5.2
Spectral Acceleration	S_s = 1.690 S_1 = 0.702	Figure 1613.5(1) Figure 1613.5(2)
Seismic Coefficient, F_a	F_a = 1.000	Table 1613.5.3(1)
Seismic Coefficient, F_v	F_v = 1.500	Table 1613.5.3(2)
Adjusted Spectral Response S_{MS}, S_{M1}	S_{MS} = 1.690 S_{M1} = 1.053	Equation 16-37 Equation 16-38
Design Spectral Acceleration S_{DS}, S_{D1}	S_{DS} = 1.127 S_{D1} = 0.702	Equation 16-39 Equation 16-40

5.4 Grading – General, Site Preparation

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

5.4.2 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*.

5.4.3 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.

5.4.4 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.

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

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

5.4.7 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.

5.5 Soil Handling, Excavation Characteristics, Bedding and Backfill Criteria

5.5.1 In our opinion, excavation will require conventional grading/excavation equipment suitable for excavation of loose to dense, sand and sandy gravels.

5.5.2 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.

5.5.3 Temporary excavations, such as utility trench sidewalls excavated within undisturbed native soils should remain near-vertical to depths of at least three feet. 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.

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

5.5.5 Within non-structural areas, the excavated native soils may be utilized as backfill if properly dried or moisture-conditioned (as applicable). Rocks in excess of four inches in diameter and organic material or debris should be removed from such soils prior to use as backfill.

5.5.6 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 Carson City Standard Details. Within areas of dry construction, pipe bedding material should consist of Class A backfill material as defined by the *Standard Specifications for Public Works Construction (Orange Book).*

5.5.7 Within areas of wet construction (if any) and with the approval of Carson City Utility Department, pipe bedding material should consist of Class C backfill material as defined by the *Orange Book.* The Class C material should be surrounded entirely with filter fabric (Mirafi 140N or equivalent) to prevent the piping of fines into the Class C material.

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

TABLE 5.5.8a – CLASS A BACKFILL 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	Per Orange Book Table I, 200.01.01

TABLE 5.5.8b – CLASS E 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	12 maximum

5.5.9 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 Class C backfill encapsulated by filter fabric (Mirafi 180N or equivalent) to the bottom of pipe elevation. Greater overexcavation and replacement with Class C Backfill 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.

5.6 Compaction Criteria for Bedding and Backfill

5.6.1 Bedding material (Class A backfill) 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. Class C materials placed should be tamped or rodded to provide for consolidation prior to placement of additional backfill above the water surface.

5.6.2 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 soils may require time and effort to dry sufficiently so as to achieve 90% relative compaction.

5.6.3 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.

5.7 Structure Grading

5.7.1 As currently proposed, the Booster Pump Station building is planned to be situated in a cut-fill area. The building will straddle an existing drainage ditch that is from three to four feet deep and up to approximately 10 feet wide. The ditch currently conveys road drainage along the east side of Vista Grande Blvd. to a culvert pipe which is located approximately 35 feet east of the southeast corner of the intersection. The drainage ditch cuts through the building footprint from approximately the southwest to the northeast corner.

5.7.2 It is recommended that the entire booster pump building pad be overexcavated to a minimum of three feet below footing elevation. The overexcavation should extend a minimum of five feet outside of the building perimeter measured from the edge of the footing stem wall. 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 relatively thickness of compacted fill beneath load-bearing elements.

5.7.3 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.

5.7.4 All bottoms of excavations 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 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 as defined in the Standard Specification for Public Works as shown below:

TABLE 5.7- 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
Plastic Index	12 maximum

5.7.5 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.

5.7.6 The floor slab should be underlain by a minimum thickness of eight inches of Type II, Class B aggregate base compacted to at least 95% relative compaction.

5.7.7 Utility trenches beneath and adjacent to the building footprint should be minimized. Utility penetrations into the building footprint should be made perpendicular to the foundation. Utilities outside the perimeter of the building 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.

5.7.8 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 Class A Backfill material as

defined by the *Orange Book*. Bedding and pipe zone material should be hand compacted in 6-inch maximum lifts.

5.7.9 Utility trench backfill for trenches located beneath and directly adjacent to the building should consist of Type II, Class B aggregate base, moistened to near optimum moisture content and compacted to at least 95% relative compaction.

5.8 Foundation Design for Pump Buildings and Accessory Structures

5.8.1 Foundations for the Pump buildings or any small accessory structures should have a minimum width of 12 inches and be embedded at least 24 inches below the exterior grade for frost protection and confinement.

5.8.2 Foundations prepared in accordance with the recommendations presented in this report may be designed on an allowable bearing capacity of 2,500 psf.

5.8.3 The allowable bearing capacity is for dead plus live loading conditions. A one-third increase may be used to evaluate transient total loads including wind and seismic forces.

5.8.4 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 per foot of depth. The allowable coefficient of friction to resist sliding is 0.35 for concrete cast against the structural fill or native alluvium. Combined passive resistance may be utilized for footing design provided the frictional resistance is reduced by 50%.

5.8.5 Settlements on foundations constructed and designed as described are estimated to be less than one inch total and three-quarters (¾) inch differential respectively.

5.9 Retaining Wall Design and Lateral Earth Pressures

Retaining structures although currently not shown on project plans, may be planned in association with the Booster Pump Station and 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.

5.9.1 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.

5.9.2 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.

TABLE 5.10- LATERAL EARTH PRESSURES – LEVEL BACKFILL

Condition	Equivalent Fluid Pressure for Level Backfill Condition (pcf)
Active Lateral Earth Pressure	45
Passive Lateral Earth Pressure	350
At-Rest Lateral Earth Pressure	65

5.10 Grading – Pavement

5.10.1 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.

5.10.2 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.

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

5.10.4 Asphalt pavement and aggregate base thicknesses should conform to Carson City requirements as presented on the project plans and in the accompanying specifications.

5.11 Preliminary Pavement Design Recommendations

The following preliminary pavement sections are intended for Vista Grande Boulevard. and on-site parking and driveway areas. Pavement sections are based on AASHTO methods for roadways and on the 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 roadways, parking and driveways.

TABLE 5.12a
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

	AC Thickness (inches)	AB Thickness (inches)
Vista Grande Boulevard	4.0	8.0
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:

- The majority of the new roadway will be constructed on imported fill. It is assumed that subgrade soil has an R-Value of 35 or higher.
- The ADT for Vista Grande is assumed to be 950 based on traffic data supplied from Carson City collected in April of 2011.
- Truck distribution was conservatively assumed to comprise approximately 4% of all traffic (Carson City data from 2011 showed less than 2%).
- Growth was assumed to average four percent over the 20 year design life.
- The Type 2, Class B Aggregate Base (AB) has a minimum R-Value of 70 and meets the requirements of the Standard Specifications for Public Works.
- The aggregate base is compacted to 95% or higher relative compaction at or near optimum moisture content.
- Asphalt concrete should conform to Section 320.02 of the Orange Book.

It is recommended that the use of PG64-29NV (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 effect structural calculations.

5.13 Slabs-on-Grade

5.13.1 Conventional concrete slab-on-grade floors are suitable for the building pads prepared as recommended in this report. A minimum 10-mil-thick vapor retarder meeting ASTM E1745-97 Class C requirements may be placed below the slab where interior moisture is considered undesirable. The vapor retarder may be covered by an optional 2-inch layer of medium sand as a cushion. To reduce the potential for punctures, a higher quality vapor retarder (15 mil, Class A or B) may be used. The vapor retarder, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations. Slabs should be underlain by a minimum of 4 inches of compacted (95% minimum relative density) aggregate base. Slab thickness and reinforcement should be determined by the structural engineer based on the anticipated loading.

5.13.2 If a significant amount of time has passed since building pad grading and the soil surface of the building pad has become dry, then it should be re-moistened prior to placing the moisture retarding system. 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.

5.13.3 Some floor coverings, such as tile or linoleum, are sensitive to moisture that can be transmitted from and through the slab. Slab floors should be moist cured for a minimum of 7 days prior to placing any floor coverings. Floor coverings should be installed in accordance with the manufacturer's recommendations including any moisture transmissivity testing requirements.

5.13.4 Crack control spacing should be determined by the project structural engineer based on slab thickness and intended usage.

5.13.5 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.

5.13.6 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.

5.13.7 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.

5.14 Erosion Control

5.14.1 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 Nevada State requirements.

5.14.2 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 Carson City area at this time. No warranty as to the continuity of subsurface conditions is expressed or implied.

8.0 REFERENCES

Digital Flood Insurance Rate Map Nos.: 320001011E, 3200010113E, 3200010226E, and 3200010207E, all dated January, 2009, Carson City & Douglas County, Nevada, Digital Flood Insurance Rate Map accessed May, 2010, from website: https://hazards.fema.gov/wps/portal/mapviewer

Douglas County Water System Improvements, Water System Overview, provided by Manhard Consulting, Ltd.

Earthquake Hazards Map, Genoa Quadrangle, Nevada Bureau of Mines and Geology, 1981

Genoa Quadrangle Groundwater Map, Map 1Cf, Nevada Bureau of Mines and Geology, 1992

Geology of the Carson City 30 X 60 Minute Quadrangle, Nevada, Nevada Bureau of Mines and Geology, 1999

Geotechnical Investigation Report, Proposed North Valley/BLM Infrastructure-Phase II, Douglas County, Nevada, Kleinfelder, January 14, 2005

Quaternary Fault and Fold Database for the United States, accessed May, 2010, from USGS website: http://earthquake.usgs.gov/regional/qfaults/

Soil Survey Geographic (SSURGO) database for Carson City Area, Nevada, US Department of Agriculture, NRCS, 2006

Standard Specifications for Public Works Construction, Washoe County RTC, 2007

State of Nevada, Division of Water Resources, Well Driller's Log Database (http://water.nv.gov/engineering/wlog/wlog.cfm) accessed January, 2011

VICINITY MAP FOR A PORTION OF:
COSTCO INTER-TIE/VISTA GRANDE WATER LINE IMPRROVEMENT PROJECT
CARSON CITY AND DOUGLAS COUNTY, NEVADA

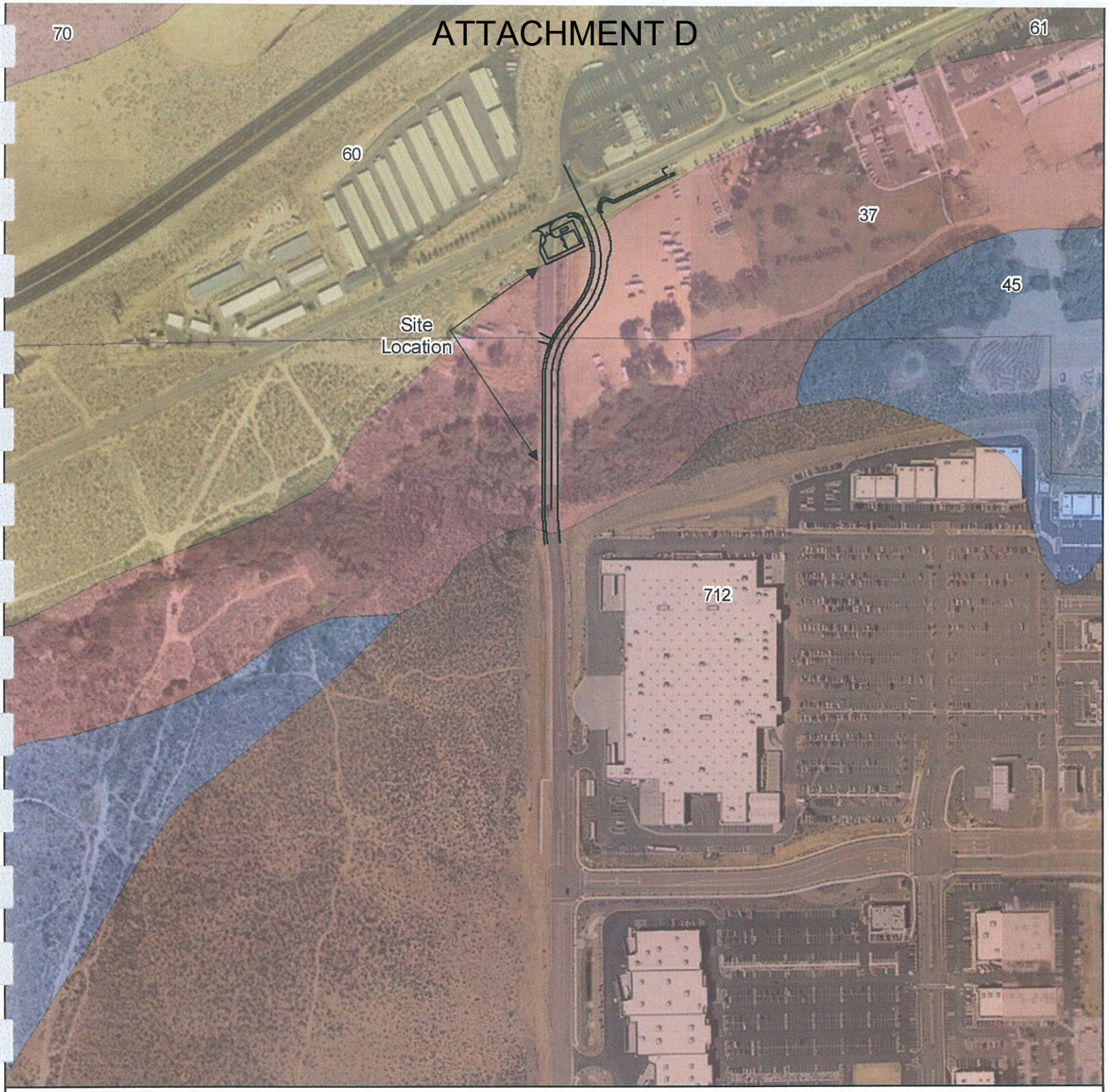
FIGURE 1
JUNE 2011
PROJECT NO.
R8757-06-01

GEOCON

PROJECT SITE MAP FOR A PORTION OF:
COSTCO INTER-TIE/VISTA GRANDE WATER LINE IMPROVEMENT PROJECT
CARSON CITY AND DOUGLAS COUNTY, NEVADA

GEOCON

FIGURE 2
JUNE 2011
PROJECT NO.
R8757-06-01



SITE SOIL UNITS

37 - Jubilee sandy loam, 2 to 4 percent slopes
60 - Surprise sandy loam, 8 to 15 percent slopes
712 - Prey gravelly loam, 0 to 4 percent slopes

Reference: USDA, Natural Resource Conservation Service, Soil Survey Geographic (SSURGO) Databases for Carson City Area and Douglas County Area, Nevada, December 2006.

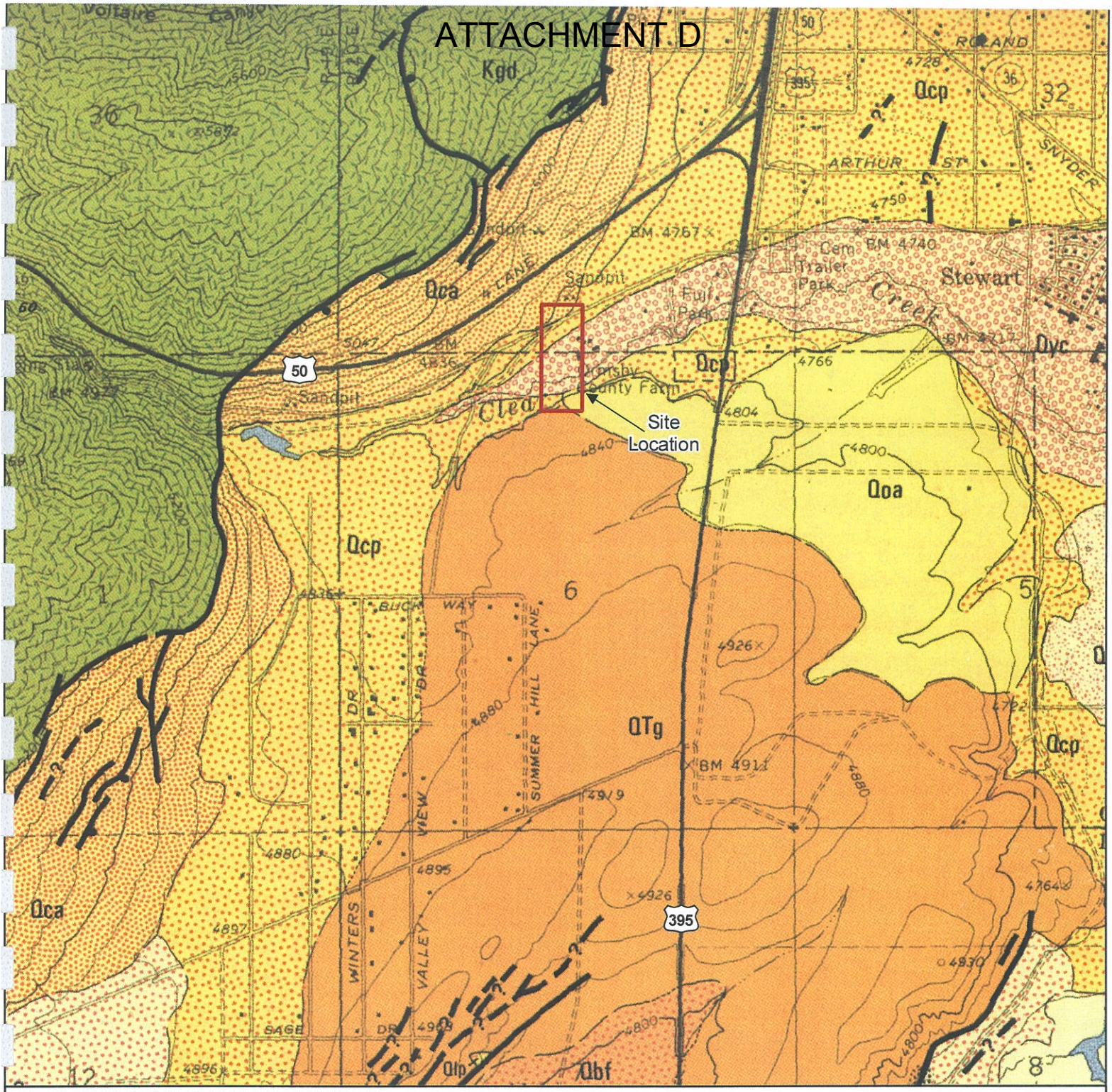
N

0 125 250 500 Feet

SOILS MAP FOR A PORTION OF:
COSTCO INTER-TIE/VISTA GRANDE WATER LINE IMPROVEMENT PROJECT
CARSON CITY AND DOUGLAS COUNTY, NEVADA

GEOCON

FIGURE 3
JUNE 2011
PROJECT NO.
R8757-06-01



GEOLOGIC UNITS IN THE SITE VICINITY

Qyc - Flood plain deposits of Clear Creek (Holocene)
Qca - Colluvial and alluvial deposits (Holocene)
Qcp - Alluvial plain deposits of Clear Creek (Holocene)
Qoa - old alluvium (Pleistocene)

Reference: Geonoa Quadarangle Geologic Map, Nevada Bureau of Mines and Geology, 1980.

QTg - Pediment deposits of Indian Hills (late Tertiary to early Pleistocene)
Kgd - Hornblende-biotite granodiorite (Jurassic)
Fault - Dashed where approximately located. Ball on down-dropped side.

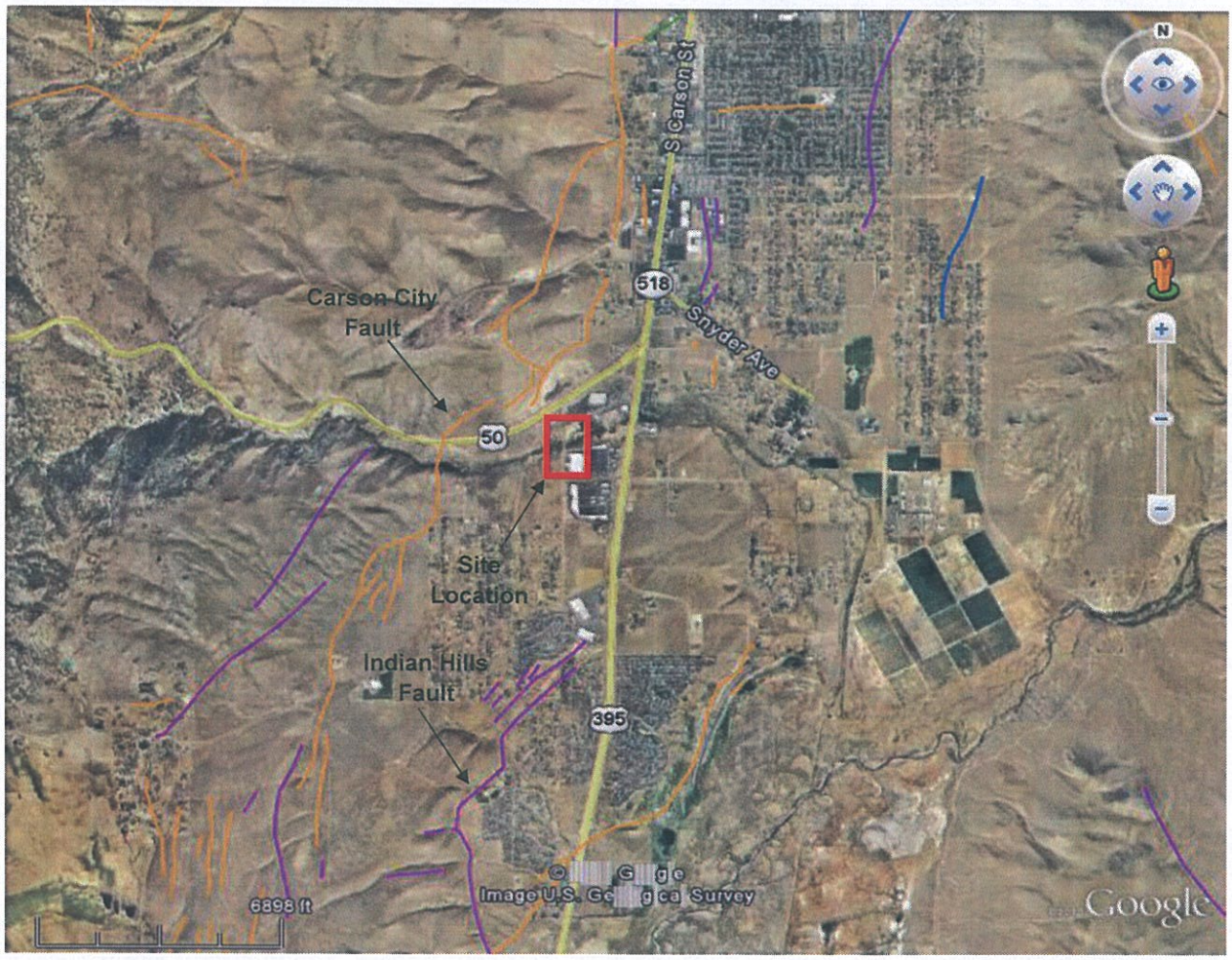
0 490 980 1,960 Feet

N

GEOLOGIC MAP FOR A PORTION OF:
COSTCO INTER-TIE/VISTA GRANDE WATER LINE IMPROVEMENT PROJECT
CARSON CITY AND DOUGLAS COUNTY, NEVADA

GEOCON

FIGURE 4
JUNE 2011
PROJECT NO.
R8757-06-01



AGE OF YOUNGEST FAULT DISPLACEMENT

- Historic (less than 150 years)
- Holocene (less than 15,000 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)

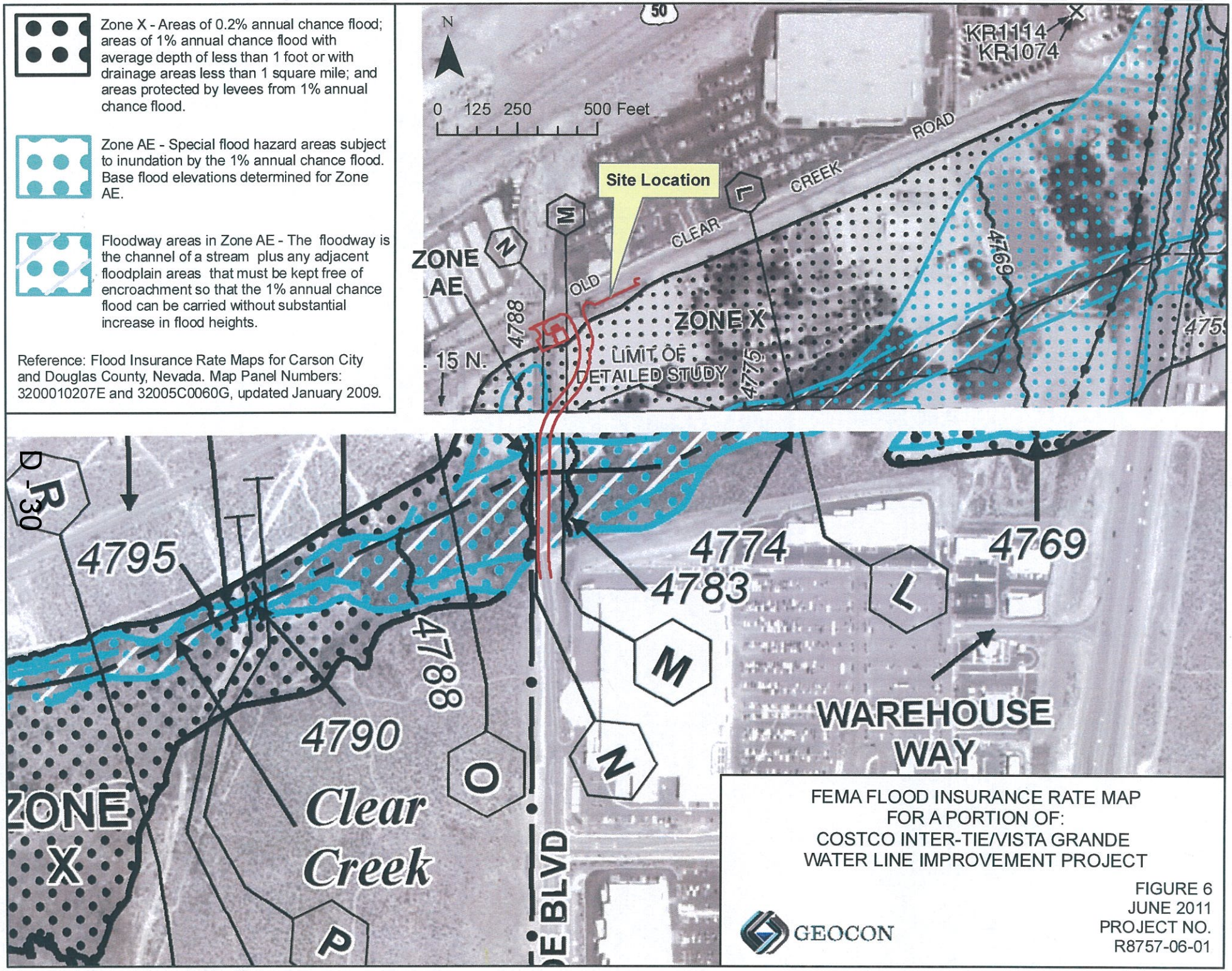
Map Reference:
U.S. Geological Survey and Nevada Bureau of Mines and Geology, 2008, Quaternary fault and fold database for the United States, May 2011, from USGS web site: http//earthquake.usgs.gov/regonal/qfaults/

FAULT MAP

GEOCON
CONSULTANTS, INC.
4010 TECHNOLOGY WAY - SUITE D - CARSON CITY, NV 89706
PHONE 775.888.9900 - FAX 775.888.9904

Costco Inter-Tie/Vista Grande
Water Line Improvement Project
Carson City and Douglas County, Nevada

R8757-06-01	June 2011	FIGURE 5



FEMA FLOOD INSURANCE RATE MAP
FOR A PORTION OF:
COSTCO INTER-TIE/VISTA GRANDE
WATER LINE IMPROVEMENT PROJECT

FIGURE 6
JUNE 2011
PROJECT NO.
R8757-06-01

GEOCON

ATTACHMENT D

APPENDIX A

ATTACHMENT D

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS					TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW		WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP		POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM		SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC		CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW		WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP		POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM		SILTY SANDS WITH OR WITHOUT GRAVEL
			SC		CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL		ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH		ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT		PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample
- Bulk Sample
- SPT Sample
- Modified California Sample
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

SAND AND GRAVEL			SILT AND CLAY			
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 2	0 - 3	0 - 0.25
LOOSE	5 - 10	7 - 16	SOFT	3 - 4	4 - 6	0.25 - 0.50
MEDIUM DENSE	11 - 30	17 - 48	FIRM	5 - 8	7 - 13	0.50 - 1.0
DENSE	31 - 50	49 - 79	STIFF	9 - 15	14 - 38	1.0 - 2.0
VERY DENSE	OVER 50	OVER 79	VERY STIFF	16 - 30	39 - 64	2.0 - 4.0
			HARD	OVER 30	OVER 64	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

MOISTURE DESCRIPTIONS

FIELD TEST	APPROX. DEGREE OF SATURATION, S (%)	DESCRIPTION
NO INDICATION OF MOISTURE; DRY TO THE TOUCH	S<25	DRY
SLIGHT INDICATION OF MOISTURE	25≤S<50	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	50≤S<75	MOIST
MINOR VISIBLE FREE WATER	75≤S<100	WET
VISIBLE FREE WATER	100	SATURATED

QUANTITY DESCRIPTIONS

APPROX. ESTIMATED PERCENT	DESCRIPTION
<5%	TRACE
5 - 10%	FEW
11 - 25%	LITTLE
26 - 50%	SOME
>50%	MOSTLY

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

CRITERIA	DESCRIPTION
PASS THROUGH A 3-INCH SIEVE AND BE RETAINED ON A NO. 4 SIEVE (#4 TO 3")	GRAVEL
PASS A 12-INCH SQUARE OPENING AND BE RETAINED ON A 3-INCH SIEVE (3"-12")	COBBLE
WILL NOT PASS A 12-INCH SQUARE OPENING (>12")	BOULDER

CEMENTATION/INDURATION DESCRIPTIONS

FIELD TEST	DESCRIPTION
CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE	WEAKLY CEMENTED/INDURATED
CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE	MODERATELY CEMENTED/INDURATED
WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE	STRONGLY CEMENTED/INDURATED

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
¼-INCH INDENTATIONS WITH SHARP END GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH ONE BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH COUPLE BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH MANY BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

FIELD TEST	DESCRIPTION
NO OBSERVED FRACTURES	UNFRACTURED/UNJOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1 TO 3 FOOT INTERVALS	SLIGHTLY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 4-INCH TO 1 FOOT INTERVALS	MODERATELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1-INCH TO 4-INCH INTERVALS WITH SCATTERED FRAGMENTED INTERVALS	INTENSELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT LESS THAN 1-INCH INTERVALS; MOSTLY RECOVERED AS CHIPS AND FRAGMENTS	VERY INTENSELY FRACTURED/JOINTED

BEDDING SPACING DESCRIPTIONS

THICKNESS/SPACING	DESCRIPTOR
GREATER THAN 10 FEET	MASSIVE
3 TO 10 FEET	VERY THICKLY BEDDED
1 TO 3 FEET	THICKLY BEDDED
3 ½-INCH TO 1 FOOT	MODERATELY BEDDED
1 ½-INCH TO 3 ½-INCH	THINLY BEDDED
½-INCH TO 1 ½-INCH	VERY THINLY BEDDED
LESS THAN ½-INCH	LAMINATED

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

DEGREE OF DECOMPOSITION	FIELD RECOGNITION	ENGINEERING PROPERTIES
SOIL	DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED	EASY TO DIG
COMPLETELY WEATHERED	DISCOLORED, CHANGED TO SOIL, FABRIC MAINLY PRESERVED	EXCAVATED BY HAND OR RIPPING (Saprolite)
HIGHLY WEATHERED	DISCOLORED, HIGHLY FRACTURED, FABRIC ALTERED AROUND FRACTURES	EXCAVATED BY HAND OR RIPPING, WITH SLIGHT DIFFICULTY
MODERATELY WEATHERED	DISCOLORED, FRACTURES, INTACT ROCK-NOTICEABLY WEAKER THAN FRESH ROCK	EXCAVATED WITH DIFFICULTY WITHOUT EXPLOSIVES
SLIGHTLY WEATHERED	MAY BE DISCOLORED, SOME FRACTURES, INTACT ROCK-NOT NOTICEABLY WEAKER THAN FRESH ROCK	REQUIRES EXPLOSIVES FOR EXCAVATION, WITH PERMEABLE JOINTS AND FRACTURES
FRESH	NO DISCOLORATION, OR LOSS OF STRENGTH	REQUIRES EXPLOSIVES

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST ¼-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN ¼-INCH THICK	LAMINATED
BREAKS ALONG DEFINITE PLANES OF FRACTURE WITH LITTLE RESISTANCE TO FRACTURING	FISSURED
FRACTURE PLANES APPEAR POLISHED OR GLOSSY, SOMETIMES STRIATED	SLICKENSIDED
COHESIVE SOIL THAT CAN BE BROKEN DOWN INTO SMALLER ANGULAR LUMPS WHICH RESIST FURTHER BREAKDOWN	BLOCKY
INCLUSION OF SMALL POCKETS OF DIFFERENT SOIL, SUCH AS SMALL LENSES OF SAND SCATTERED THROUGH A MASS OF CLAY	LENSED
SAME COLOR AND MATERIAL THROUGHOUT	HOMOGENOUS

COMMENTS:

LOG OF BORING No. BH-1

LOGGED BY: J. Koch DATE: 4/21/11

EQUIPMENT: CME 55

TOTAL DEPTH: 5.5 WATER DEPTH:

MISC. TESTS	R VALUE	UNIT DRY WEIGHT, PCF	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	PASSING NO. 200 SIEVE, %	DEPTH, FT	SYMBOL	SAMPLE	BLOWS PER FOOT	DESCRIPTION / CLASSIFICATION	LAYER ELEV./ DEPTH
											SURFACE ELEVATION:	
			8			20				67	ASPHALT- 3 inches thick	0.3
											AGGREGATE BASE - 9 inches thick-dark grayish brown, sl. moist	1.2
			9			22				43	SILTY SAND (SM)- Dense to medium dense, moist, reddish brown, w/ gravel	
			11			24	5			62		
												5.5

LOG OF BORING R8757-06-01 VISTA GRANDE.GPJ GEOCON NV.GDT 6/1/11

GEOCON CONSULTANTS, INC

PROJECT:

Costco Inter-tie/Vista Grande Water Line Project
Carson City-Douglas County, Nevada

JOB NO.: R8757-06-01 DATE: 6/1/11

Figure-1

COMMENTS:

LOG OF BORING No. BH-2

LOGGED BY: J. Koch DATE: 4/21/11

EQUIPMENT: CME 55

TOTAL DEPTH: 17.1 WATER DEPTH:

MISC. TESTS	R VALUE	UNIT DRY WEIGHT, PCF	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	PASSING NO. 200 SIEVE, %	DEPTH, FT	SYMBOL	SAMPLE	BLOWS PER FOOT	DESCRIPTION / CLASSIFICATION	LAYER ELEV./ DEPTH
											SURFACE ELEVATION:	
											ASPHALT- 3 inches thick	0.3
											AGGREGATE BASE - 8 inches thick-dark grayish brown, sl. moist	1.0
										64	ROAD EMBANKMENT FILL- Dense to very dense, moist, reddish brown to brown, silty SAND (SM) with gravel	
											2'- pH = 7.93 Resistivity = 4.02 ohm-cm Chloride = 41.5 ppm Sulfate = 0.4 ppm	
		108	8				5			73		
							10			50		
											SILTY SAND (SM)- Very dense, sl. moist, brown to grayish brown with gravel	10.5
		109	10				15			50	Refusal @ 17.1	
												17.0

LOG OF BORING R8757-06-01 VISTA GRANDE.GPJ GEOCON NV.GDT 6/1/11

GEOCON CONSULTANTS, INC

PROJECT:

Costco Inter-tie/Vista Grande Water Line Project
Carson City-Douglas County, Nevada

Figure-2

JOB NO.: R8757-06-01 DATE: 6/1/11

COMMENTS:

LOG OF BORING No. BH-3

LOGGED BY: J. Koch DATE: 4/21/11

EQUIPMENT: CME 55

TOTAL DEPTH: 20.1 WATER DEPTH:

MISC. TESTS	R VALUE	UNIT DRY WEIGHT, PCF	WATER CONTENT, %	LIQUID LIMIT, %	PLASTICITY INDEX, %	PASSING NO. 200 SIEVE, %	DEPTH, FT	SYMBOL	SAMPLE	BLOWS PER FOOT	DESCRIPTION / CLASSIFICATION	LAYER ELEV./ DEPTH
											SURFACE ELEVATION:	
											ASPHALT- 6 inches thick	
											AGGREGATE BASE - 8.5 inches thick-dark	0.5
										65	grayish brown, sl. moist	1.2
		100	6								SILTY SAND (SM)- Dense, sl. moist, reddish brown, sl. clayey 11 to 13 feet	
							5			48	2'- pH = 6.81 Resistivity = 7.5 ohm-cm Chloride = 15.9 ppm Sulfate = 10.7 ppm	
							10			49		
		102	13									
											POORLY GRADED-SILTY SAND (SP-SM)- Very dense, moist, reddish brown with gravel	13.0
							15				Refusal at 20.1 feet	
										50		
							20					20.1

LOG OF BORING R8757-06-01 VISTA GRANDE.GPJ GEOCON NV.GDT 6/1/11

GEOCON CONSULTANTS, INC

PROJECT:

Costco Inter-tie/Vista Grande Water Line Project
Carson City-Douglas County, Nevada

JOB NO.: R8757-06-01 DATE: 6/1/11

Figure-3

COMMENTS:

LOG OF BORING No. BH-4

LOGGED BY: J. Koch DATE: 4/21/11

EQUIPMENT: CME 55

TOTAL DEPTH: 11.5 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:	
										SILTY SAND (SM)- Loose to medium dense, moist, reddish brown	
										SILTY SAND (SM)-Dense, moist, grayish brown to dark brown	3.0
		105	8				5				
							10			SILTY SAND (SM)-Dense, moist, gray	7.5
											11.5

GEOCON CONSULTANTS, INC

PROJECT:

Costco Inter-tie/Vista Grande Water Line Project
Carson City-Douglas County, Nevada

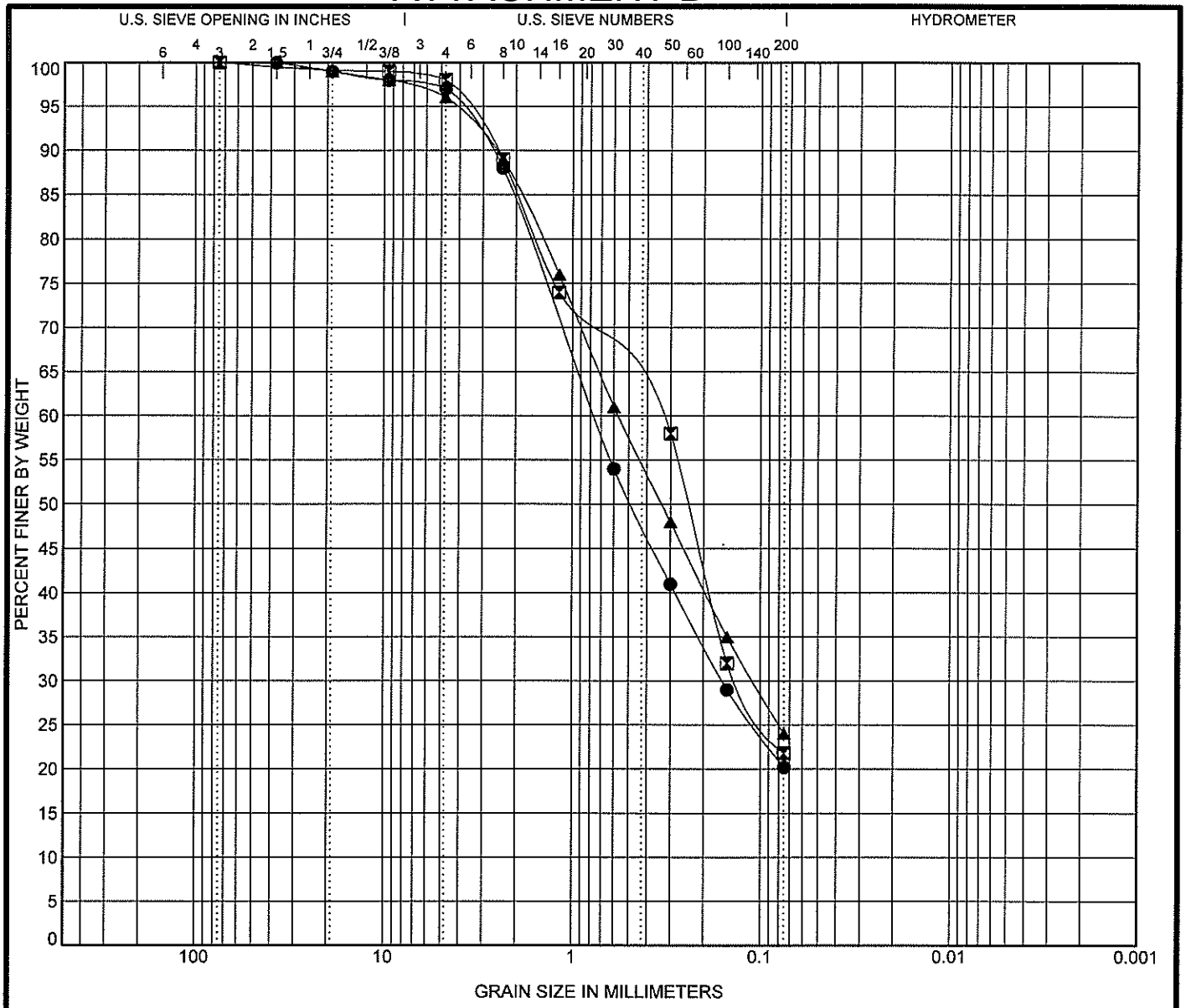
JOB NO.: R8757-06-01 DATE: 6/1/11

Figure-4

LOG OF BORING R8757-06-01 VISTA GRANDE.GPJ GEOCON NV.GDT 6/1/11

APPENDIX B

ATTACHMENT D



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	BH-1	1.0	Reddish Brown Silty Sand (SM)					
⊠	BH-1	2.7	Reddish Brown Silty Sand (SM) with gravel					
▲	BH-1	4.0	Brown Silty Sand (SM)					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	BH-1	1.0	37.5	0.764	0.159		3.0	76.8	20.2	
⊠	BH-1	2.7	75	0.356	0.131		2.0	76.2	21.8	
▲	BH-1	4.0	37.5	0.569	0.109		4.0	72.0	24.0	

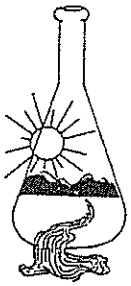
GEOCON CONSULTANTS, INC.

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

GRAIN SIZE DISTRIBUTION

Project: Costco Inter-tie/Vista Grande Water Line Project
Location: Carson City-Douglas County, Nevada
Number: R8757-06-01

US GRAIN SIZE R8757-06-01 VISTA GRANDE.GPJ GEOCON NV.GDT 6/1/11



Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/06/2011
Date Submitted 05/03/2011

To: Paul Oswald
Geocon
3160 Gold Valley Dr. #800
Rancho Cordova, CA 95742

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

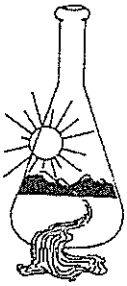
The reported analysis was requested for the following location:
Location : R8757-06-01/VISTA GR Site ID : BH-02 @ 1.5-2'.
Thank you for your business.

* For future reference to this analysis please use SUN # 60017-122668.

EVALUATION FOR SOIL CORROSION

Soil pH	7.93	
Minimum Resistivity	4.02 ohm-cm (x1000)	
Chloride	41.5 ppm	00.00415 %
Sulfate	0.4 ppm	00.00004 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422



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11353 Pyrites Way, Suite 4
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To: Paul Oswald
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3160 Gold Valley Dr. #800
Rancho Cordova, CA 95742

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : R8757-06-01/VISTA GR Site ID : BH-03 @ 1.5-2'.
Thank you for your business.

* For future reference to this analysis please use SUN # 60017-122669.

EVALUATION FOR SOIL CORROSION

Soil pH	6.81		
Minimum Resistivity	7.50 ohm-cm (x1000)		
Chloride	15.9 ppm	00.00159	%
Sulfate	10.7 ppm	00.00107	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

APPENDIX C

ATTACHMENT D

DETERMINATION OF EQUIVALENT AXLE LOAD
AND STRUCTURAL NUMBER

PROJECT: Vista Grande DATE: Jun-11
JOB #: R8757-06-01

LOCATION	Vista Grande
2011 ADT	925
GROWTH RATE (%)	4.00
DESIGN LIFE (YRS)	20
GROWTH FACTOR	29.778
TOTAL VEHICLES (DESIGN LIFE)	1.01E+07
PER CENT TRUCKS	100
DIRECTIONAL SPLIT (%)	50
DESIGN LANE (%)	100
TRUCKS IN DESIGN LANE	5026912

DETERMINE EQUIVALENT SINGLE AXLE LOAD (ESAL's)

	%	TRUCK FACTOR	EASL
2-AXLE 4-TIRE	96.35	0.0004	1937
BUSES	0.4	0.8422	16935
2-AXLE 6-TIRE	1.67	0.2352	19745
3-AXLE SINGLE UNIT	0.46	0.5400	12487
4-AXLE SEMI-TRUCK	0.25	0.5873	7381
5-AXLE SEMI-TRUCK	0.81	1.2698	51704
6-AXLE SEMI-TRUCK	0.04	1.3554	2725
5-AXLE MULTIPLE TRLR	0.008	2.0070	807
6-AXLE MULTIPLE TRLR	0.01	1.0754	541
7 AXLE MULTIPLE TRLR	0	1.6757	0
	99.998		114261
.---> DESIGN ESAL (1,000,000)			0.11

. DETERMINE STRUCTURAL NUMBER

RELIABILITY:	0.70	
STD DEVIATE(4.1):	-0.524	
STD DEVIATION(So):	0.45	
TERM SERVICEABILITY:	2.5	
CBR		
Rvalue		35
Subgrade Modulus,Mr		20580
----> REQUIRED Structural Number, SN		1.4
.---> CALCULATED ESAL (1,000,000)		0.11

PAVEMENT TYPICAL SECTION:

MATERIAL	COEF.	DEPTH	SN
Asphalt	0.35	4.0	1.4
Aggregate Base	0.10	8.0	0.8
Sub-base	0.07	0.0	0.0
_____			0.0

------> CALCULATED Struc Number,SN			2.2
			=========

GEOCON
CONSULTANTS, INC.
4010 TECHNOLOGY WAY – SUITE D – CARSON CITY, NV 89706
PHONE 775.888.9900 – FAX 775.888.9904

Project: Vista Grande BPS
Project No: R86757-06-01

Date: 6/1/2011

Bearing Capacity Calculation

Peck, Hanson, and Thornburn

Calculated By: G. Luce

Loading Variables and Capacities

Horizontal Load, H	0
Vertical Load, V	1
Ultimate Bearing Capacity, q_{ult}	
Allowable Bearing Capacity, q_{allow}	
Safety Factor, SF	3

Footing Properties-Continuous Footing

Width, B	1.5
Embed. Depth, D	2
Length, L	100

Soil Properties*

Unit Weight, γ_x	125
Unit Weight, γ_y	125
Phi Angle Φ	31
Cohesion, C	0
N_{gamma}	20
N_c	0
N_q	21
Surcharge, W	0

* Soil properties based on fill meeting Structural Fill requirements (Orange Book)

Notes: No surcharge loads or inclined loads considered.

Comments:

Bearing Capacity Equation

$$q_{ult} = [1/2\ B\gamma_y N\gamma\ (1- 0.3\ (B/L))\ (1\text{-}1.5\ (H/V))^2] + [CNc\ (1+0.2\ B/L)\ (1+0.2D/B)\ (1- 1.3\ H/V)] + [(w + D\gamma_x)\ (Nq - 1)\ (1 + 0.2\ B/L)\ (1 + 0.1\ D/B)\ (1 - 1.5\ H/V)] + (w + D\gamma_x)$$

Ultimate Bearing Capacity **7,800.2 psf**

Safety Factor **3 psf**

Allowable Bearing Capacity **2,600.1 psf**

Use 2,500 pounds per squarefoot

Components of PHT Equation

1st Term	1866.6
2nd Term	0.0
3rd Term	5683.7
4th Term	250.0
Total	**7,800.2 psf**

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Project: Vista Grande BPS
Project No: R86757-06-01

Date: 6/1/2011

Bearing Capacity Calculation

Peck, Hanson, and Thornburn

Calculated By: G. Luce

Loading Variables and Capacities

Horizontal Load, H	0
Vertical Load, V	1
Ultimate Bearing Capacity, q_{ult}	
Allowable Bearing Capacity, q_{allow}	
Safety Factor, SF	3

Footing Properties- Column Footing

Width, B	3
Embed. Depth, D	1.5
Length, L	3

Soil Properties*

Unit Weight, γ_x	125
Unit Weight, γ_y	125
Phi Angle Φ	31
Cohesion, C	0
N_{gamma}	20
N_c	0
N_q	21
Surcharge, W	0

* Soil properties based on fill meeting Structural Fill requirements (Orange Book)

Notes: No surcharge loads or inclined loads considered.

Comments:

Bearing Capacity Equation

$$q_{ult} = [1/2\, B\gamma_y N\gamma\, (1 - 0.3\,(B/L))\,(1 - 1.5\,(H/V))^2] + [CNc\,(1 + 0.2\,B/L)\,(1 + 0.2D/B)\,(1 - 1.3\,H/V)] + [(w + D\gamma_x)\,(Nq - 1)\,(1 + 0.2\,B/L)\,(1 + 0.1\,D/B)\,(1 - 1.5\,H/V)] + (w + D\gamma_x)$$

Ultimate Bearing Capacity **7,537.5 psf**

Safety Factor **3 psf**

Allowable Bearing Capacity **2,512.5 psf**

Use 2,500 pounds per squarefoot

Components of PHT Equation

1st Term	2625.0
2nd Term	0.0
3rd Term	4725.0
4th Term	187.5
Total	**7,537.5 psf**

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PHONE 775.888.9900 – FAX 775.888.9904

Project: Vista Grande BPS
Project No: R8757-06-01

Date: 6/1/2011

Run # 1

Settlement Computation For Cohesionless Soils FHWA Soils and Foundations

Considering 1.5 ft.Continuous Footing.

B=	1.5
D=	2
Q allow =	2,500

$$\Delta H = H \, 1/C' \log \frac{Po + \Delta P}{Po}$$

Thickness of soil Layer considered, H	3	Blow Count*	25
Bearing Capacity Index, C'	120	SPT Corr. Factor	2
Existing Overburden Pressure, psf, P_o	250	Corrected Blow Count	50
Distributed embankment Pressure, psf	1000		
Final Pressure, PF	1250	*Structural fill compacted to 90% min.	

Note: $PF = P_o + P$

$C' = \frac{1 + e_o}{1}$

Comments: Analysis is for structural fill supporting footing at least three feet below footing.

Settlement **0.017 ft. = 0.210 inches**

NOTES:

GEOCON
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Project: Vista Grande BPS
Project No: R8757-06-01

Date: 6/1/2011

Run # 1

Settlement Computation For Cohesionless Soils FHWA Soils and Foundations

Considering 3 ft. Wide Column Footing.

B=	3
D=	1.5
Q allow =	2,500

$$\Delta H = H \, 1/C' \log \frac{Po + \Delta P}{Po}$$

Thickness of soil Layer considered, H	3	Blow Count*	25
Bearing Capacity Index, C'	120	SPT Corr. Factor	2
Existing Overburden Pressure, psf, P_o	200	Corrected Blow Count	50
Distributed embankment Pressure, psf	1000		
Final Pressure, PF	1200	*Structural fill compacted to 90% min.	

Note: $PF = P_o + P$

$C' = \frac{1 + e_o}{1}$

Comments: Analysis is for structural fill supporting footing at least three feet below footing.

Settlement **0.019 ft. = 0.233 inches**

NOTES: