

Geotechnical Investigation

Project No.
7217.7.001.01

May 18, 2006

Mr. Bob Peterson
Landmark Communities
985 Damonte Ranch Parkway, Suite 300
Reno, NV 98521

Subject: State Street Project
Carson City, Nevada

GEOTECHNICAL REPORT

Dear Mr. Peterson:

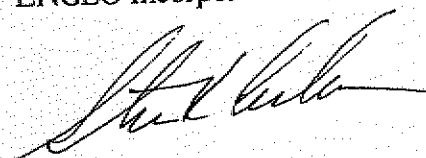
ENGEO Incorporated prepared this geotechnical report for the State Street Project as outlined in our agreement dated April 19, 2006. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicates that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated



Steve R. Crenshaw
Project Geologist

Reviewed by:

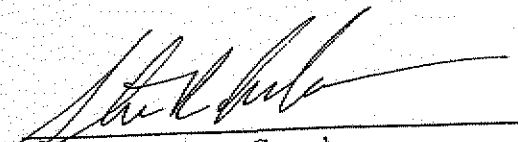


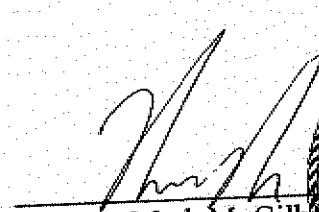
Mark M. Gilbert, PE
Principal

GEOTECHNICAL REPORT
STATE STREET PROJECT
CARSON CITY, NEVADA

SUBMITTED
TO
LANDMARK COMMUNITIES
RENO, NEVADA

PREPARED
BY
ENGEO INCORPORATED
PROJECT NO. 7217.7.001.01


Steve Crenshaw


Mark M. Gilbert

May 18, 2006
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1. INTRODUCTION

ENGEO Incorporated prepared this geotechnical report for design of the State Street Project in Carson City, Nevada. This report contains geotechnical recommendations for design of single-family residences on the 7.7-acre parcel of APN 002-441-23 and commercial buildings on the 2.4-acre parcel of APN 002-441-21.

For our use we received the following:

1. A revised Assessors Parcel Map of the site dated March 10, 2003 and a site map prepared by Capital Engineering, dated February 2006.
2. Lumos & Associates, Inc.; "Geotechnical Investigation and Fault Study for APN 2-440-19 & 20"; File No. 3847.000; dated June 3, 1997.
3. Trexler D.T., 1977, Carson City Folio Geologic Map, Environmental Series, Nevada Bureau of Mines and Geology, scale 1:24,000.
4. Bingler, E.C., 1977, New Empire Geologic Map, Nevada Bureau of Mines and Geology, scale 1:24,000.
5. United States Geological Survey, Earthquake Hazards Program, Complete Report for Carson City Fault (Class A), No. 1286, <http://qfaults.cr.usgs.gov/faults>.
6. United States Geological Survey, Earthquake Hazards Program, Quaternary Fault and Fold Database, <http://gldims.cr.usgs.gov/servlet.com.esri.esrimap>.
7. Price, J. G., 1998, Guidelines for Evaluation Potential Surface Fault Rupture/Land Subsidence Hazards in Nevada, Revision 1, Nevada Earthquake Safety Council (NESC).

1.1 SCOPE OF SERVICES

ENGEO prepared this report as outlined in our agreement dated April 19 2006 and subsequent amendments. Landmark Communities authorized ENGEO to conduct the proposed scope of services, which included the following:

- Geologic and Aerial Photo Review
- Subsurface Field Exploration and Fault Trenching
- Cone Penetration Testing

- Soil Laboratory Testing
- Data Analysis and Conclusions
- Report Preparation

1.2 PROJECT LOCATION

Figure 1 displays a Site Vicinity Map. The site is located north of Highway 50 (E. Williams Street) and west of State Street in Carson City, Nevada. Access to the site is by an undeveloped road along the southern boundary of APN 002-441-23 that enters from State Street.

Figure 2 shows site boundaries and our exploratory locations. The site is bounded in the south by Highway 50 (E. Williams Street) along the southwest parcel (APN 002-441-21) and by APNs 002-441-9, 10, 11, and 12. The northwestern portion of the site is bounded by a sloped bank measuring approximately 20 feet in height. The site is bounded to the north by residential apartments and to the east by State Street.

1.3 PROJECT DESCRIPTION

Based on our discussion with Landmark Communities and review of the referenced information, we understand that site improvements will consist of construction of:

1. Approximately 100 one- and two-story single-family houses;
2. Three single story commercial buildings with slab-on-grade foundations. These buildings range from approximately 4,000 to 12,000 square feet in plan and are to be located in the southwest corner of the site.;
3. Paved streets;
4. Underground utilities;
5. Retaining Walls;
6. Drainage Way;
7. Sidewalks; and,
8. Landscaping.

2. FINDINGS

2.1 FIELD EXPLORATION

We performed subsurface exploration at the site to characterize the geotechnical and geological conditions for design of the proposed development. We performed our exploration in three phases. The first phase included excavation of ten test pits on February 17, 2006 at the locations shown on the Site Plan, Figure 2. Our second phase of exploration included four cone penetrometer test (CPT) borings on March 28, 2006 to evaluate potential liquefaction concerns. The third phase of exploration included trench excavations performed between April 26 and 28, 2006 to explore the presence of a known active fault.

Section 2 presents descriptions of surface and subsurface conditions observed during our explorations.

2.2 SURFACE CONDITIONS

According to the topographic information provided, the site slopes gently to the southeast towards Highway 50 (E. Williams Street) from the sloped bank located along the northwestern boundary of the site. Site grades range from approximately Elevation 4660 feet (along the sloped bank) to Elevation 4650 feet mean sea level (msl) along the eastern boundary.

We performed a site reconnaissance on February 17, 2006 to identify prominent features including vegetation, roads, utilities, wells, drainage swales, waterways, stockpiled soil, existing structures, rock outcrops, desiccation cracks in clay, wet surface conditions or seepage. We observed the following site features during our reconnaissance:

- The site was covered by a moderate to dense growth of sage brush and grasses.
- We did not observe any surface drainages on the property.
- An electrical transmission line and an undeveloped dirt road are located along the southern portion of APN 002-441-23.
- An undeveloped dirt road or trail also exist in the middle of APN 002-441-23 and extends from the northeastern corner of the property to the western boundary.
- An existing single story wood frame business and vacant sheds are located along the southern portion of APN 002-441-21.
- A 20-foot high slope is located along the northwestern boundary of the site. This slope is identified by the United States Geological Survey (USGS) Earthquake Hazard Program as one of the Carson City faults.

2.3 SITE BACKGROUND

Previous to our site exploration, Lumos & Associates, Inc. conducted a geotechnical (pit) investigation and fault (trench) study for APN 2-440-19 & 20 (APN 2-440-20 later to be renamed APN 002-441-23). Their work consisted of six test pits to a maximum depth of 9½ feet for geotechnical purposes and three trenches completed along the fault scarp that exists between the two parcels. Four of the six test pits were excavated northwest of the fault and include TP-1, 3, 5, and 6. The remaining two test pits were excavated southeast of the fault and include TP-2 and TP-4. These test pits identify clayey-sands and silty-sands beneath the site. The trenches were excavated to an approximate depth of 7 feet and approximately 100 feet in length. The branch of the Carson City fault was identified in the trenches and cut across recent deposits.

2.4 SITE GEOLOGY

We present the following discussion of site geology based on our field reconnaissance, subsurface exploration, and review of the *Carson City Folio Geologic Map* (Trexler, D. T., Nevada Bureau of Mines and Geology, 1977) and *Map 59, New Empire Geologic Map* (E. C. Binger, Nevada Bureau of Mines and Geology, 1977).

The site is located along the western margin of the Basin and Range geomorphic province of the State of Nevada. The Basin and Range is characterized by its horst and grabben topography that is formed by the Earths extension. Normal and normal-listric faults are prominent in the region and bound the valleys that are separated by the ranges. Large volumes of sediments have been deposited in the valleys ranging from a few hundred feet to over 10,000 feet. The site is located within Eagle Valley which is reported to contain approximately 2,000 feet of sediment.

A portion of the Carson City Geologic Map is presented as Figure 3. The geologic map shows that the site overlies Quaternary Alluvial-plain deposits (Qal). These deposits are identified as being Holocene in age (<11,000 years). As described on the geologic map, these deposits are yellowish brown to gray in color, massive to poorly bedded, and moderately to well graded. They consist of fine silty-sand, sandy-silts, granular muddy coarse sand, and minor sandy gravels. In addition, the Carson City Geologic Map shows Quaternary Older Alluvial-plain deposits (Qoa) located west and northwest of the site and exist on the topographic high. The Qoa deposits are described as moderately sorted; sandy small cobble gravel; slight gravely sand; and sandy coarse silt; and weakly to moderately weathered. The geologic map shows that Qal and Qoa deposits are dissected by a Quaternary fault.

2.5 SEISMIC SETTING

The site lies within a seismically active region. According to a search using the software program EQFAULT Version 3.00b (Blake, 2000), the nearest active fault is the Genoa Fault, which is mapped approximately 4 miles west of the site. The Genoa Fault is considered capable of a moment magnitude earthquake of 6.9. The software also lists the Antelope Valley Fault, located 29 miles to the southeast, and the Honey Lake fault zone, located 59 miles to the north, which are capable of moment magnitudes of 6.7 and 7.3, respectively.

It should be noted that the East Carson City fault zone was not listed from the EQFAULT summary. The USGS Quaternary Fault and Fold Database for the United States was used for reference to this fault zone. The specific citation for this fault zone, "Fault number 1286, East Carson Valley fault zone, in Quaternary fault and fold database of the United States: U.S. Geological website, Adams, K.D., and Sawyer, T.L, compilers, 1998" states that this fault zone is part of the Sierra Nevada frontal fault system and extends from the California border to along the east side of Carson Valley to the Carson River. The site overlies this fault zone and one of its branches (Carson City fault No. 1286) is observed along the northwestern boundary of the site. This fault slip rate has been calculated at 0.02 millimeter per year and is classified as an active fault.

2.6 GEOTECHNICAL EXPLORATION

To evaluate the geotechnical conditions for design, we observed excavation of ten test pits to depths of 10 to 12 feet at the locations shown on the Site Plan, Figure 2. In general, the excavations encountered medium dense silty sand to depths between 3 to 7 feet below existing grade. The majority of the excavations contained this upper silty sand from 6 to 7 feet below existing grade. Beneath the silty sand, we observed a 2 to 8-foot thick layer of medium dense sand. In two of the excavations, a medium dense gravelly-sand was encountered at the bottom of the test pit.

Following our test pit exploration, we learned that the groundwater table in nearby wells was relatively shallow. Since the site lies in a seismically active region and we encountered sands on the site, we performed supplemental liquefaction exploration. On March 28, 2006, four CPTs were performed on the site, at the locations shown in Figure 2. These CPTs were advanced to refusal, which ranged from 27 to 40 feet below the ground surface. The CPT logs are provided in Appendix A. In general, the CPT logs indicate that the subsurface is composed of alluvial material ranging from sandy clay to gravel. Groundwater was not detected in the CPT borings.

Consult the Site Plan and test pit logs for specific soil and groundwater conditions at each exploration location. We include our exploration logs in Appendix A. The logs contain the soil/rock type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. Appendix A also provides additional exploratory information in the general notes to the logs.

2.7 FAULT EXPLORATION

Previous work conducted by Lumos & Associates, Inc. in 1997 (Reference 2) identifies the Carson City fault along the northwestern boundary of the site. Lumos & Associates, Inc. work related to the site included previous fault studies, air photographs, geotechnical reports for adjacent sites, and field exploration trenching. Three exploration trenches were excavated across the scarp associated with the fault. It was interpreted by Lumos and Associates that the fault displaced the youngest soil by approximately 4 feet. Lumos and Associates concluded that fault should be considered active (Holocene age). The Lumos & Associates, Inc. fault trench locations and the fault trenches completed by ENGEO in April 2006 are shown on Figure 2.

We performed additional fault trenches between April 26 and 28, 2006 to further delineate the location of the active fault located along the northwestern boundary of the site. Our exploration included two fault trenches across the scarp associated with the active fault. Trench locations are shown on Figure 2 and graphical displays of the fault trench logs are presented in Appendix A. We also show the location of the former trenched by Lumos.

In Trench 1, which is located in the western portion of the site, the fault zone was encountered from stations 0+25 to 0+40. Clay and sand beds are observed to be highly disturbed and show approximately 3½ feet of vertical displacement. The trend of the fault in Trench 1 was measured at N20E with a dip of 80 degrees to the south. In Trench 2, which is located in the northwest portion of the site, the fault zone was encountered from stations 0+00 to 0+30 and shows approximately 2 feet of vertical displacement. The trend of the fault in Trench 2 was measured at N40E with a dip of 78 degrees to the south. Young surficial deposits were observed to be deformed in both of the trenches, indicating that the fault is active. Outside the main fault rupture zone, we saw no evidence of secondary ground deformation such as filled soil fissures or deformation of soil layers.

During the trench exploration, the trench end points and fault zone locations were staked for future surveying. ENGEO correlated with Capital Engineering, Inc. out of Reno, Nevada, for surveying the fault trenches and fault zones. The trenches and fault zones were surveyed on May 5, 2006. An ENGEO representative accompanied the surveyor to show the locations of the trenches and fault zones. After the site was surveyed, Bill Ray of Capital Engineering, Inc provided us the surveyed coordinates that were used to plot the trench and fault zone locations on Figure 2.

2.8 GROUNDWATER CONDITIONS

The State of Nevada, Division of Water Resources web page was researched for area water levels. Water levels from wells located in the area range in depth between 10 and 42 feet below ground surface.

We did not observe static or perched groundwater in any of our test pits completed in February 2006. The Cone Penetration Tests (CPT) conducted in March 2006 also did not indicate any static or perched groundwater. However, perched groundwater was observed at a few isolated locations within the fault trenches completed in April 2006. Perched groundwater was observed along the fault zone and in isolated sand horizons.

In the previous geotechnical investigation conducted by Lumos & Associates in May 1997, they observed saturated soil at 8½ feet beneath the existing surface in there test pit #2. This test pit was located on the east side of the fault escarpment. As mentioned previously, groundwater was not encountered in the excavations conducted by ENGEO in February 2006. This may indicate that there are pockets of perched water in the area. This perched zone may have been the result of the fault.

Groundwater was not observed in February and March 2006, during the test pit explorations and CPT work, respectively, but perched groundwater was observed during our and Lumos & Associates fault trenching activities in April 2006 and in May 1997, This may indicate there are fluctuations in the level of groundwater that occurs due to variations in rainfall or other factors not evident at the time of original observations.

2.9 LABORATORY TESTING

We performed laboratory tests on selected soil samples to determine their engineering properties. Laboratory test data is presented in Appendix B.

3. CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development. The primary geotechnical concerns that could affect development on the site is the active fault located along the northwestern boundary of the property. We summarize our conclusions below.

3.1 ACTIVE FAULTING AND SURFACE RUPTURE

Based on the evidence observed in our trenches and review of the previous trenches logs, it is our opinion that the fault trace crossing the property should be considered active. According to State Guidelines (Price, 1998), habitable structures should not be constructed across active fault traces. Typically, building setbacks are recommended to mitigate potential damage from surface fault rupture hazards. Building setbacks are intended to minimize the risk of structural collapse or damage due to ground movement below building foundations by providing a separation between the building and the known surface fault projection. The setback width is typically sized to provide a reasonable separation between the known fault location and proposed structures. Factors that typically influence setback widths are the fault inclination and dip direction, the width of the zone of previous faulting, and any evidence of fault-related ground cracking beyond the fault zone. The trenches observed by ENGEO were extended well beyond the primary fault rupture zone. The exposed fault zone was observed to dip steeply to the south-southeast. No secondary cracking was observed outside the fault zone. Based on this information, it is our opinion that previous episodes of displacement along this fault have produced a relatively well-defined zone of surface rupture close to the main fault trace.

Based on our findings, we recommend that no habitable structures be constructed within 25 feet of the mapped fault trace. Non-habitable improvements, such as landscaping, fences, flatwork, or sheds, for example, may be constructed within 25 feet of the mapped fault trace.

Outside the 25-foot wide structural setback zone, it is our opinion that there may be potential for secondary ground movements that could affect building performance. To reduce the potential for building damage, we recommend a "reinforced building zone" that is 50 feet wide, extending from 25 to 75 feet from the fault trace. We recommend that any habitable structures that lie or encroach within this zone include the following special design recommendations:

1. Construct a reinforced fill zone beneath the building envelope and to 10 feet laterally beyond.
2. Support the house on a post-tensioned mat foundation system

We recommend that the reinforced fill zone include overexcavation of existing soil to a depth of 6 feet, and backfill and compaction with two layers of uniaxial geogrid. The overexcavation and geogrid is to be constructed parallel to the surveyed location of the fault zone. The reinforced fill zone should be constructed within the structure footprint plus 10 feet laterally and should follow the schematic shown on Figure 4. The grid should be oriented so that the high strength direction is perpendicular to the edge of the setback. As shown on Figure 2, lots 10 through 26 will require the improved building subgrade. Lot 9 may require the improved building subgrade if any structures are to be constructed in the northwest corner of the lot.

We provide recommendations for post-tensioned mat design in the "Foundation Recommendations" section of this report (Section 5). In areas outside of the reinforced building zone of 75 feet, an alternative shallow, reinforced continuous perimeter footings may be used where crawl spaces are desired. To mitigate the potential for foundation damage due to liquefaction, all interior footings should be structurally tied together with grade beams to create a grid-type foundation system.

3.2 EXISTING FILL

Although not observed in our test pits or fault trenches, non-engineered fill may exist in the southern portion of the property where the existing structures have been constructed. Non-engineered fill can undergo excessive settlement, especially under new fill or building loads. Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. We present detailed fill removal recommendations in Section 4.

3.3 EXPANSIVE SOIL

Based on our subsurface exploration, laboratory test results, and preliminary project data presented in Section 4, presence of native expansive soils is minimal and in our opinion should not affect the proposed development. In order to confirm our conclusions, we should be retained to review final grading and site improvement plans, and to observe and test engineered fill materials during site grading operations.

3.4 SECONDARY SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture mentioned in Section 3.1, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections. Based on topographic and lithologic data, the risk of regional subsidence or uplift is considered low to negligible at the site. Secondary seismic hazards related to water bodies such as flooding from tsunamis or seiches is considered to be non-existent.

Ground Shaking

The most significant seismic hazard at the proposed site is the secondary hazard of ground shaking. Earthquakes of moderate to high magnitude may occur near the subject site during the design life of the development, and these events may cause moderate ground shaking at the site that may affect the proposed structures.

To mitigate the ground shaking effects, all structures should be designed using sound engineering judgment and the latest International Building Code (IBC) requirements as a minimum. To provide seismic design parameters, we reviewed the 2003 IBC and the February 1998 California Divisions of Mines and Geology "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada".

Based on our review, we provide the 2003 IBC seismic parameters in Table 1 below:

TABLE 1
2003 International Building Code
Chapter 16

ITEM	DESIGN VALUE	IBC SOURCE
Site Class	D	Table 1615.1.1
0.2 second Spectral Response Acceleration, S_s	1.50	Figure 1615(3)
1.0 second Spectral Response Acceleration, S_1	0.60	Figure 1615(4)
Site Coefficient, F_a	1.0	Table 1615.1.2(1)
Site Coefficient, F_v	1.5	Table 1615.1.2(2)
Maximum considered earthquake spectral response accelerations for short periods, S_{MS}	1.5	Equation 16-38
Maximum considered earthquake spectral response accelerations for 1-second periods, S_{M1}	0.90	Equation 16-39
Design spectral response acceleration at short periods, S_{DS}	1.0	Equation 16-40
Design spectral response acceleration at 1-second periods, S_{D1}	0.6	Equation 16-41

Liquefaction

Liquefaction is a phenomenon in which saturated, cohesionless soil is subject to a temporary, but essentially total loss of shear strength because of pore pressure buildup under the reversing cyclic shear stresses associated with earthquakes.

Our liquefaction analysis suggests that static groundwater was not encountered to refusal, between 27 and 40 feet below ground surface from the CPT data. However, since groundwater has the potential to fluctuate overtime due to seasonal variation and/or local use of the aquifer may change, and that some perched groundwater zones were observed in the fault trenches at approximately 12 feet below ground surface, an evaluation was conducted for the site using the CPT data and a proposed groundwater level of 10 feet below ground surface. Using this data, a design-level earthquake that has a 10 percent probability of exceedance in 50 years, and a peak horizontal ground motion of 0.4g that is predicted to occur at the site during a design-level earthquake, our analysis of liquefaction-induced ground settlement suggests approximately 1 to 3 inches of potential settlement could occur. Capping effects of overlying non-liquefiable soil will tend to reduce the theoretical settlements, potentially reducing liquefaction-induced settlements to approximately 1 to 2 inches. Differential settlements may be on the order of one-half to two-thirds of the above values.

It is our opinion that this level of ground settlement could potentially induce minor damage to surface improvements. As a result, we provide mitigation recommendations to reduce potential liquefaction induced surface settlements to tolerable levels for the proposed improvements.

We recommend that all residential buildings be supported on post-tensioned mat foundations designed to withstand the additional potential differential caused by liquefaction induced settlement. We believe the post-tensioned mats will be suitable to reduce the effects of liquefaction induced ground settlement to tolerable levels. Alternatively shallow, reinforced continuous perimeter footings may be used where crawl spaces are desired. However, all interior footings should be structurally tied together with grade beams to create a grid-type foundation system to mitigate the potential for foundation damage due to liquefaction.

It is our opinion that commercial structures may be supported on conventional shallow footings provided a minimum of 3 feet of compacted, engineered fill is placed beneath all building pads. In addition, it may be desirable to utilize thicker slabs with heavier reinforcing, depending on the actual conditions encountered at each building. We anticipate that placement of a layer of engineered fill beneath structures will be suitable to reduce the effects of liquefaction induced ground settlement to tolerable levels.

Dynamic Densification Due to Earthquake Shaking

Densification of loose granular soils above the groundwater level can cause settlement due to earthquake-induced vibrations. The potential for dynamic densification at the site is expected to be low.

Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone that causes the overlying soil mass to move down a gentle slope or toward a free face such as a creek or open body of water. Lateral spreading is most often associated with strength loss due to liquefaction. As indicated above, there is minimal potential of liquefaction at the site. Topography is generally flat and settlement is considered to be less than 2 inches, thus the potential for lateral spreading at the site during seismic shaking is considered to be low, in our opinion.

Lurching

Ground lurching occurs as a result of the rolling motion imparted to the ground surface during energy released by an earthquake. The deformation of the ground surface by such rolling motion can cause ground cracks to form. The potential for the formation of these cracks is considered greater at contacts between material with significantly different properties, such as deep soft soil and bedrock. Such an occurrence is possible at the subject site as in other locations in the area, but in our opinion the offset or strain is expected to be minor.

3.5 SOIL CORROSION POTENTIAL

We submitted three soil samples to an analytical lab for determination of pH, chloride, resistivity, and sulfate. The pH results ranged from 7.10 to 7.52. The chloride results ranged from 3.1 to 6.0 mg/kg. The resistivity results for the samples ranged from 12,330 to 15,280 ohm-cm indicating that the samples tested were marginally corrosive. The sulfate results ranged from 0.1 to 0.7 mg/kg and are thus considered "Negligible" in accordance with Table 1904.3 of the 2003 International Building Code (IBC). For "Negligible" sulfate exposure, the IBC indicates that Type I-II Portland Cement should be used for concrete mix designs for the project.

If desired to investigate this further, we recommend a corrosion consultant be retained to determine if specific corrosion recommendations are necessary for the project. We present the analytical lab test results in Appendix B.

3.6 EXCAVATABILITY

We used a 4x4 Model 4-107 backhoe during our test pit exploratory work. Based upon our observation and experience, conventional grading and backhoe equipment will likely be able to excavate the soil deposits.

We provide the above excavatability information for general planning purposes only. This information is not intended for bidding purposes.

3.7 STATIC AND PERCHED GROUNDWATER

It does not appear that the groundwater level beneath the site will affect the proposed development. However, if perched groundwater develops, it can:

1. Impede grading activities;
2. Cause moisture damage to sensitive floor coverings;
3. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment; and
4. Cause premature pavement failure if hydrostatic pressures build up beneath the pavement section.

Recommendations to reduce the effects of perched water are presented in following sections addressing Over Optimum Soil Conditions, Site Drainage, Landscaping Considerations, Slab Moisture Vapor Transmission Reduction, and Cut-off Curbs.

4. EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil, rock, and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should not exhibit *flexing* or *pumping*, as determined by an ENGEO representative.

As used in this report, the term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define "structural areas" in Section 4 of this report as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

4.1 REINFORCED BUILDING ZONE OVEREXCAVATION

Within the 50-foot wide reinforced building subgrade zone discussed in Section 3.1, we recommend that the existing soil be overexcavated to a depth of 6 feet below finished grade and backfilled with compacted engineered fill reinforced with two layers of Tensar UX 1500 geogrid or equivalent. Place the geogrid at depths of 6 feet and at 4 feet below surface grade. The over excavation and geogrid is to be constructed parallel to the surveyed location of the fault zone and should underlay the entire building footprint. The improved building subgrade should be constructed within the structure footprint plus 10 feet laterally and should follow the schematic shown on Figure 4. Orient the grid so that the high strength direction is perpendicular to the direction of the fault. Compact backfill in accordance with Section 4.7.

4.2 COMMERCIAL BUILDING PAD OVEREXCAVATION

As mentioned previously, we recommend that commercial building pads be underlain by a minimum of 3 feet of engineered fill to mitigate minor liquefaction issues at the site. Overexcavation should extend a minimum of 10 feet laterally beyond the building footprint.

4.3 EXISTING FILL AND SOIL STOCKPILES

As mentioned previously, without proper documentation of the existing fill placed at the site, we recommend complete fill removal and recompaction. This includes the loosely backfilled test pits and trenches excavated for this geotechnical and fault exploration. Test pit and fault trench locations are presented on Figure 2.

Fill stockpiles may be reused as engineered fill provided the material is processed to conform to the recommendations in Section 4.5, Acceptable Fill. Resulting trash, building rubble, and other processed debris, if any are encountered should be removed from the site.

4.4 GENERAL SITE CLEARING

Clear areas to be developed of all surface and subsurface deleterious materials including existing building foundations, slabs, buried utility and irrigation lines, pavements, debris, shrubs, and associated roots, if any. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 4.6. ENGEO should observe and test all backfilling.

Following clearing, strip the site to remove any surface organic materials present. Strip organics from the ground surface to a depth of at least 2 to 3 inches below the surface. Remove strippings from the site or use them in landscape fill.

4.5 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during or following periods of rain or snow. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather;
2. Mixing with drier materials;
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated and approved by ENGEO prior to implementation.

4.6 FREEZING SOIL CONDITIONS

Freezing climatic conditions during winter grading may produce ice lenses that could result in soil heave. Areas subjected to frost induced heave during grading should be repaired before final grading or street subgrade work is completed.

4.7 ACCEPTABLE FILL

On-site soil and rock material is suitable as fill material provided it is processed to remove concentrations of organic material, building rubble, trash and other debris, and particles greater than 6 inches in maximum dimension. Unacceptable material should be removed from the site.

Imported fill materials should meet the above requirements and have a plasticity index less than 12. Allow ENGEO to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

4.8 COMPACTION

4.8.1 Grading in Structural Areas

Perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 8 inches;
2. Moisture condition soil to at least 1 percentage point above the optimum moisture content;
and
3. Compact the subgrade to at least 90 percent relative compaction. Compact the upper 6-inches of finish pavement subgrade to at least 95 percent relative compaction prior to aggregate base placement.

After the subgrade soil has been compacted, place and compact acceptable fill (defined in Section 4.5) as follows:

1. Spread fill in loose lifts that do not exceed 8 inches;
2. Moisture condition lifts to at least 1 percentage point above the optimum moisture content;
and
3. Compact fill to a minimum of 90 percent relative compaction; Compact the upper 6 inches of fill in pavement areas to 95 percent relative compaction prior to aggregate base placement.

Compact the pavement Type 2 Class B Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition the aggregate base at or slightly above the optimum moisture content prior to compaction.

4.8.2 Underground Utility Backfill

The contractor is responsible for conducting all trenching and shoring in accordance with Local, State, and OSHA requirements. Project consultants involved in utility design should specify pipe bedding materials.

Place and compact trench backfill in structural areas as follows:

1. Trench backfill should have a maximum particle size of 6 inches;
2. Moisture condition trench backfill to or slightly above the optimum moisture content. Moisture condition backfill outside the trench;
3. Place fill in loose lifts not exceeding 12 inches;
4. Compact fill to a minimum of 90 percent relative compaction (ASTM D1557).

Where utility trenches cross beneath buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. The plug should be constructed using a sand cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend for a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

4.8.3 Landscape Fill

Process, place and compact fill in accordance with Sections 4.5 and 4.6, except compact to at least 85 percent relative compaction (ASTM D1557).

4.9 SLOPE GRADIENTS

Construct final slope gradients to 2:1 (horizontal:vertical) or flatter. The contractor is responsible to construct temporary construction slopes in accordance with Local, State, and OSHA requirements.

4.10 SITE DRAINAGE

4.10.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we provide the following minimum recommendation for surface drainage.

1. Slope pavement areas a minimum of 1 percent towards drop inlets or other surface drainage features.
2. Slope finished grade away from building exteriors at a minimum of 2 percent for a distance of at least 5 feet.
3. Install roof gutters on all houses and connect into closed drain pipes which are led to drain outlets such as front yards or storm drains.

4.10.2 Subsurface Drainage

Based on our site exploration and current grading concepts for the site, we do not anticipate that subdrainage systems will be necessary. We will further evaluate the need for subdrainage systems following review of site grading plans and again during site grading.

We recommend that the landscape architect incorporate these items into the landscaping plans, and that we review the plans before construction.

5. RESIDENTIAL FOUNDATION RECOMMENDATIONS

We developed structural improvement recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. Where habitable structures are constructed within 25 to 75 feet of the fault line, we recommend that the proposed single-family houses be supported on post-tensioned mat foundations to mitigate seismic damage. Based on the development layout shown on Figure 2, this includes lots 9 through 26. Alternative foundations may be considered for structures at least 75 feet from the fault line. As an alternative, shallow, reinforced continuous perimeter footings may be used where crawl spaces are desired, provided that all interior footings are structurally tied together with grade beams to create a grid-type foundation system.

5.1 POST-TENSIONED MAT FOUNDATIONS

Design post tensioned concrete (PT) mat foundations in accordance with the 2003 International Building Code. We recommend that PT mats be at least 10 inches thick and have a thickened edge at least 2 inches greater than the mat thickness. The thickened edge should be at least 12 inches wide.

Design PT mats for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads, with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for all loads including wind or seismic. PT mats should be designed using the criteria presented below.

TABLE 2
Post-Tension Design Criteria

Condition	Center Lift	Edge Lift
Edge Moisture Variation (ft.)	5½	2½
Differential Soil Movement (in.)	1.3	1.5*

*The differential soil movement due to edge lift has been increased to account for frost heave.

The above design criteria are based on the procedure presented by the Post-Tensioning Institute (1996, 2nd Edition).

Moisture condition the pad subgrade to a moisture content at least 3 percentage points above optimum prior to foundation construction. The subgrade should not be allowed to dry prior to concrete placement.

When buildings are constructed with concrete slab-on-grade, such as post-tensioned mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Install a vapor retarder membrane directly beneath the PT mat. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs."

2. Concrete shall have a concrete water-cement ratio of no more than 0.45.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.
4. Moist cure slabs for a minimum of 3 days or use appropriate alternative curing techniques with approval of the project structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

5.2 CONTINUOUS FOOTINGS WITH CRAWLSPACES

If desired to construct the proposed single-family houses with crawlspaces, we recommend that reinforced continuous footings be used, provided that all interior footings are structurally tied together with grade beams to create a grid-type foundation system. All footings should bear on native soil or fill that is placed and compacted in accordance with the recommendations in this report.

5.2.1 Footing Dimensions and Allowable Bearing Capacity

Provide minimum footing dimensions as follows:

Table 3
Minimum Footing Dimensions

Footing Type	*Minimum Depth (in.)	Minimum Width (in.)
Continuous	18	12
Isolated	18	18

*below lowest adjacent pad grade (including frost effects)

Minimum footing depths shown above are taken from lowest adjacent grade. Isolated footings should be structurally tied to the perimeter continuous footings by reinforced concrete grade beams having a minimum width of 12 inches and minimum embedment of 12 inches.

Design foundations recommended above for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead plus live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

5.2.2 Reinforcement

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Reinforce all continuous footings and grade beams with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, design all continuous footings and grade beams to structurally span a clear distance of 10 feet, except as noted above for perimeter continuous footings with a reduced footing depth.

5.2.3 Foundation Lateral Resistance

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design:

Passive Lateral Pressure: 300 pcf
Coefficient of Friction: 0.35

The above allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading. Passive lateral pressure should not be used for footings on or above slopes.

5.2.4 Crawl Space Seepage Mitigation

Water accumulation within the crawl space beneath the proposed structure is possible even if adequate surface drainage is provided adjacent to the structure. Water seepage into the crawl space could substantially affect the foundation performance by allowing expansive soil to swell and heave footings. Crawl space seepage may create other undesirable impacts, such as rusting joist hangers, mildew, or excessive moisture rising through floors.

To mitigate crawl space water, we recommend reducing the potential for water to migrate into the crawl space and installing a drainage system within the crawl space to collect and discharge the water if it does migrate beneath the house. We recommend the following:

1. Construct a subsurface trench drain around the outside perimeter of the house, but no closer than 1½ feet from the foundation. The subdrain should be 1½ to 2 feet deep and at least 12 inches wide. The subdrain should generally be located along the uphill and two sides of the house; actual locations will need to be determined once the lot layouts are known. The subdrain should slope a minimum of 1 percent and discharge to a away from foundations. No vegetation should be placed within 2 feet of the subdrain.
2. Grade crawl spaces to drain to common low points; install area drains at low points to collect and discharge water through solid pipes.
3. Construct a series of shallow drainage channels (4 to 6 inches deep and 6 to 12 inches wide) around the perimeter of the crawl space. These channels should also drain toward a common low point; install area drains or sump pumps at low points to collect and discharge water.
4. Install adequate crawl space ventilation to help drying of wet or moist soil.

We recommend that we review the finished grading and landscaping plans to check for conformance with the above recommendations.

6. COMMERCIAL FOUNDATION RECOMMENDATIONS

Three commercial buildings are proposed within the southwest portion of the site. Since these building footprints do not encroach within 75 feet of the fault trace, we recommend that they be supported on conventional perimeter continuous and isolated interior footings bearing in engineered fill. The building pads shall be overexcavated in accordance with Section 4.2 to mitigate potential liquefaction issues. We provide the following recommendations for footing design:

6.1 FOOTING DIMENSIONS AND ALLOWABLE BEARING CAPACITY

Provide minimum footing dimensions as follows:

Table 3
Minimum Footing Dimensions

Footing Type	*Minimum Depth (in.)	Minimum Width (in.)
Continuous	18	12
Isolated	18	18

*below lowest adjacent pad grade (including frost effects)

Minimum footing depths shown above are taken from lowest adjacent grade. Isolated footings should be structurally tied to the building slab.

Design foundations recommended above for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead plus live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

6.2 REINFORCEMENT

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Reinforce all continuous footings with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, design all continuous footings to structurally span a clear distance of 10 feet.

6.3 FOUNDATION LATERAL RESISTANCE

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design:

Passive Lateral Pressure: 300 pcf
Coefficient of Friction: 0.35

The above allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading. Passive lateral pressure should not be used for footings on or above slopes.

7. SLABS-ON-GRADE

7.1 INTERIOR CONCRETE FLOOR SLABS

We recommend the following minimum design for the proposed commercial building floor slabs:

- Place minimum No. 3 rebar on 18-inch centers within the middle third of the slab to help control the width of shrinkage crack, which inherently occur as concrete cures.

- Provide a minimum concrete thickness of 5 inches.

The structural engineer should provide final design thickness and additional reinforcement, as necessary, for the intended structural loads.

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following:
 - a) Impermeable vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs." The vapor retarder should be **underlain by**
 - b) 4 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve
2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.
4. Moist cure slabs for a minimum of 3 days.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

7.2 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum concrete flatwork thickness of 5 inches. To reduce frost heave effects, provide a minimum of 6 inches of clean, well-graded aggregate base beneath flatwork. Compact aggregate base to at least 90 percent relative maximum density (ASTM D1557) at or slightly above the optimum moisture content.

In addition to the above, place wire mesh or rebar within the middle third of the flatwork to help control the width and offset of cracks. Use dowels at all construction joints and install frequent joints. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

8. RETAINING WALLS

8.1 FOUNDATIONS

Proposed residential retaining walls may be supported on continuous footings with a minimum width of 18 inches and a minimum depth of 18 inches below lowest adjacent soil grade.

Design retaining wall foundations for a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus live loads, for footings bearing in drained soil conditions. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

Lateral foundation loads may be resisted by friction along the base and by passive pressure along the sides of footing and toe of the wall, as appropriate. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend an allowable passive lateral pressure of 300 pcf, and coefficient of sliding friction of 0.35. These allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading.

The allowable passive lateral pressure must be reduced for retaining wall footings on or above slopes.

8.2 LATERAL SOIL PRESSURES

Design retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that drainage is included as recommended below, design unrestrained retaining walls with adequate drainage to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressure assumes level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater or artificial lake level. If drainage cannot be provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Furthermore, design all retaining walls for a seismic lateral force of $7 \cdot H^2$ in pounds per foot of retaining wall, where H^2 is the squared height of the wall. The seismic lateral force shall be applied at 0.6 times the height of the wall, from the base of the wall.

Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining walls.

8.3 RETAINING WALL DRAINAGE

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. We recommend that rock drains consist of a minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the $\frac{3}{4}$ -inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a nonwoven geotextile filter fabric such as Mirafi 140NC, or equivalent.

For rock drains:

1. Place the rock drain directly behind the walls of the structure.
2. Extend rock drains from the wall base to within 12 inches of the top of the wall.
3. Place a minimum of 4-inch-diameter perforated pipe at the base of the wall, inside the rock drain and fabric, with perforations placed down.
4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

8.4 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 4.6. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

9. PAVEMENT DESIGN

To provide data for pavement design, we performed two R-value tests on representative soil samples that we anticipated will be present in the roadway subgrade after grading.

The roadways may be designed for R = 40 subgrade provided that clayey subgrade is appropriately mitigated where encountered during roadway construction. Clayey subgrade areas will need to be subexcavated and replaced with minimum R = 40 material. Subexcavations would need to extend to a depth of at least 18 inches and 12 inches for Collector and Local roadways, respectively. This approach will likely require additional geotechnical observation and R-value testing during construction.

To date, a traffic study has not been performed for this project; therefore pavement design for the project will be based on the requirements of the Carson City Development Standards, Transportation Division 12. We present estimated cumulative Equivalent Single Axle Loads (ESAL's) below for an expected 20 year design life.

TABLE 5
Estimated Cumulative Equivalent Single Axle Loads (ESAL's)

Roadway Classification	Maximum Average Daily Traffic (ADT)	Percent Heavy Trucks*	Average Truck ESAL Factor*	ESAL's
Two Lane Collector (Without Residential Driveway Access)	8,000	5	0.5	7.3×10^5
Two Lane Collector, (With Residential Driveway Access)	2,000	3	0.5	1.1×10^5
Two Lane Local	1,000	3	0.5	5.5×10^4

*Estimated value

Based on the estimated ESAL's calculated above, we developed pavement recommendations using the 1981 Asphalt Institute Asphalt Pavement for Highways & Street Manual Series No. 1 (MS-1) recommended by Carson City Development Standards. We assumed 75 minimum percentile design value since the estimated ESAL values are between 10,000 and 1,000,000, a standard deviation in the estimated traffic frequencies of 0.4, and a change in the present serviceability index of 2.0, for all roadways.

TABLE 6
Asphalt Concrete Pavement Sections

Proposed Roadway	Sandy Subgrade Minimum R = 40	
	Asphalt Concrete (in.)	Class B Aggregate Base (in.)
Two Lane Collector (Without Residential Driveway Access)	4	8*
Two Lane Collector, (With Residential Driveway Access)	4*	8*
Two Lane Local	3*	6*

*Carson City minimum pavement sections govern the thickness of asphalt concrete and aggregate base.

9.1 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and Type 2 Class B Aggregate Base in accordance with Section 4.6.

9.2 PAVEMENT MAINTENANCE AND REHABILITATION

Due to variability in environmental conditions, frost and ice heave effects, thermal cracking, traffic conditions, construction quality, and pavement materials, routine maintenance and/or rehabilitation of the pavement may become necessary during the pavement design life. Such periodic maintenance may include crack sealing, seal coats, and patching, as necessary. Rehabilitation may include structural overlay or reconstruction, as necessary.

9.3 RESIDENTIAL DRIVEWAYS/GARAGE SLABS

We were not retained to provide design recommendations for residential driveways or garage slabs. They should be designed to resist the anticipated traffic and structural loads.

10. RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to provide construction monitoring services as outlined below:

1. Retain ENGEO to review the final grading and foundation plans prior to construction to determine whether our recommendations have been implemented, and to provide additional or modified recommendations, if necessary.
2. Retain ENGEO to perform construction monitoring to check the validity of the assumptions we made to prepare this report. Our services would include testing and observation during site clearing, mass grading, foundation excavation, underground utility construction, and pavement subgrade and aggregate base compaction.
3. If any changes occur in the nature, design or location of the proposed improvements, then retain ENGEO to review the changes and prepare a written response and validate the conclusions and recommendations in this report.
4. If 2 years or more lapse between the time this report was prepared and construction, or if conditions have changed because of natural causes or construction operations on or near the site, then retain ENGEO to review this report for applicability to the new conditions. This report is applicable only for the project and site studied.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

11. LIMITATIONS

This report presents geotechnical recommendations for construction of improvements discussed in Section 1.3 for the State Street Development project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any.

We strive to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the project area; no warranty is expressed or implied.

We developed this report with limited subsurface exploration data. We assume that our exploration data is representative of subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

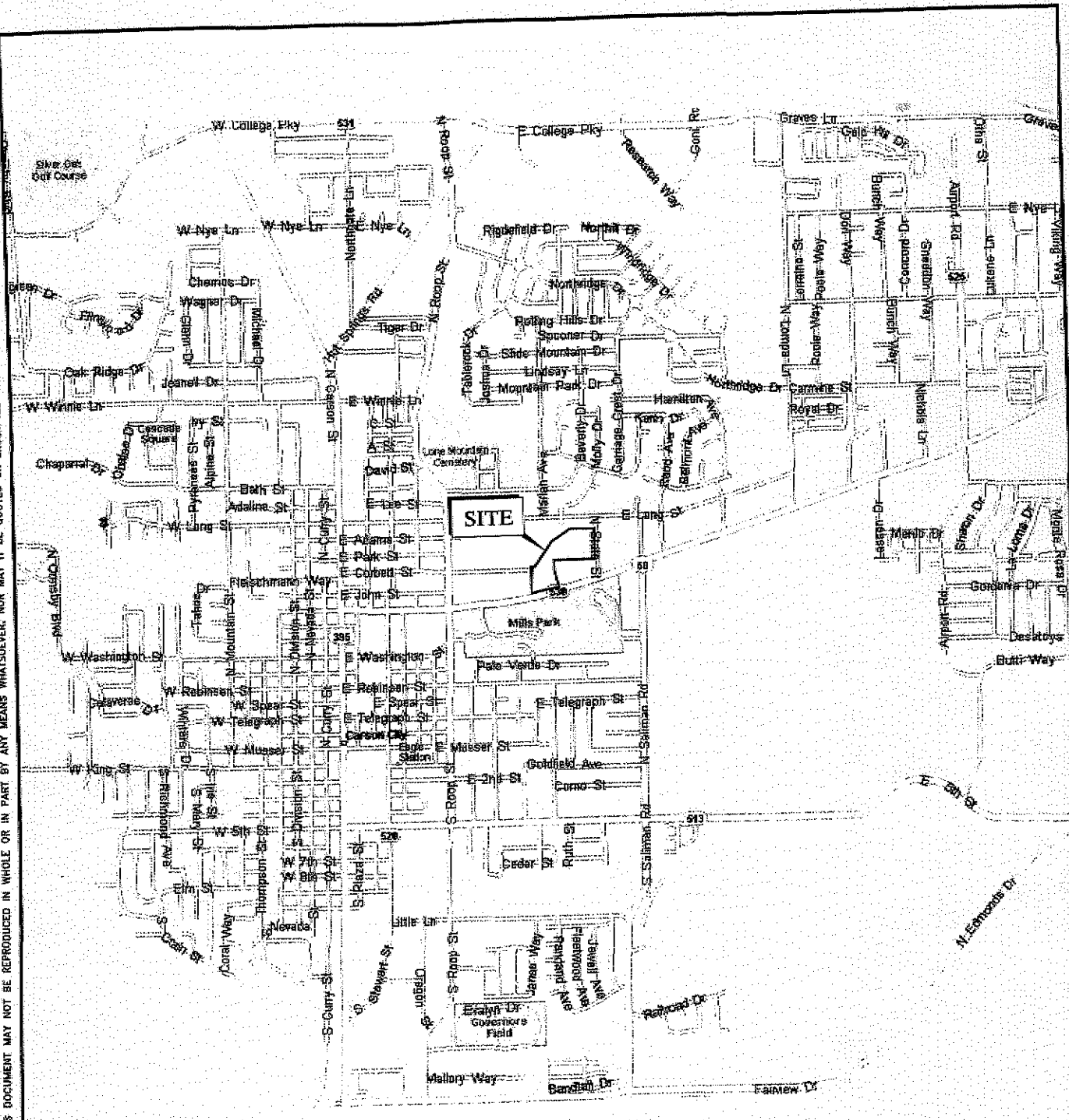
The location and elevations of our test pits are approximate and were estimated by pacing from features shown on the Site Map prepared by Capital Engineering dated February 2006.

The locations of our fault trenches and fault zones were surveyed by a representative of Capital Engineering on May 5, 2006.

Our services did not include excavation sloping or shoring, soil volume change factors, or a flood potential.

This geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, then notify the proper regulatory officials immediately.

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BASE MAP SOURCE: MS STREETS AND TRIPS

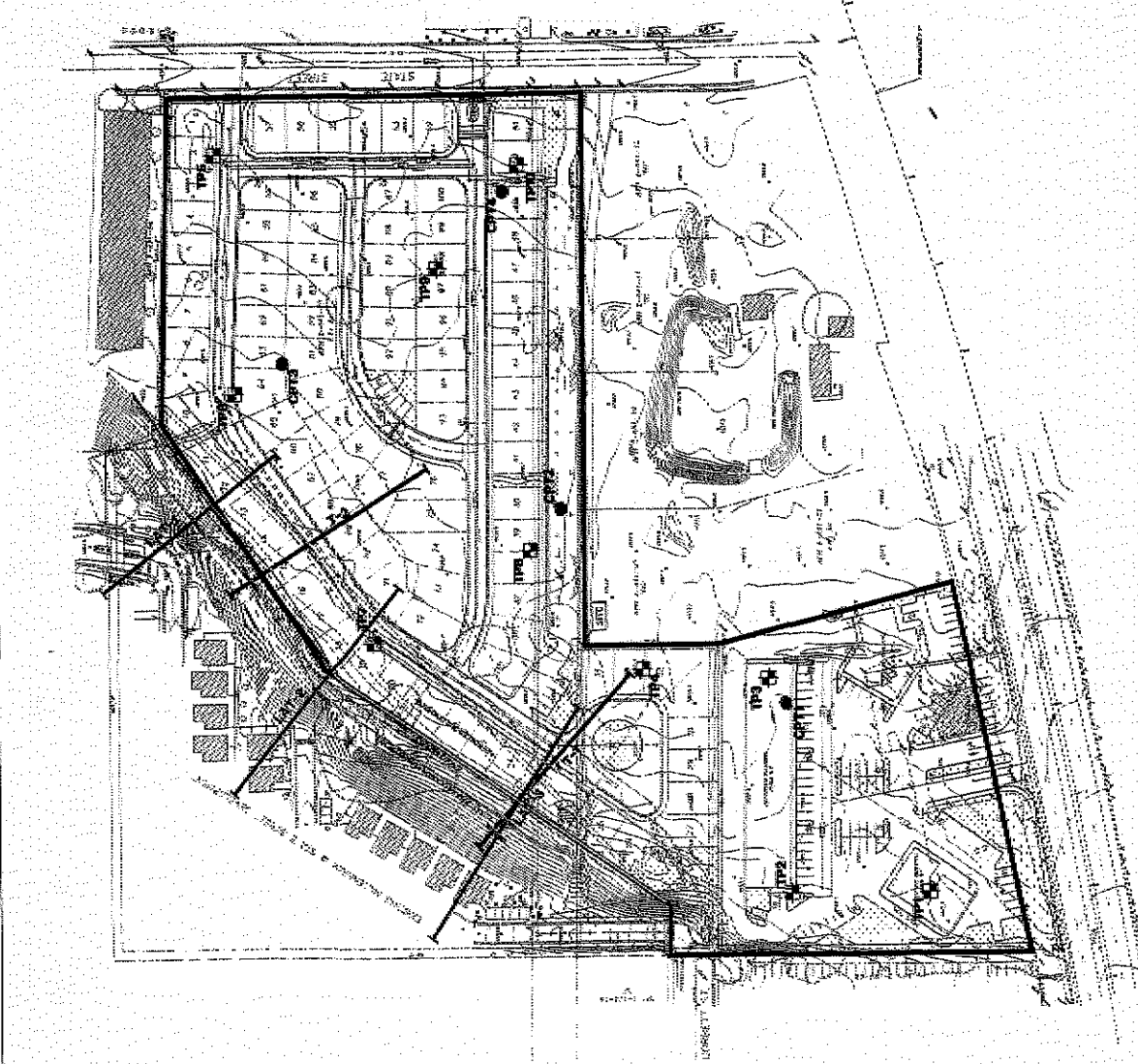


VICINITY MAP
STATE STREET
CARSON CITY, NEVADA

PROJECT NO.: 7217.7.001.01
DATE: MAY 2006
DRAWN BY: DLB CHECKED BY: MG

FIGURE NO.
1

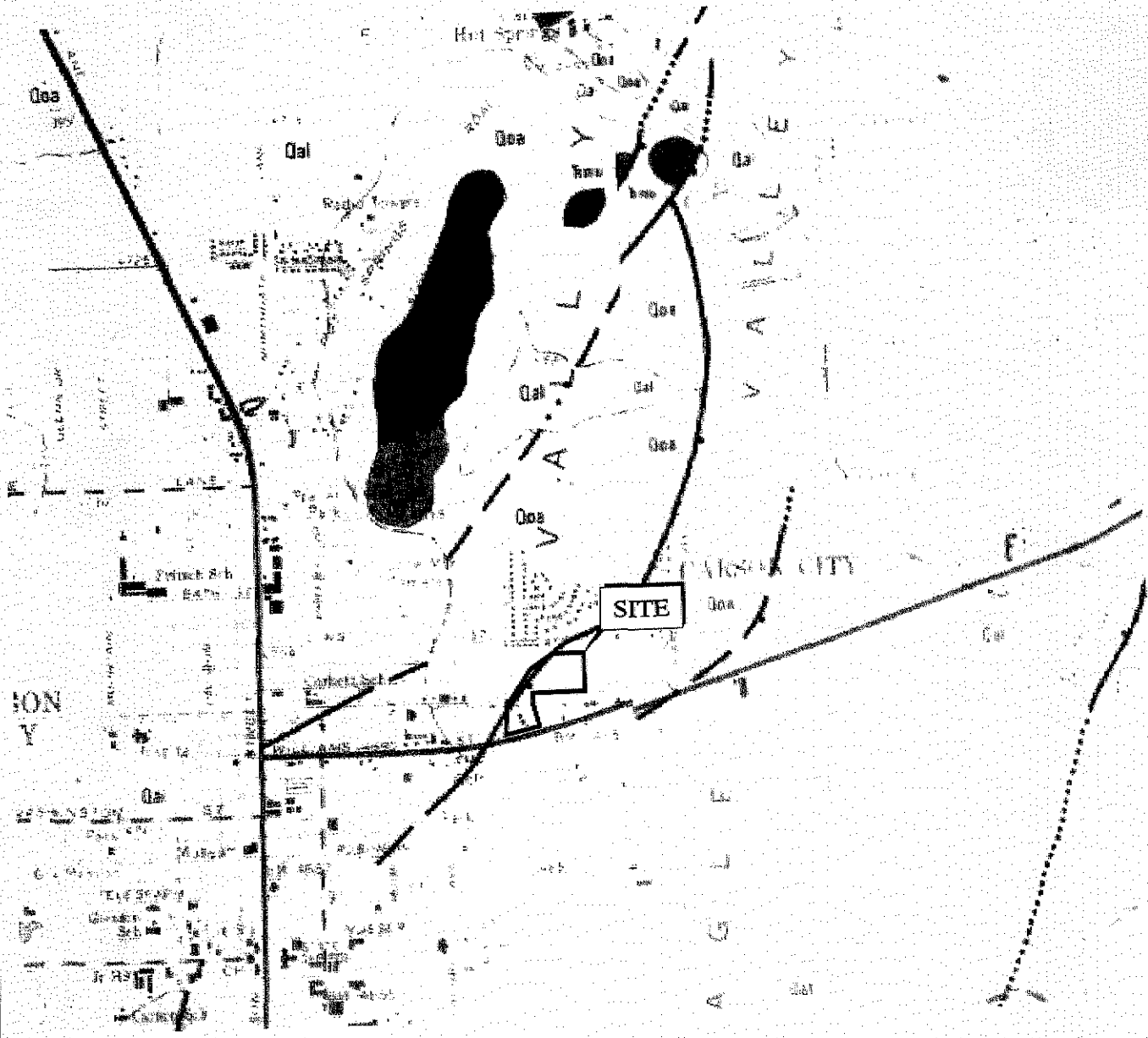
ORIGINAL FIGURE PRINTED IN COLOR



SCALE: 1/4" = 1'-0"
 APPROVED FOR THE CITY OF LOS ANGELES
 APPROVED FOR THE COUNTY OF LOS ANGELES
 APPROVED FOR THE STATE OF CALIFORNIA
 APPROVED FOR THE FEDERAL GOVERNMENT



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EXPLANATION

- Qal ALLUVIAL PLAIN DEPOSITS
- Qoa OLDER ALLUVIAL PLAIN DEPOSITS
- Trfs FELSIC SCHIST, UNDIFFERENTIATED
- Jd DECITE PORPHYRY
- FAULT - LONG DASHES WHERE INFERRED OR APPROXIMATELY LOCATED, DOTTED WHERE CONCEALED



BASE MAP SOURCE: BINGLER, 1977



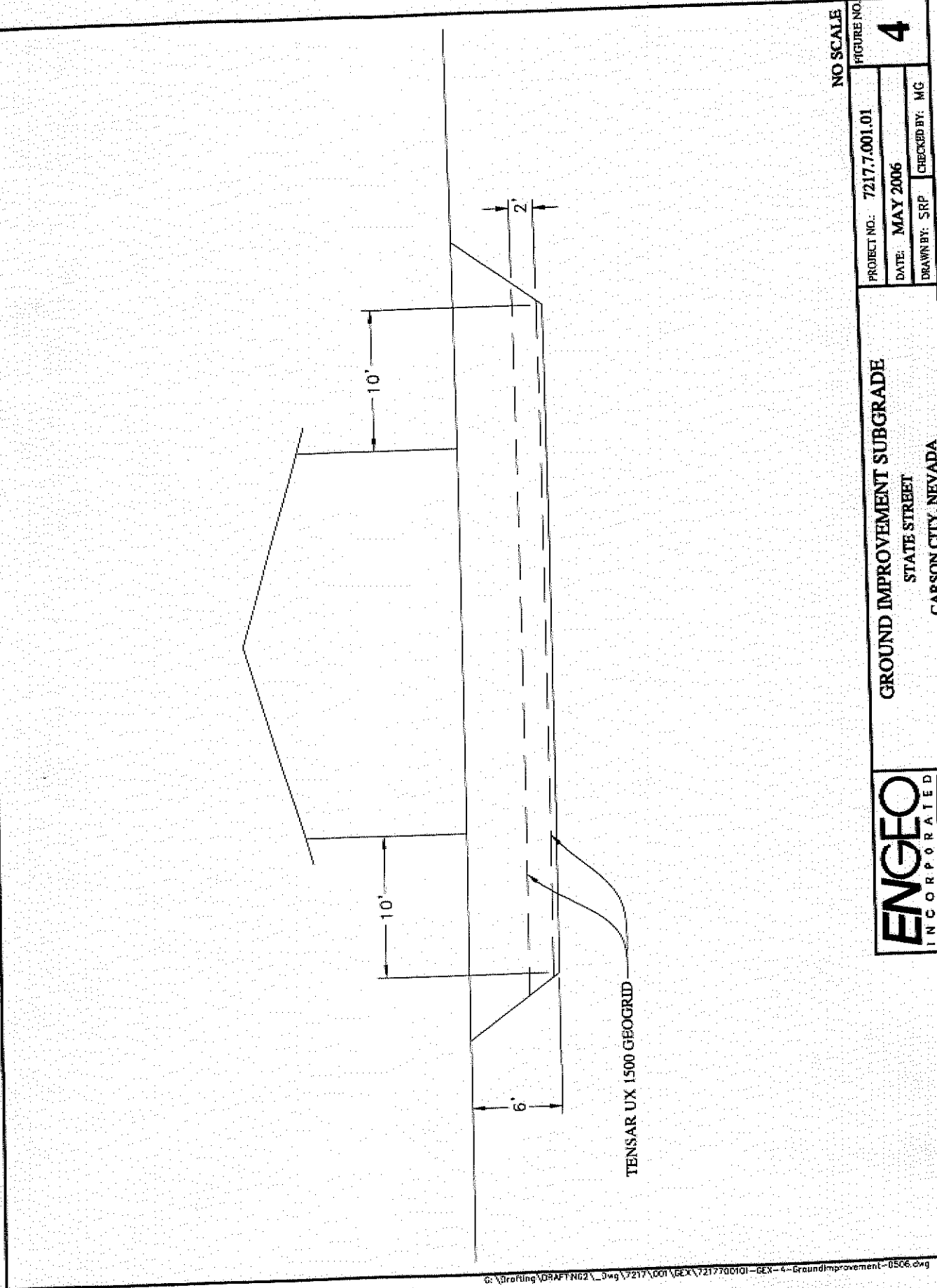
SITE GEOLOGIC MAP
STATE STREET
CARSON CITY, CALIFORNIA

PROJECT NO.: 7217.7.001.01
DATE: MAY 2006
DRAWN BY: SRP CHECKED BY: MG

FIGURE NO.
3

ORIGINAL FIGURE PRINTED IN COLOR

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NO SCALE
FIGURE NO. 4

PROJECT NO.: 7217.7.001.01
DATE: MAY 2006
DRAWN BY: SRP CHECKED BY: MC

GROUND IMPROVEMENT SUBGRADE
STATE STREET
CARSON CITY, NEVADA



APPENDIX A

**Field Exploration Notes
Key to Boring Logs
Exploratory Logs
CPT Logs
Fault Trench Logs**

FIELD EXPLORATION NOTES

We evaluated data gathered from 5 trenches (3 completed by Lumos and Associates) and 10 test pits on the site for this report. An ENGEO representative supervised the recent trenching and test pits, and logged the type, location, and uniformity of the underlying soil/rock. A model 4-107 backhoe was used to excavate the test pits using a 2 foot wide bucket.

The exploratory trench logs present descriptions and graphically depict the subsurface soil, rock and groundwater conditions encountered. The maximum depth penetrated by the trenches and test pits was 15 feet.

We obtained bulk soil samples from the test pits using hand sampling techniques.

We also evaluated data gathered from the four cone penetration tests conducted on the site for this report. An ENGEO representative supervised the boring activities which were bored to a maximum depth to refusal of 40 feet below ground surface.

NOTES TO THE LOGS

We determined the lines designating the interface between soil/rock materials on the logs using visual observations. The transition between the materials may be abrupt or gradual.

The logs contain information concerning samples recovered, indications of the presence of various materials such as sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the soil/rock conditions between samples. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs. The final logs represent our interpretation of the contents of the field logs.

KEY TO BORING LOGS

MAJOR TYPES		DESCRIPTION	
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES 	
		GRAVELS WITH OVER 12 % FINES 	
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES 	
		SANDS WITH OVER 12 % FINES 	
	FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS	
		SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %	
HIGHLY ORGANIC SOILS			
		ML - Inorganic silt with low to medium plasticity	
		CL - Inorganic clay with low to medium plasticity	
		OL - Low plasticity organic silts and clays	
	MH - Inorganic silt with high plasticity		
	CH - Inorganic clay with high plasticity		
	OH - Highly plastic organic silts and clays		
	PT - Peat and other highly organic soils		

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE				CLEAR SQUARE SIEVE OPENINGS				
		200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			

RELATIVE DENSITY		SILTS AND CLAYS		CONSISTENCY	
		STRENGTH*		BLOWS/FOOT (S.P.T.)	
SANDS AND GRAVELS		BLOWS/FOOT (S.P.T.)		BLOWS/FOOT (S.P.T.)	
VERY LOOSE	0-4	VERY SOFT	0-1/4	0-2	
LOOSE	4-10	SOFT	1/4-1/2	2-4	
MEDIUM DENSE	10-30	MEDIUM STIFF	1/2-1	4-8	
DENSE	30-50	STIFF	1-2	8-15	
VERY DENSE	OVER 50	VERY STIFF	2-4	15-30	
		HARD	OVER 4	OVER 30	

MOISTURE CONDITION

DRY	Absence of moisture, dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater
SATURATED	Below the water table

MINOR CONSTITUENT QUANTITIES (BY WEIGHT)

TRACE	Particles are present, but estimated to the less than 5%
SOME	5 to 15%
WITH	15 to 30%
.....Y	30 to 50%

- ### SAMPLER SYMBOLS
- Modified California (3" O.D.) sampler
 - California (2.5" O.D.) sampler
 - S.P.T. - Split spoon sampler
 - Shelby Tube
 - Continuous Core
 - Bag Samples
 - Grab Samples
 - NR No Recovery

- ### LINE TYPES
- Solid - Layer Break
 - Dashed - Gradational or approximate layer break

- ### GROUND-WATER SYMBOLS
- Groundwater level during drilling
 - Stabilized groundwater level



(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler
 * Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

State Street Project GEX
Carson City, NV

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP1	0 - 6 ½	SILTY SAND (SM), with some clay, yellowish brown, loose, moist, fine- to medium-grained sand. Sample taken at 2 feet. At 3 ½ to 4 ½ feet, partial cementation / calcification. Sample taken at 4 feet. Sample taken at 6 feet.
	6 ½ - 8 ½	SAND (SP), yellowish brown, medium dense, moist, fine- to medium-grained sand, traces of gravels.
	8 ½ - 10	GRAVELLY SAND (SW), yellowish brown, moderate density, moist, medium- to coarse-grained sand. Sample taken at 9 feet. Bottom of test pit at 10 feet. No groundwater encountered.
TP2	0 - 2	SILTY SAND (SM), yellowish brown, moderate density, moist, trace of clay, trace of fine gravel, upper 6 inches is a hard dense layer. Sample taken at 1 foot.
	2 - 7	SILTY SAND (SM), with gravel, yellowish brown, moderate density, moist, 10 - 20% angular gravels. Sample taken at 5 feet.

Logged By: Steve Crenshaw
7217.7.001.01
Date Logged: 2/17/06

Figure A-1
Page 1 of 7

State Street Project GEX
Carson City, NV

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
	7 - 12	<p>SAND (SP), light yellowish brown, moderate density, moist, fine- to medium-grained sand, traces of gravel.</p> <p>Sample taken at 9 feet.</p> <p>At 9 feet, increase in gravel, dense.</p> <p>At 11 feet, slight cementation.</p> <p>Bottom of test pit at 12 feet. No groundwater encountered.</p>
TP3	0 - 2	<p>SILTY SAND (SM), brown, moderate density, moist, fine-grained sand, traces of clay.</p>
	2 - 6 ½	<p>SILTY SAND (SM), yellowish brown, moderate density, moist, fine- to medium-grained sand, traces of gravel.</p> <p>Sample taken at 2 feet.</p> <p>Sample taken at 5 feet.</p>
	6 ½ - 11	<p>SAND (SP), with gravel, yellowish brown, moist, dense, fine- to coarse-grained sand, angular gravels.</p> <p>Sample taken at 8 feet.</p> <p>At 9 feet, increase in gravels.</p> <p>Bottom of test pit at 11 feet. No groundwater encountered.</p>

Logged By: Steve Crenshaw
7217.7.001.01
Date Logged: 2/17/06

State Street Project GBX
Carson City, NV

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP4	0 - 2	SILTY SAND (SM), brown, moderate density, moist, fine-grained sand, traces of clay. Sample taken at 2 feet.
	2 - 6 ½	SILTY SAND (SM), yellowish brown, moderate density, moist, fine- to medium-grained sand. Sample taken at 5 feet.
	6 ½ - 10 ½	SAND (SP), yellowish brown, moderate density, moist, medium- to coarse-grained sand, traces of gravel. Sample taken at 9 feet. At 9 feet, increase in gravel. Bottom of test pit at 10 ½ feet. No groundwater encountered.
TP5	0 - 1 ½	SILTY SAND (SM), with clay, brown, moderate density, moist, fine-grained sand, traces of gravel.
	1 ½ - 3	SILTY SAND (SM), yellowish brown, moderate density, very moist, fine- to medium-grained sand, traces of clay. Sample taken at 2 feet.
	3 - 5	CLAYEY SAND (SC), brown, very dense to stiff, very moist, clay nodules, medium-grained sand. Sample taken at 3 feet. Sample taken at 4 feet.

Logged By: Steve Crenshaw
7217.7.001.01
Date Logged: 2/17/06

State Street Project GEX
Carson City, NV

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
	5 - 7	SILTY SAND (SM), yellowish brown, moderate density, moist, fine- to medium-grained sand.
	7 - 10 ½	SAND (SP), with gravel, yellowish brown, moderate density, moist, fine- to coarse-grained sand. Sample taken at 9 feet. Bottom of test pit at 10 ½ feet. No groundwater encountered.
TP6	0 - 2	SILTY SAND (SM), brown, moderate density, moist, fine-grained sand, with some clay. Sample taken at 1 foot.
	2 - 5	SILTY SAND (SM), light brown, moderate density, moist, fine- to coarse-grained sand, traces of gravel. Sample taken at 4 feet.
	5 - 7	CLAYEY SAND (SC), brown, dense, moist, hard fragments. Sample taken at 6 feet.
	7 - 10 ½	SAND (SP), yellowish brown, dense, moist, with gravel. Sample taken at 8 feet. Bottom of test pit at 10 ½ feet. No groundwater encountered.

Logged By: Steve Crenshaw
7217.7.001.01
Date Logged: 2/17/06

Figure A-1
Page 4 of 7

State Street Project GEX
Carson City, NV

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP7	0 - 2	SILTY SAND (SM), brown, moderate density, moist, fine- to medium-grained sand, traces of gravel.
	2 - 6	SILTY SAND (SW), yellowish brown, moderate density, moist, fine- to medium-grained sand, traces of gravel. Sample taken at 2 feet. Sample taken at 4 feet.
	6 - 10	SAND (SP), yellowish brown, medium dense, moist, medium-grained sand, with clay, gravel. Sample taken at 7 feet. At 7 feet, clayey sand noduals with matrix. Bottom of test pit at 10 feet. No groundwater encountered.
TP8	0 - 2	SILTY SAND (SM), brown, moderately dense, moist, fine-grained sand.
	2 - 4 ½	SILTY SAND (SM), light brown, moderate density, moist, fine- to medium-grained sand, traces of clay and gravel. Sample taken at 2 feet. Sample taken at 4 feet.
	4 ½ - 10	SAND (SP), with gravels, yellowish brown, moderate density, moist, fine- to coarse-grained sand, and clayey sand noduals. Sample taken at 6 feet.

Logged By: Steve Crenshaw
7217.7.001.01
Date Logged: 2/17/06

Figure A-1
Page 5 of 7

State Street Project GEX
Carson City, NV

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
	10 - 12	GRAVELLY SAND (GW), light yellowish brown, dense, moist, medium- to coarse-grained sand, angular. Sample taken at 10 feet. Bottom of test pit at 12 feet. No groundwater encountered.
TP9	0 - 2	SILTY SAND (SM), brown, moderate density, moist, fine-grained sand, traces of gravel. Sample taken at 1 1/2 feet.
	2 - 4	SILTY SAND (SM), with some clay, light yellowish brown, moderate density, moist to slightly wet, fine- to medium-grained sand, traces of gravel.
	4 - 6	SILTY SAND (SM), with clayey sand nodules, dense, moist, fine- to medium-grained sand. Sample taken at 4 feet. Sample taken at 5 feet.
	6 - 11 1/2	SAND (SP), with gravels, light brown, dense, moist, medium-grained sand, angular sand and gravel. Sample taken at 9 feet. Bottom of test pit at 11 1/2 feet. No groundwater encountered.

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7217.7.001.01
Date Logged: 2/17/06

Figure A-1
Page 6 of 7

State Street Project GEX
Carson City, NV

TEST PIT LOGS

Test Pit Number	Depth (Feet)	Description
TP10	0 - 2	SILTY SAND (SM), brown, moderate density, moist, fine-grained sand, traces of gravel. Sample taken at 1 ½ feet.
	2 - 3	SILTY SAND (SM), with some clay, light brown, moderate density, moist, fine- to medium-grained sand. Sample taken at 2 ½ feet.
	3 - 11	SAND (SP), with gravel, light orangish brown, moderate density, moist, fine- to medium-grained sand, some clay noduals. Sample taken at 5 feet. At 9 feet, increase grain size, to coarse-grained sand. Sample taken at 10 feet. Bottom of test pit at 11 feet. No groundwater encountered.

Logged By: Steve Crenshaw
7217.7.001.01
Date Logged: 2/17/06

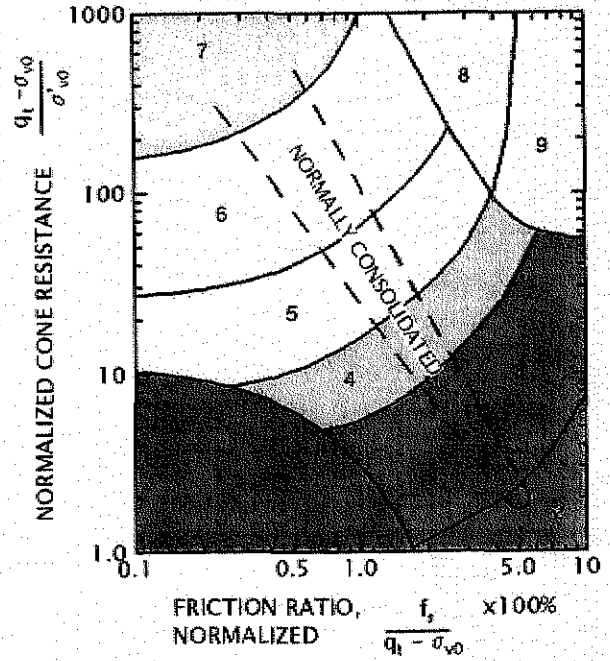
Figure A-1
Page 7 of 7

CPT Soil Classification Legend

Zone	$\frac{(q_c/p_a)}{N_{eq}}$	Description
1	2	Sensitive, Fine Grained
2	1	Organic Soils-Peats
3	1.5	Clays-Clay to Silty Clay
4	2	Silt Mixtures-Clayey Silt to Silty Clay
5	3	Sand Mixtures-Silty Sand to Sandy Silt
6	4.5	Sands-Clean Sand to Silty Sand
7	6	Gravelly Sand to Sand
8	1	Very Stiff Sand to Clayey Sand *
9	2	Very Stiff, Fine Grained *

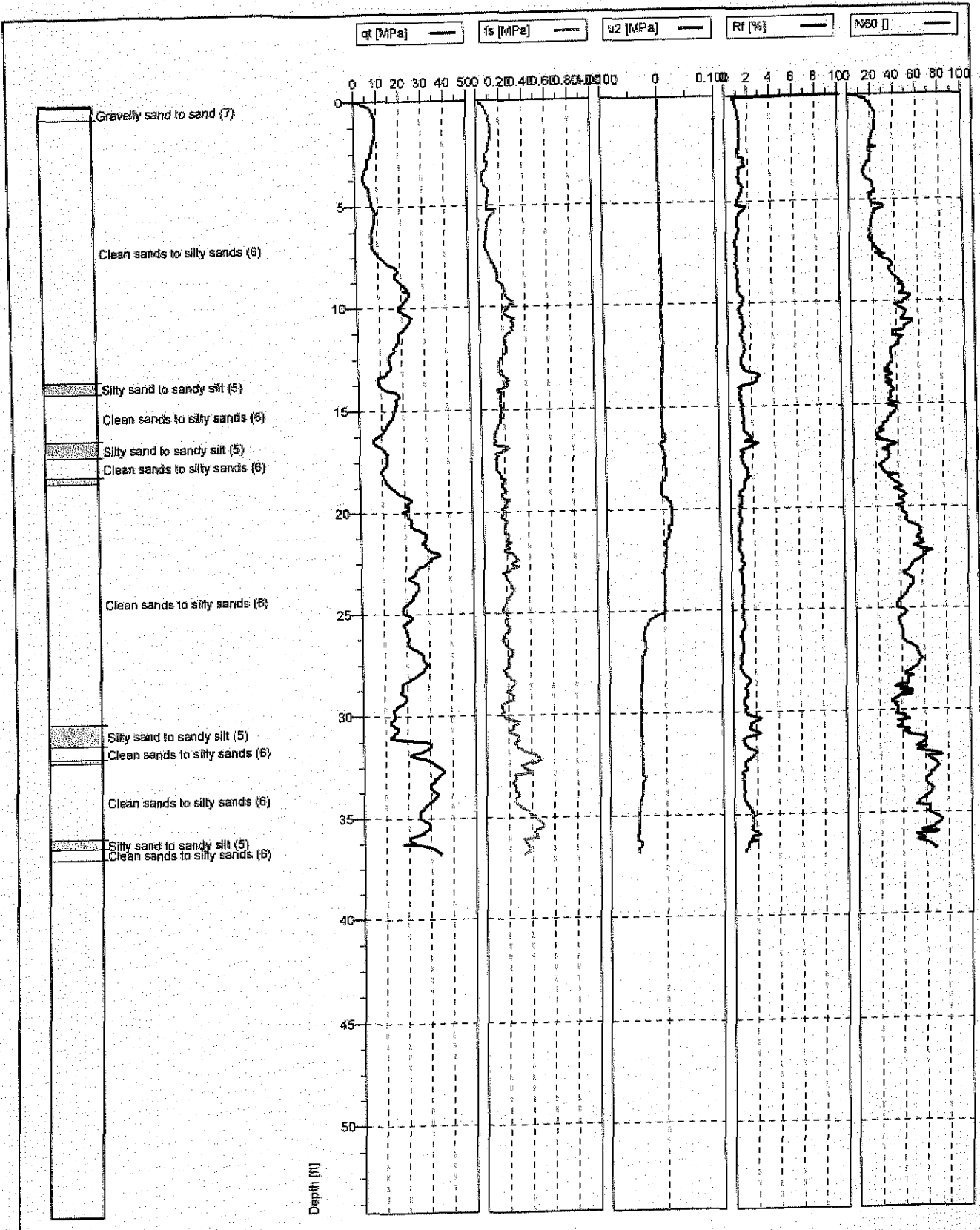
(*) Heavily Overconsolidated or Cemented

Normalized Friction Ratio Classification Chart



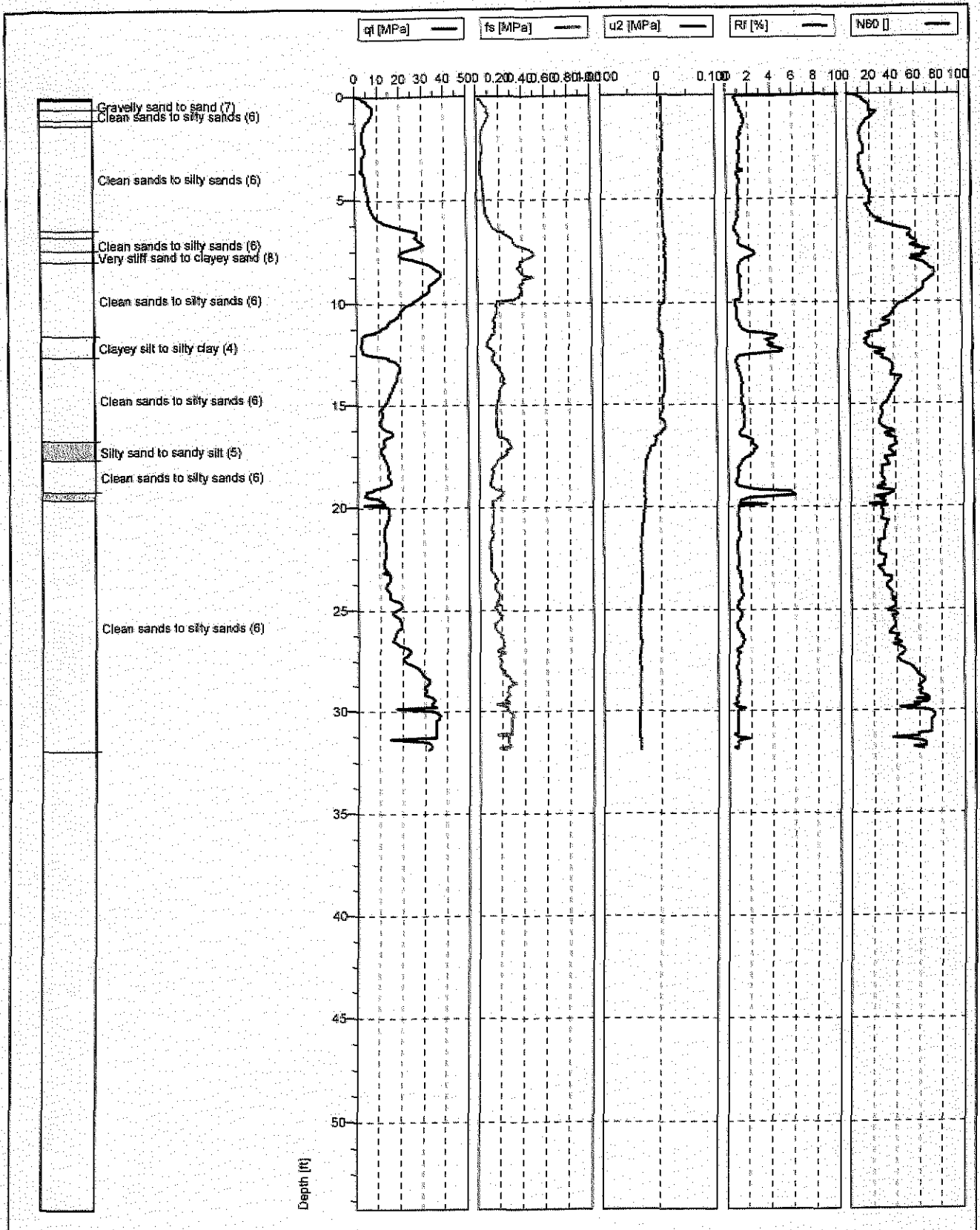
General Notes:

Class. FR - Classification based on Friction Ratio, PK Robertson, 1990




 Cone No: 3121
 Tip area [cm²]: 10
 Sleeve area [cm²]: 160

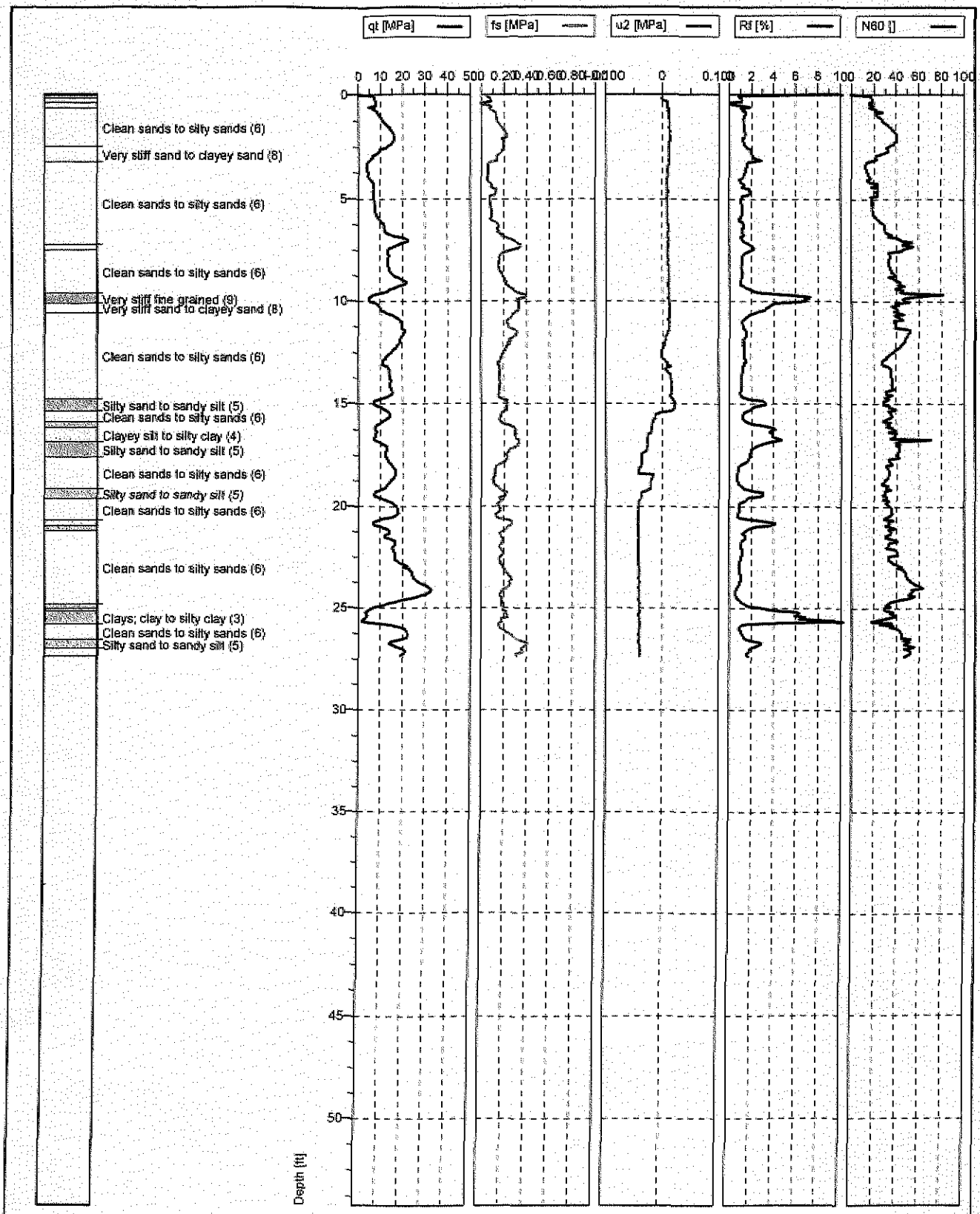
Location: Carson City, NV	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-1
Project ID: 67288	Client: ENGEO, Inc.	Date: 3/28/2006	Scale: 1 : 80
Project: Carson City		Page: 1/1	Fig: 1
		File: CPT-1CC.cpd	



Spectrum
EXPLORATION

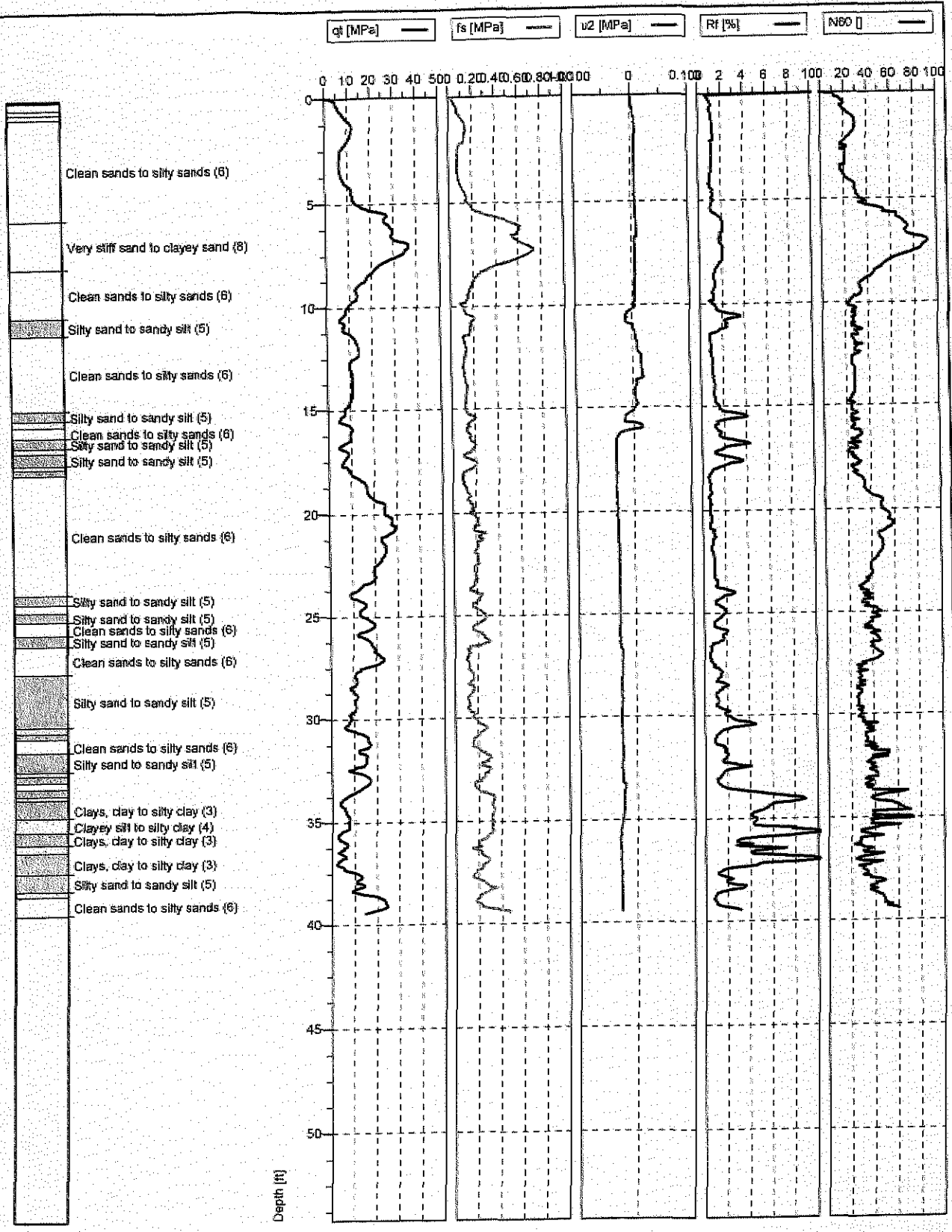
Cone No: 3121
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Location: Carson City, NV	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-2
Project ID: 67288	Client: ENGEO, Inc.	Date: 3/28/2006	Scale: 1 : 80
Project: Carson City		Page: 1/1	Fig:
		File: CPT-2CC.cpd	



Cone No: 3121
 Tip area [cm²]: 10
 Sleeve area [cm²]: 150

Location: Carson City, NV	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-3
Project ID: 67288	Client: ENGEO, Inc.	Date: 3/28/2006	Scale: 1 : 80
Project: Carson City		Page: 1/1	Fig: 1
		File: CPT-3CC.cpd	



Spectrum
EXPLORATION

Cone No: 3121
Tip area [cm²]: 10
Sleeve area [cm²]: 150

Location: Carson City, NV	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT-4
Project ID: 67288	Client: ENGEO, Inc.	Date: 3/28/2006	Scale: 1 : 80
Project: Carson City		Page: 1/1	Fig: 1
		File: CPT-4CC.cpd	

**APPENDIX B
LABORATORY TEST DATA**

**Summary of Moisture-Density Test Data
Expansion Index Test Results
Liquid and Plastic Limits Test Report
Particle Size Distribution Report
R-Value Test Report (2 pages)
Analytical Results of Soil Corrosion (3 pages)**

Summary of Moisture Test Data

Exploratory Location	Depth (feet)	Moisture Content
TP1	2	7.8
TP1	4	8.1
TP2	9	5.0
TP3	2	7.0
TP3	5	8.4
TP4	2	7.8
TP4	5	6.1
TP5	3	11.3
TP6	6	13.1
TP6	8	10.3
TP7	2	7.6
TP7	4	7.4
TP8	6	14.0
TP8	10	5.4
TP9	1½	7.2
TP9	4	10.7
TP10	2½	10.1
TP10	5	9.1

EXPANSION INDEX TEST RESULTS

ASTM D 4829

DATE: 3/1/06

JOB NO.: 7217.5.001.01

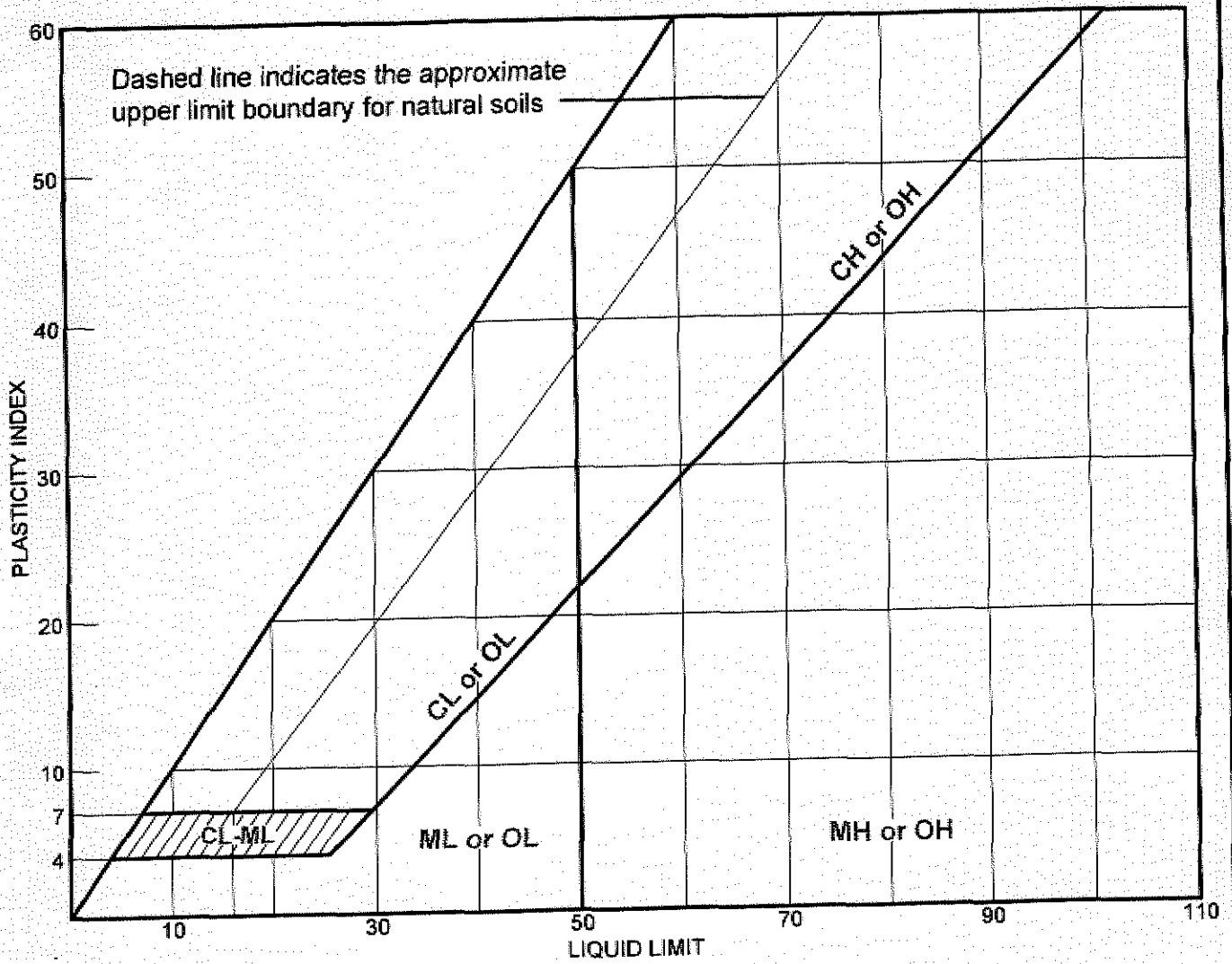
JOB NAME: State Street

SAMPLE I.D.	SOIL DESCRIPTION	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE CONTENT (%)	FINAL MOISTURE CONTENT (%)	EXPANSION INDEX
TP2@2	Brown silty sand (SM)	116.9	8.0	11.4	0
TP9@2.5	Brown silty sand (SM)	115.0	7.0	13.2	0

CLASSIFICATION OF EXPANSIVE SOIL ASTM D 4829

EXPANSION INDEX	POTENTIAL EXPANSION
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	GEX	TP2-1&2	1-2 feet		NP	NV	NP	SM
■	GEX	TP8-2	2 feet		NP	NV	NP	SM

LIQUID AND PLASTIC LIMITS TEST REPORT

**ENGE
INCORPORATED**

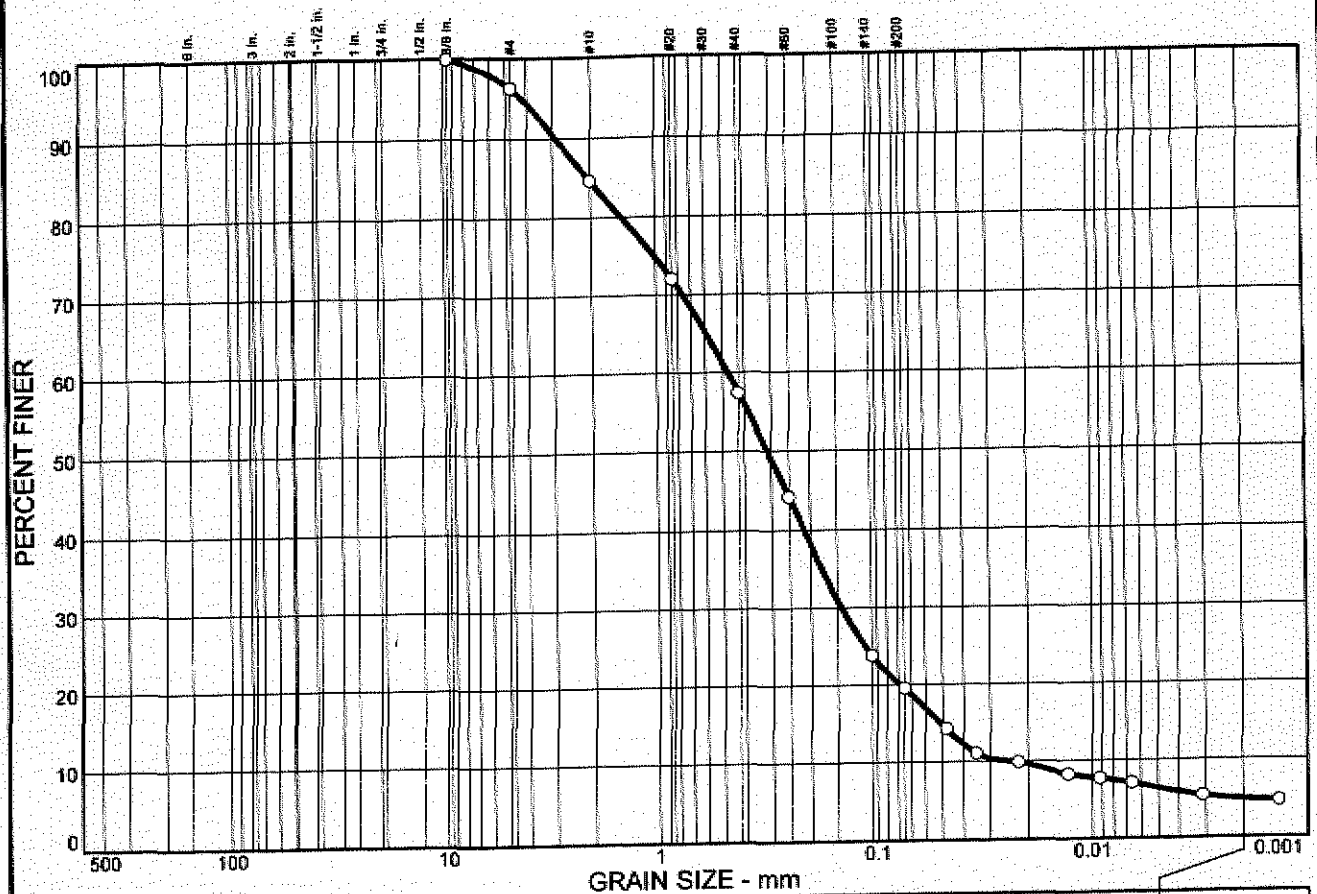
Client:

Project: State Street

Project No.: 7217.7.001.01

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.7	76.8	14.6	4.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	96.3		
#10	84.6		
#20	72.0		
#40	57.4		
#60	44.2		
#140	23.8		
#200	19.5		

* (no specification provided)

Soil Description

Brown silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D₈₅= 2.06 D₆₀= 0.475 D₅₀= 0.313
D₃₀= 0.145 D₁₅= 0.0508 D₁₀= 0.0236
C_u= 20.11 C_c= 1.86

Classification

USCS= SM AASHTO=

Remarks

Sample No.: TP8-2
Location:

Source of Sample: GEX

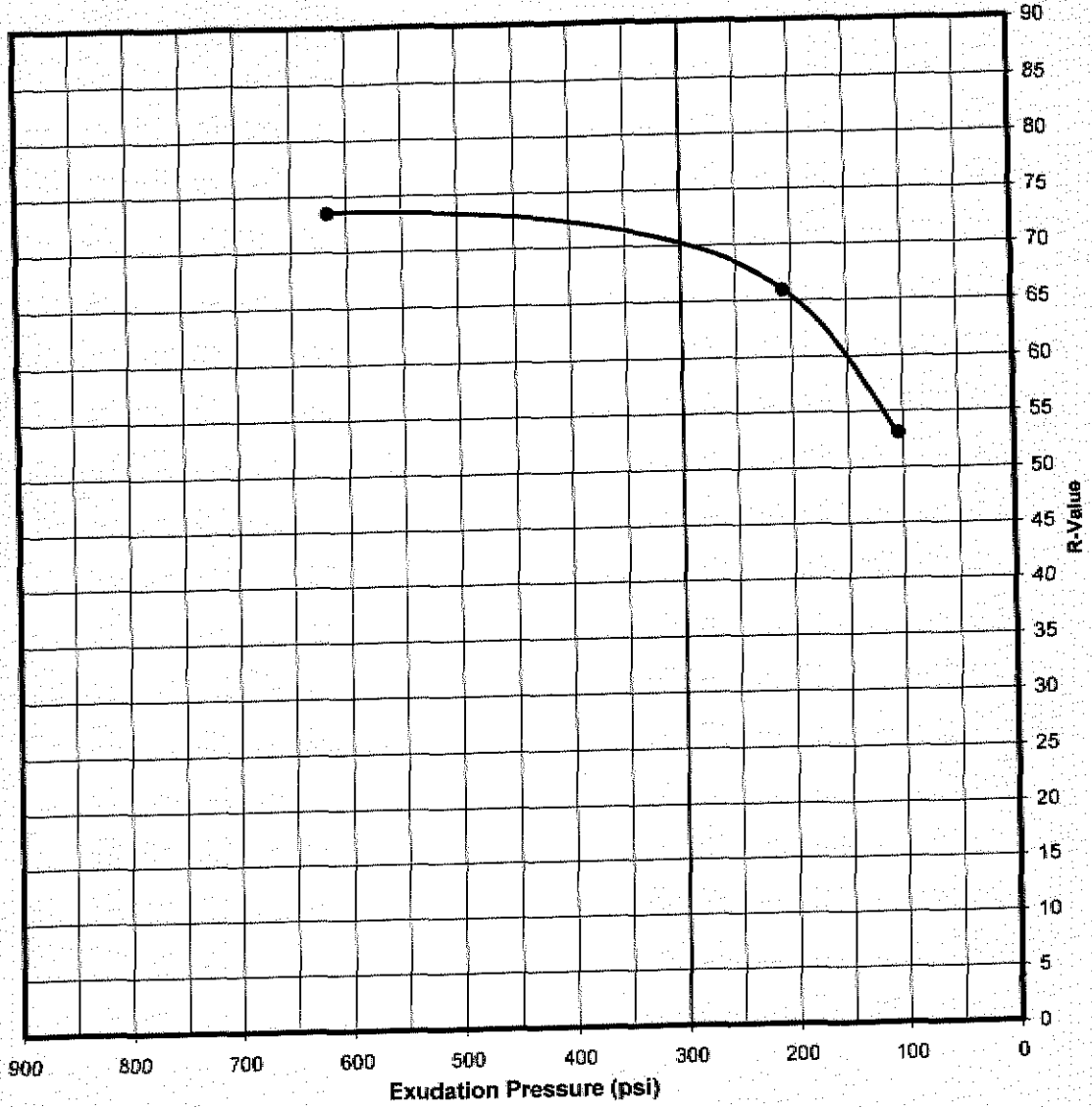
Date: 3/1/06
Elev./Depth: 2 feet

ENGEO INCORPORATED

Client:
Project: State Street

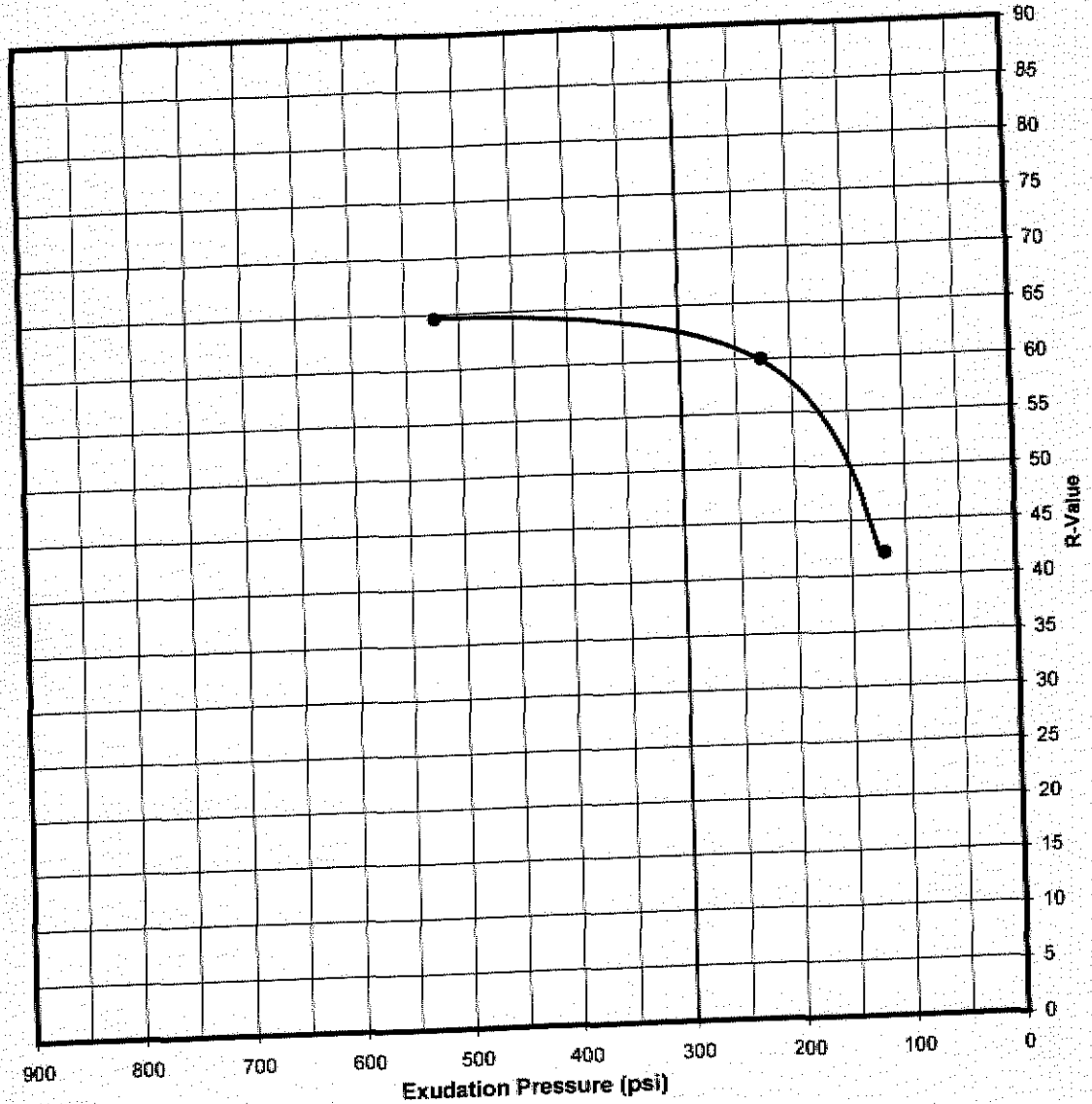
Project No: 7217.7.001.01

Figure



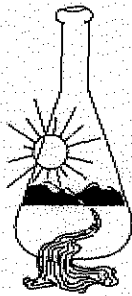
Date: 3/1/2006
 Project Name: State Street
 Project Number: 7217.7.001.01
 Sample: TP2@2
 Description: Brown silty sand (SM)

Specimen	A	B	C
Exudation Pressure, p.s.i.	617	207	103
Expansion dial (.0001")	0	0	0
Expansion Pressure, p.s.f.	0	0	0
Resistance Value, "R"	73	66	53
% Moisture at Test	8.1	9.0	9.8
Dry Density at Test, p.c.f.	116.2	123.6	123.1
"R" Value at 300 p.s.i., Exudation Pressure	70		



Date: 3/1/2006
 Project Name: State Street
 Project Number: 7217.7.001.01
 Sample: TP9@2.5
 Description: Brown silty sand (SM)

	A	B	C
Specimen	521	227	119
Exudation Pressure, p.s.i.	0	0	0
Expansion dial (.0001")	0	0	0
Expansion Pressure, p.s.f.	65	60	42
Resistance Value, "R"	8.7	9.5	10.8
% Moisture at Test	124.1	123.9	123.0
Dry Density at Test, p.c.f.			
"R" Value at 300 p.s.i., Exudation Pressure	63		



Sunland Analytical

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

Date Reported 03/01/2006
Date Submitted 02/22/2006

To: Steve Crenshaw
Engco Inc.
2213 Plaza Dr.
Rocklin, CA 95765

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

The reported analysis was requested for the following location:
Location : 7127.7.001.01 Site ID : TP2-1.
Thank you for your business.

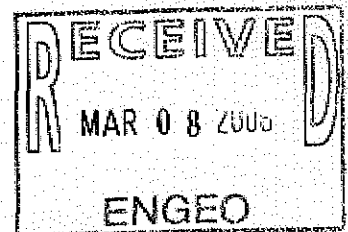
* For future reference to this analysis please use SUN # 46959-93031.

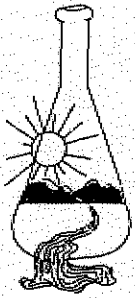
EVALUATION FOR SOIL CORROSION

Soil pH	7.52		
Minimum Resistivity	12.33	ohm-cm (x1000)	
Chloride	4.9 ppm	00.00049	%
Sulfate	0.7 ppm	00.00007	%

METHODS

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





Sunland Analytical

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

Date Reported 03/01/2006
Date Submitted 02/22/2006

To: Steve Crenshaw
Engeo Inc.
2213 Plaza Dr.
Rocklin, CA 95765

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

The reported analysis was requested for the following location:
Location : 7127.7.001.01 Site ID : TP5-2.
Thank you for your business.

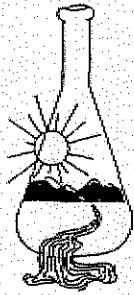
* For future reference to this analysis please use SUN # 46959-93032.

EVALUATION FOR SOIL CORROSION

Soil pH	7.24		
Minimum Resistivity	13.67	ohm-cm (x1000)	
Chloride	6.0 ppm	00.00060	%
Sulfate	0.5 ppm	00.00005	%

METHODS

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



Sunland Analytical

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

Date Reported 03/01/2006
Date Submitted 02/22/2006

To: Steve Crenshaw
Engeo Inc.
2213 Plaza Dr.
Rocklin, CA 95765

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

The reported analysis was requested for the following location:
Location : 7127.7.001.01 Site ID : TP8-2.
Thank you for your business.

* For future reference to this analysis please use SUN # 46959-93033.

EVALUATION FOR SOIL CORROSION

Soil pH	7.10		
Minimum Resistivity	15.28	ohm-cm (x1000)	
Chloride	3.1 ppm	00.00031	%
Sulfate	0.1 ppm	00.00001	%

METHODS

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

Drainage Analysis

Conceptual Drainage Study

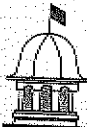
For

MILLS LANDING

Prepared for:

Landmark Homes
985 Damonte Ranch Pkwy. #300
Reno, NV 89521

Prepared by:



CAPITAL ENGINEERING

P.O. Box 3750

(775)882-5630 Fax(775)885-7282

Carson City, NV 89702

Aug, 2006

Conceptual Drainage Study For The Mills Landing

Description Of Project

This Conceptual Drainage Report has been prepared at the request of Landmark Homes to support the Tentative Map plans for Mills Landing. The Mills Landing Site is approximately 10.1 acres in size, and is located along U.S. Highway 50 as well as north of the intersection of State Street & U.S. Highway 50. This project is located in Carson City. (See vicinity map, figure 1, included in this report for general Location.) Plans call for the development of a Planned - Unit Development along with a commercial portion. The project is 94 lots and 3 commercial buildings ranging in size from 4,000 sq. ft. up to 12,000 sq. ft. (See figure 2 for site plan.) Currently the site is in FEMA Flood Zone B. (See Appendix D).

The Conceptual Drainage Study for Mills Landing takes into account the existing drainage, both pre and post development, of Millennium I at State Street. This is because much of the drainage flowing off of the Millennium I site directly affects the Mills Landingsite. Also included, is some runoff from Long Street that flows down Molly Drive into the project site. The existing sub-basins take these areas into account.

This Conceptual Drainage Study is provided to support the conceptual plans for the Mills Landing site. A Technical Drainage Study will be done at a future date to address the final design of the onsite storm drainage system and sizing of the pipes and detention basins.

Method

The rainfall data for this project comes from NOAA Atlas 14, Volume 1, Version 4. Times of Concentration (TC's) were calculated for each of the drainage basins using the type of ground cover and approximate slopes for the different TC paths for each basin.

The Rational Method was used to determine the flow rates of the site. Because the site is about 10 acres, the Rational Method accurately calculates the pre-development and post-development flows.

The site conditions range from gently sloping terrain with a 0.5% to 2% slope to a steeper terrain on the westerly side of the project sloping at approximately 20%. The existing soils fit into the hydrologic soil group of B, and have been obtained from the latest Geotechnical Report of the site. The existing ground cover consists of bitterbrush, sagebrush that are native to this area. The existing soils are classified as moderate to good condition with a runoff coefficient of .33, based on the native ground cover. The type description has been included in this report.

Pre-Development Conditions and Discharge Points

As shown in Figure 3, drainage from Basin 1 includes the existing Millennium I pre-development drainage, the Long Street drainage and the current existing Mills Landing proposed development. All the water flows in a southeasterly direction down towards to the southeast corner of the Mills Landing development site. The water is

carried over existing ground and through drainage channels. Basin 2, or the proposed commercial development drainage flows toward U.S. Highway 50 from Corbett Street. The water flows over relatively flat and bare ground. The existing discharges are as follows.

TABLE 1

PRE-DEVELOPMENT FLOWS		
SUB-BASIN	5 YEAR	100 YEAR
Sub 1	4.1 cfs	8.5 cfs
Sub 2	1.4 cfs	2.7 cfs

Post Development Conditions and Discharge Points

Figure 4 shows the post development sub-basins and proposed detention basins for the Mills Landing Development. The flows from existing Millennium I Development along with the Long Street drainage will be perpetuated through the site with an extension of the proposed storm drain system.

The water will then be combined with the new improvement runoff from basin 1. The combined flows from basin 1 will then flow onto the southeastern part of the Mills Landing development through a series of gutters, pipes, and channels. Those flows will eventually be deposited into two detention ponds and one channel along the south side of the proposed development. Flows from sub-basin 1 will be carried to a detention basin near the driveway to the south of the proposed commercial property (as shown in figure 4). The post development flows, before detention, are as follows.

TABLE 2

POST-DEVELOPMENT FLOWS BEFORE DETENTION		
SUB-BASIN	5 YEAR	100 YEAR
Sub 1	7.5 cfs	13.8 cfs
Sub 2	2.6 cfs	5.1 cfs

The detention ponds for basin 1 will provide a reduction in 5 year peak discharge from 7.5 cfs to less than 4.1 cfs. The volume required is 10367 cu. ft., and the total storage provided is 16076 cu. ft. During the 100 -year storm, once the ponds fill up, the discharge rate will be approximately 11 cfs (See Table 3). This is because the ponds can hold all but 2044 cu. ft. of post-development peak drainage.

The detention pond for basin 2 will hold both the 5 and 100 year storms. The storage required for this pond for the 5-year storm is 2286 cu. ft. and the storage required for the 100-year storm is 4470 cu. ft. The total storage provided is 4869 cu. ft. This storage will allow for a discharge less than of 1.4 for the 5-year storm and less than 2.6 cfs for the 100-year storm. (See Table 3).

TABLE 3

POST-DEVELOPMENT FLOWS AFTER DETENTION		
SUB-BASIN	5 YEAR	100 YEAR
Sub 1	< 4.1 cfs	11 cfs
Sub 2	< 1.4 cfs	< 2.6 cfs

Conclusion

This conceptual study has evaluated the existing and proposed storm water runoff conditions for the proposed development. The flows from the offsite sub-basins will continue to be routed through the project only slightly impacting the proposed development. The study presents detailed evidence that the proposed drainage system will not increase the offsite storm water flows, from the pre-development levels. This study complies with Carson City Standards and requirements. A more detailed evaluation of the storm drainage system will be prepared and submitted with the Mills Landing Development Plans.

APPENDIX 'A'

Rail Fall / Soil Types





POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14



Nevada 39.168 N 119.764 W 4799 feet
 from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 4
 G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M. Yekta, and D. Riley
 NOAA, National Weather Service, Silver Spring, Maryland, 2006

Extracted: Mon Jul 17 2006

[Confidence Limits](#) |
 [Seasonality](#) |
 [Location Maps](#) |
 [Other Info.](#) |
 [GIS data](#) |
 [Maps](#) |
 [Help](#) |
 [Docs](#) |
 [U.S. Map](#)

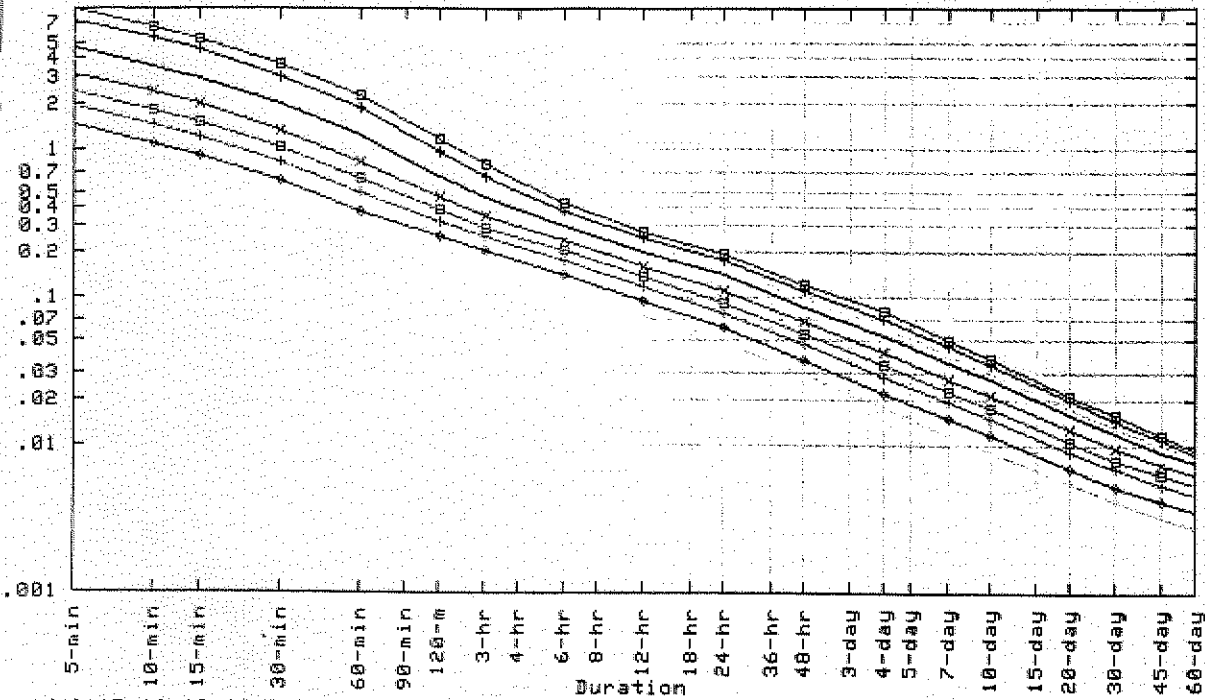
Precipitation Intensity Estimates (in/hr)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	1.15	0.88	0.73	0.49	0.30	0.20	0.16	0.11	0.07	0.05	0.03	0.02	0.01	0.01	0.01	0.00	0.00	0.00
2	1.44	1.09	0.90	0.61	0.38	0.25	0.20	0.14	0.09	0.06	0.04	0.02	0.01	0.01	0.01	0.01	0.00	0.00
5	1.92	1.46	1.21	0.81	0.50	0.32	0.26	0.18	0.12	0.08	0.05	0.03	0.02	0.01	0.01	0.01	0.01	0.00
10	2.39	1.82	1.50	1.01	0.63	0.39	0.30	0.20	0.14	0.09	0.06	0.03	0.02	0.02	0.01	0.01	0.01	0.01
25	3.14	2.39	1.98	1.33	0.82	0.48	0.36	0.24	0.16	0.11	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.01
50	3.84	2.92	2.42	1.63	1.01	0.56	0.41	0.27	0.18	0.12	0.08	0.05	0.03	0.02	0.01	0.01	0.01	0.01
100	4.67	3.55	2.94	1.98	1.22	0.66	0.47	0.30	0.20	0.14	0.09	0.05	0.04	0.03	0.02	0.01	0.01	0.01
200	5.66	4.31	3.56	2.40	1.48	0.77	0.54	0.33	0.22	0.15	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01
500	7.24	5.51	4.55	3.06	1.90	0.97	0.66	0.38	0.25	0.18	0.11	0.07	0.05	0.03	0.02	0.01	0.01	0.01
1000	8.66	6.59	5.45	3.67	2.27	1.15	0.78	0.43	0.27	0.19	0.12	0.08	0.05	0.04	0.02	0.02	0.01	0.01

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.

Partial duration based Point IDF Curves
39.168 N 119.764 W 4799 ft



Mon Jul 17 16:10:13 2006

Average Recurrence Interval (years)	
2-year	+
5-year	+
10-year	+
25-year	+
100-year	—
500-year	+
1000-year	+

Confidence Limits -

* Upper bound of the 90% confidence interval
Precipitation Intensity Estimates (in/hr)

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	1.37	1.04	0.86	0.58	0.36	0.23	0.18	0.13	0.08	0.05	0.03	0.02	0.01	0.01	0.01	0.00	0.00	0.00
2	1.70	1.30	1.08	0.72	0.45	0.29	0.23	0.16	0.11	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.00	0.00
5	2.28	1.73	1.44	0.97	0.60	0.37	0.29	0.20	0.13	0.09	0.05	0.03	0.02	0.02	0.01	0.01	0.01	0.01
10	2.83	2.15	1.78	1.20	0.74	0.44	0.33	0.23	0.15	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01	0.01
25	3.73	2.84	2.35	1.58	0.98	0.55	0.40	0.27	0.18	0.12	0.08	0.05	0.03	0.02	0.01	0.01	0.01	0.01
50	4.58	3.49	2.88	1.94	1.20	0.65	0.47	0.30	0.21	0.14	0.09	0.05	0.04	0.03	0.02	0.01	0.01	0.01
100	5.64	4.29	3.54	2.39	1.48	0.77	0.54	0.34	0.23	0.15	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01
200	6.95	5.29	4.37	2.94	1.82	0.92	0.63	0.39	0.26	0.17	0.11	0.07	0.05	0.03	0.02	0.01	0.01	0.01
500	9.07	6.90	5.70	3.84	2.38	1.20	0.81	0.45	0.30	0.20	0.13	0.08	0.05	0.04	0.02	0.02	0.01	0.01
1000	11.05	8.41	6.95	4.68	2.90	1.46	0.98	0.51	0.33	0.22	0.14	0.09	0.06	0.04	0.02	0.02	0.01	0.01

The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than.

** These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.

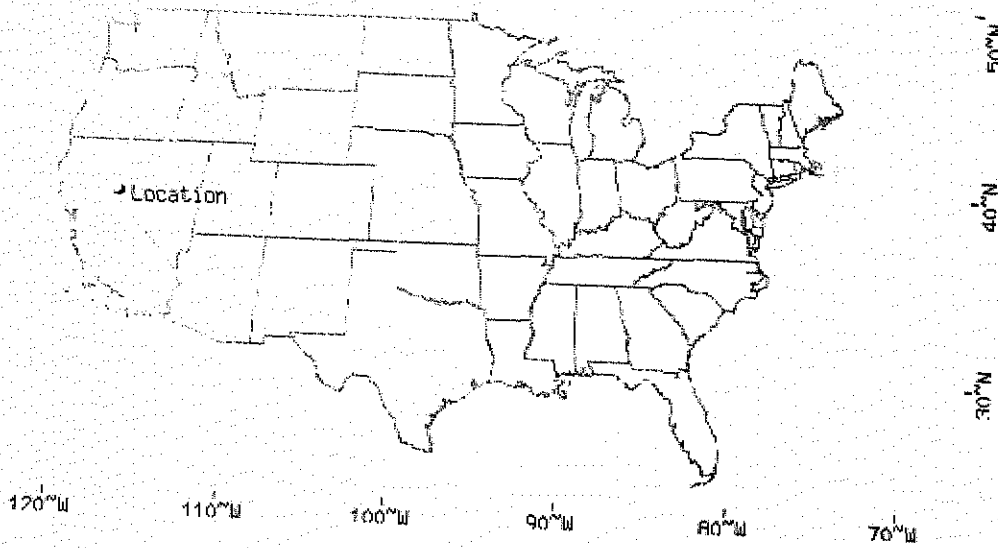
Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

*** Lower bound of the 90% confidence interval
Precipitation Intensity Estimates (in/hr)**

ARI** (ars)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	1.00	0.76	0.63	0.42	0.26	0.18	0.15	0.10	0.07	0.04	0.03	0.02	0.01	0.01	0.00	0.00	0.00	0.00
2	1.25	0.95	0.78	0.53	0.33	0.23	0.18	0.13	0.08	0.06	0.03	0.02	0.01	0.01	0.01	0.00	0.00	0.00
5	1.66	1.26	1.04	0.70	0.43	0.29	0.23	0.16	0.10	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.00	0.00
10	2.03	1.54	1.28	0.86	0.53	0.34	0.26	0.18	0.12	0.08	0.05	0.03	0.02	0.02	0.01	0.01	0.01	0.00
25	2.59	1.97	1.63	1.10	0.68	0.41	0.31	0.21	0.14	0.10	0.06	0.04	0.02	0.02	0.01	0.01	0.01	0.00
50	3.07	2.33	1.93	1.30	0.80	0.47	0.35	0.23	0.16	0.11	0.07	0.04	0.03	0.02	0.01	0.01	0.01	0.01
100	3.61	2.74	2.27	1.53	0.94	0.53	0.39	0.25	0.17	0.12	0.07	0.05	0.03	0.02	0.01	0.01	0.01	0.01
200	4.20	3.19	2.64	1.78	1.10	0.60	0.44	0.28	0.19	0.14	0.08	0.05	0.03	0.02	0.01	0.01	0.01	0.01
500	5.06	3.85	3.18	2.14	1.33	0.73	0.53	0.31	0.20	0.15	0.09	0.06	0.04	0.03	0.02	0.01	0.01	0.01
1000	5.76	4.39	3.63	2.44	1.51	0.83	0.60	0.34	0.22	0.16	0.10	0.06	0.04	0.03	0.02	0.01	0.01	0.01

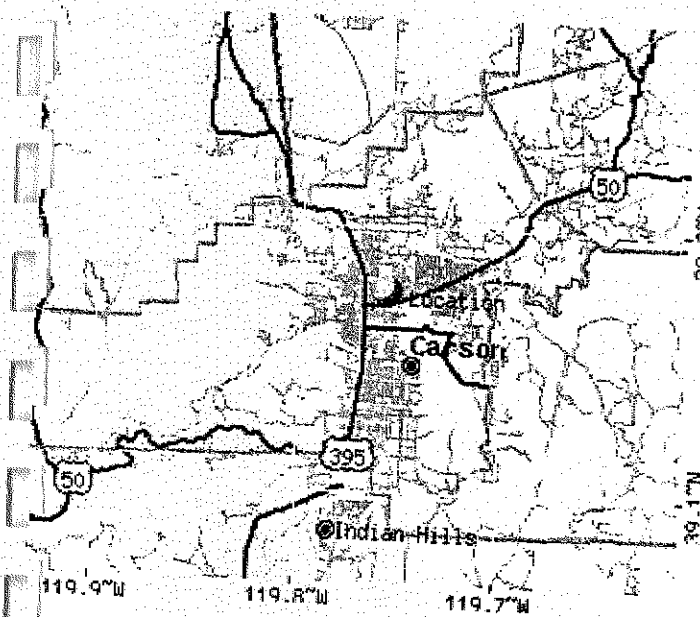
Lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than. These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence interval. Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Maps -



These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.



LEGEND

- State
- County
- Indian Resv
- Lake/Pond/Ocean
- Street
- Expressway
- Highway
- Connector
- Stream
- Military Area
- National Park
- Other Park
- City
- County

Scale 1:228583

0 2 4 6 8 10 12 14 16 18 mi

0 2 4 6 8 10 km

*average--true scale depends on monitor resolution

Other Maps/Photographs -

View USGS digital orthophoto quadrangle (DOQ) covering this location from TerraServer; USGS Aerial Photograph may also be available from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera lens has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the USGS for more information.

Watershed/Stream Flow Information -

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study, please refer to our documentation.

Using the National Climatic Data Center's (NCDC) station search engine, locate other climate stations within:

...OR... of this location (39.168/-119.764). Digital ASCII data can be obtained directly from NCDC.

Find Natural Resources Conservation Service (NRCS) SNOTEL (SNOWpack TELEmetry) stations by visiting the Western Regional Climate Center's state-specific SNOTEL station maps.

Hydrological Design Studies Center
 NOAA/National Weather Service
 East-West Highway
 Silver Spring, MD 20910
 713-1669
 Questions?: HDSC_Questions@noaa.gov

Printer

of the kind and amount of clay in the soil and on measurements of similar soils. The size of the load and the magnitude of the change in soil moisture content also influence the swelling of soils. Shrinking and swelling of some soils can cause damage to building foundations, basement walls, roads, and other structures unless special designs are used. A high shrink-swell potential indicates that special design and added expense may be required if the planned use of the soil will not tolerate large volume changes.

Risk of corrosion pertains to potential soil-induced chemical action that dissolves or weakens uncoated steel or concrete. The rate of corrosion of uncoated steel is related to soil moisture, particle-size distribution, total acidity, and electrical conductivity of the soil material. The rate of corrosion of concrete is based mainly on the sulfate content, texture, and acidity of the soil. Protective measures for steel or more resistant concrete help to avoid or minimize damage resulting from the corrosion. Uncoated steel intersecting soil boundaries or soil horizons is more susceptible to corrosion than an installation that is entirely within one kind of soil or within one soil horizon.

Erosion factors are used to predict the erodibility of a soil and its tolerance to erosion in relation to specific kinds of land use and treatment. The soil erodibility factor (K) is a measure of the susceptibility of the soil to erosion by water. Soils having the highest K values are the most erodible. K values range from 0.10 to 0.64. To estimate annual soil loss per acre, the K value of a soil is modified by factors representing plant cover, grade and length of slope, management practices, and climate. The soil-loss tolerance factor (T) is the maximum rate of soil erosion, whether from rainfall or soil blowing, that can occur without reducing crop production or environmental quality. The rate is expressed in tons of soil loss per acre per year.

Wind erodibility groups are made up of soils that have similar properties that affect their resistance to soil blowing if cultivated. The groups are used to predict the susceptibility of soil to blowing and the amount of soil lost as a result of blowing. Soils are grouped according to the following distinctions:

1. Sands, coarse sands, fine sands, and very fine sands. These soils are extremely erodible, so vegetation is difficult to establish. They are generally not suitable for crops.
2. Loamy sands, loamy fine sands, and loamy very fine sands. These soils are very highly erodible, but crops can be grown if intensive measures to control soil blowing are used.
3. Sandy loams, coarse sandy loams, fine sandy loams, and very fine sandy loams. These soils are highly erodible, but crops can be grown if intensive measures to control soil blowing are used.
4. Calcareous loamy soils that are less than 35 percent clay and more than 5 percent finely divided calcium carbonate. These soils are erodible, but crops can be grown if intensive measures to control soil blowing are used.

4. Clays, silty clays, clay loams, and silty clay loams that are more than 35 percent clay. These soils are moderately erodible, but crops can be grown if measures to control soil blowing are used.

5. Loamy soils that are less than 18 percent clay and less than 5 percent finely divided calcium carbonate and sandy clay loams and sandy clays that are less than 5 percent finely divided calcium carbonate. These soils are slightly erodible, but crops can be grown if measures to control soil blowing are used.

6. Loamy soils that are 18 to 35 percent clay and less than 5 percent finely divided calcium carbonate, except silty clay loams. These soils are very slightly erodible, and crops can easily be grown.

7. Silty clay loams that are less than 35 percent clay and less than 5 percent finely divided calcium carbonate. These soils are very slightly erodible, and crops can easily be grown.

8. Stony or gravelly soils and other soils not subject to soil blowing.

Soil and water features

Table 15 contains information helpful in planning land uses and engineering projects that are likely to be affected by soil and water features.

Hydrologic soil groups are used to estimate runoff from precipitation. Soils not protected by vegetation are placed in one of four groups on the basis of the intake of water after the soils have been wetted and have received precipitation from long-duration storms.

The four hydrologic soil groups are:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist chiefly of deep, well drained to excessively drained sands or gravels. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils that have a layer that impedes the downward movement of water or soils that have moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clay soils that have a high shrink-swell potential, soils that have a permanent high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

Flooding is the temporary covering of soil by water from overflowing streams, by runoff water from adjacent

APPENDIX 'B'

Rational Method Calculations

TRAVEL TIME VELOCITY

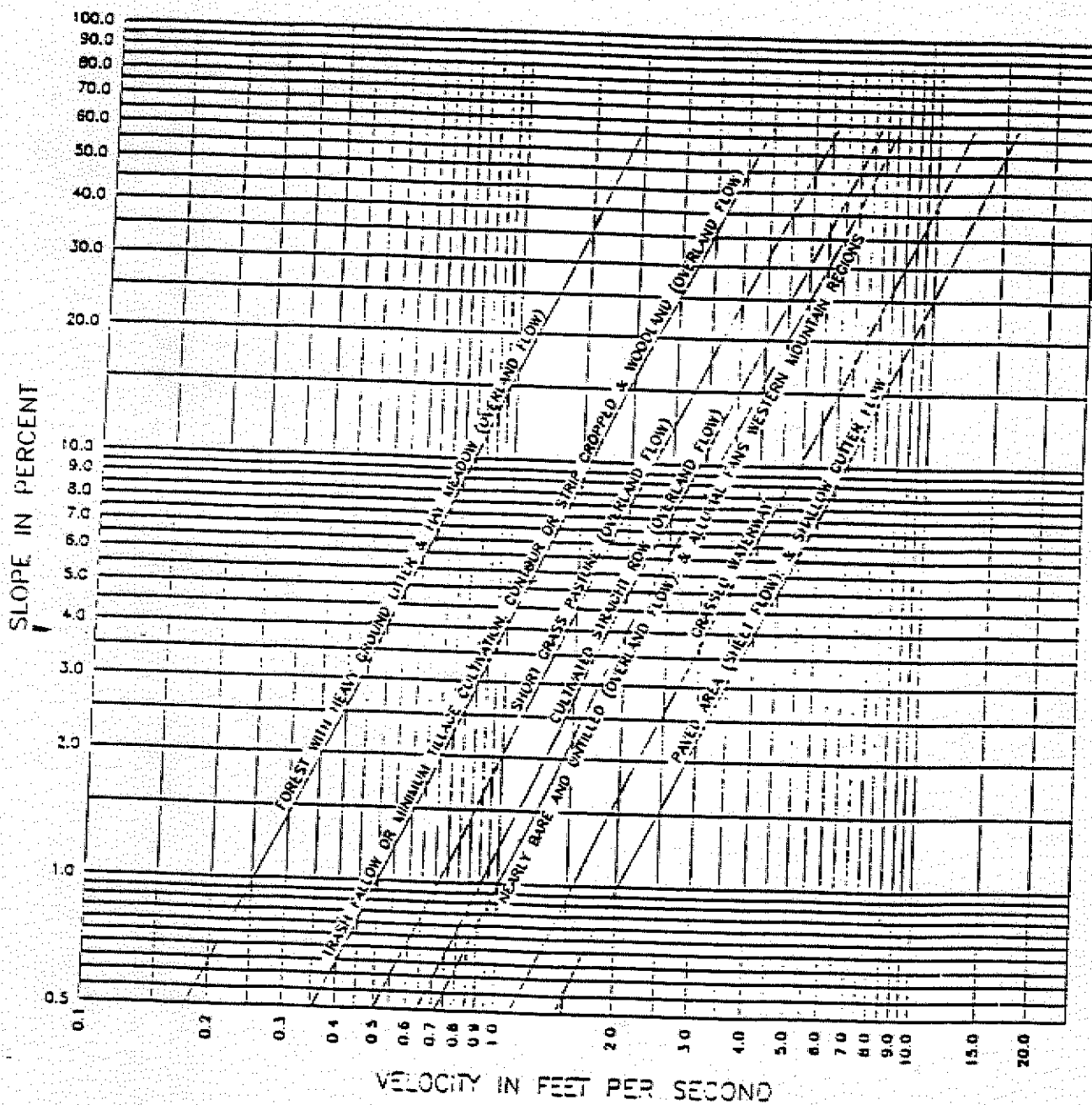


TABLE 7-7

RATIONAL METHOD
RUNOFF COEFFICIENTS

SURFACE CHARACTERISTICS	PERCENT IMPERVIOUS	RETURN FREQUENCY		
		<u>5</u>	<u>10</u>	<u>100</u>
<u>Residential</u>				
Single Family	--	.45	.50	.60
Multi-Family	70	.65	.70	.80
1/2 acre	--	.35	.40	.60
<u>Commercial</u>				
Neighborhood	95	.65	.70	.80
Downtown	95	.87	.88	.89
<u>Industrial</u>				
Light Industrial	80	.72	.76	.82
Heavy Industrial	90	.80	.85	.90
<u>Parks</u>				
	7	.10	.35	.60
<u>Playgrounds</u>				
	13	.25	.35	.65
<u>Streets</u>				
Asphalt Paved	100	.88	.90	.93
Unpaved	13	.25	.35	.89
<u>Lawns</u>				
Silty Soil*	0	.13	.18	.22
<u>Roofs</u>				
	90	.85	.90	.90
<u>Undeveloped Areas</u>				
Cultivated Land*	8	.38	.41	.51
Pasture/Range*	5	.36	.38	.49
Forest/Woodlands*	3	.34	.36	.47

*Based on Average Slope 2-7%

$$T_{\text{Exist}} = \frac{\text{dist (ft)}}{\text{vel (ft/sec)}}$$

Tot. dist = 1300'

Vel = ?

- 1. Slope 1 = 1.5% for 480' over curb vel = 2.5 ft/sec
- 2. Slope 2 = 12% for 150' over sage vel = 1.6 ft/sec
- 3. Slope 3 = 2.2% for 670' over sage vel = .7 ft/sec

$$T_{c1} = \frac{480}{2.5} = 192 \text{ sec} = 3.2 \text{ min}$$

$$T_{c2} = \frac{150}{1.6} = 93.8 \text{ sec} = 1.6 \text{ min}$$

$$T_{c3} = \frac{670}{.7} = 957 \text{ sec} = 15.9 \text{ min}$$

$$T_{\text{Total}} = T_{c1} + T_{c2} + T_{c3} = 20.7 \text{ min}$$

20.7 min + Infil. Rate of 10 min

$$20.7 + 10 \text{ min} = \underline{\underline{30.7 \text{ min}}}$$

$$T_{\text{Exist}} = \underline{\underline{30.7 \text{ min}}}$$

Engineer's Computation Pad

$T_{Exist} = 30.7 \text{ mm}$

For 30.7 mm or 31 mm

Intensity = .9 so, $i = .9 \text{ in/hr}$
- Per Carson City, Renfell Curves

$Q = CIA$

C is Renfell Coefficient, $C = .33$

- Per Carson City SO Manual,
- Slope over 10% $+ 2\%$
- Some $C = .4$ for $1\% - 10\%$

$A = \text{Area}, A = 13.8 \text{ ac}$

$Q = (.33)(.9)(13.8)$

$Q = 4.1 \text{ cfs}$

$$T_{c, \text{prep}} = \frac{d, s}{V_{el}}$$

1. Slope 1, 0.5% for 910' over curb, $V_{el} = 2.5 \text{ ft/sec}$
2. Slope 2, 12% for 50' over channel $V_{el} = 2.0 \text{ ft/sec}$
3. Slope 3, 1.5% for 360' over channel $V_{el} = 1.2 \text{ ft/sec}$
4. Slope 4, 1.8% for 55' over curbs $V_{el} = 2.6 \text{ ft/sec}$
5. Slope 5, 1.0% for 180' over curbs $V_{el} = 2.0 \text{ ft/sec}$
6. Slope 6, 1.1% for 860' over curb $V_{el} = 2.1 \text{ ft/sec}$

$$T_{c_1} = \frac{910}{2.5} = 364 \text{ sec} = 6.1 \text{ min}$$

$$T_{c_2} = \frac{50}{3} = 16.7 \text{ sec} = 0.28 \text{ min}$$

$$T_{c_3} = \frac{360}{1.2} = 300 \text{ sec} = 5 \text{ min}$$

$$T_{c_4} = \frac{55}{2.6} = 21.2 \text{ sec} = 0.35 \text{ min}$$

$$T_{c_5} = \frac{180}{2.0} = 90 \text{ sec} = 1.5 \text{ min}$$

$$T_{c_6} = \frac{860}{2.1} = 409.5 \text{ sec} = 6.8 \text{ min}$$

$$T_{c, \text{total}} = 17.33 \text{ min} + 10 \text{ min Infiltration delay}$$

$$T_{c, \text{prep}} = 27.3 \text{ min}$$

$T_{c \text{ Prop}} = 27.3 \text{ min}$

For 27.3 min or 27 min

$C = 1.1 \text{ in/hr}$

$Q = CIA$

Catchment development is a suburban house, from
Cutter's Manual, use constant 1.5

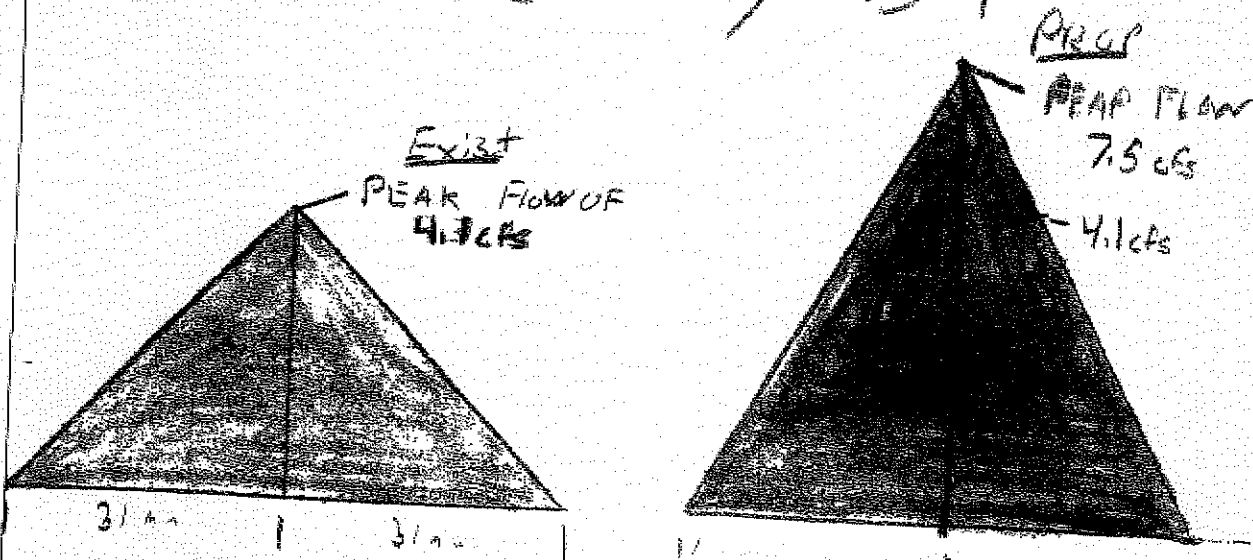
$C = 1.5$

$A = 13.6 \text{ ac}$

$Q = (1.5)(1.1)(13.6)$

$Q = \underline{7.5 \text{ cfs}}$

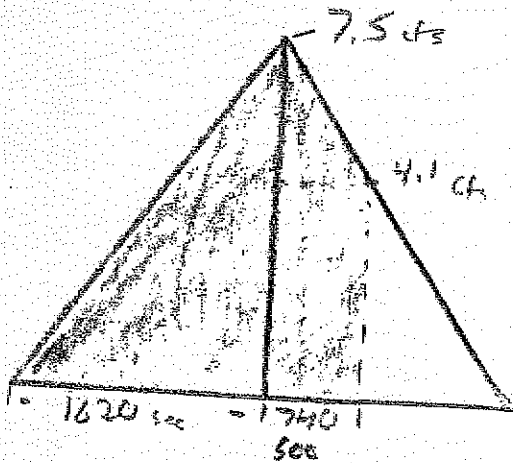
Assume Triangular Hydrograph



$$27 \text{ min} = 1620 \text{ sec}$$

$$\frac{7.5}{1620} = .0046$$

$$.0046 = \frac{7.5 - 4.1}{\text{time sec}} \quad \text{time} = 740 \text{ sec} = 12 \text{ min}$$



Diff of 3.4 ft

$$\text{Vol} = (.5 \times 1620 \times 7.5) + (.5 \times 740 \times 3.4) + (740 \times 4.1) = \underline{\underline{10367 \text{ cu. ft}}}$$

$$\text{Vol Provided} = 16076 \text{ cu. ft}$$

$$\underline{\underline{10367 < 16076}}$$

$$T_c = 30.7 \text{ mm}$$

For 31 mm $C = 1.9 \text{ in/hr}$

$$Q = C \cdot A$$

$$C = .33 \text{ (standard metric)}$$

$$A = 13.6 \text{ m}^2$$

$$Q = (.33)(1.9)(13.6)$$

$$Q = \underline{\underline{8.5 \text{ cfs}}}$$

$$T_{\text{design}} = 27.3 \text{ min}$$

For 27 min $i = 2 \text{ in/hour}$

$$i = 2$$

$$Q = C \cdot A$$

$C = .5$ for post dev/f.

$$A = 13.8 \text{ ac}$$

$$Q = (.5)(2)(13.8)$$

$$Q = \underline{\underline{13.8 \text{ cfs}}}$$

$$27 \text{ mm} = 1620 \text{ sec}$$

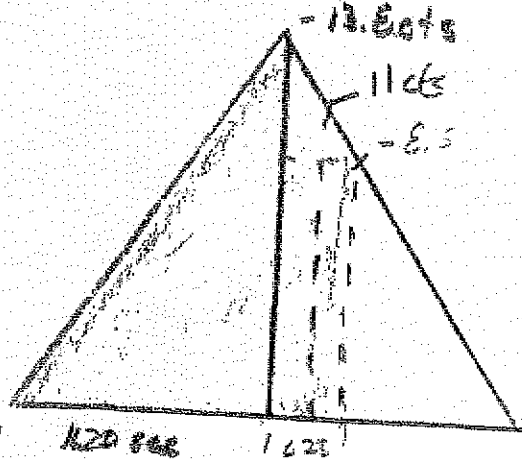
$$\frac{13.8}{1620} = 10085$$

$$\frac{2.8}{x} = 10085$$

$$x = 330$$

$$10085 = \frac{13.8 - 8.5}{\text{time (sec)}}$$

$$\text{time} = 623.5 \text{ sec} = 10 \text{ min}$$



$$D.H. = 5.3 \text{ cfs}$$

$$\text{Vol} = (.5 \times 1620 \times 13.8) + (.5 \times 623 \times 5.3) + (623 \times 8.5) = \underline{\underline{18120 \text{ cu. ft.}}}$$

$$\text{Vol Provided} = 16076 \text{ cu. ft.}$$

$$100 \text{ year overflow} = \underline{\underline{2044 \text{ cu. ft.}}}$$

PRB

$$T_{c \text{ Exist}} = \frac{\text{dist}}{\text{vel}}$$

$$\text{dist} = 460'$$

1. Slopes at about 2% for 460' over Nearly Bare Land
Vel = 1.4 ft/sec

$$T_c = \frac{460}{1.4} = 328 \text{ sec} = 5.5 \text{ min}$$

$$T_c = 5.5 \text{ min} + 10 \text{ min infiltration time} \\ = 15.5 \text{ min}$$

$$T_{c \text{ Exist}} = 15.5 \text{ min}$$

Post

$$\text{dist} = 460'$$

1. Slope at 15% for 350' over curb vel = 1.4 ft/sec
2. Slope at 1.0% for 140' " " " " vel = 2.0 ft/sec
3. Slope at 2.0% for 70' " " " " vel = 2.8 ft/sec

$$T_{c1} = \frac{350}{1.4} = 250 \text{ sec} = 4.2 \text{ min}$$

$$T_{c2} = \frac{140}{1.0} = 140 \text{ sec} = 2.3 \text{ min}$$

$$T_{L3} = \frac{70}{20} = 3.5 \text{ sec} = .58 \text{ min}$$

$$T_{L \text{ total}} = 4.2 + 2.3 + .58 = 7.1 \text{ min}$$

+ 10 min. initial window time
17.1 min

5 year storm (PREF)

$$T_{L \text{ Exist}} = 15.5 \text{ min}$$

For 16 mm, Intensity (i) = 1.5 in/hr
- Per Carson City Rainfall Curves

$$Q = C i A$$

$C = .38$ - Blank lane, no bushes, been driven on.
- Per Carson City SD manual.

$$A = 2.43 \text{ ac}$$

$$Q = (.38)(1.5)(2.43)$$

$$\underline{Q = 1.4 \text{ cfs}}$$

$$T_{\text{prop}} = 17.1 \text{ min}$$

$$\text{For } 17.1 \text{ } C = 1.5 \text{ m/hr}$$

$$Q = C \cdot A$$

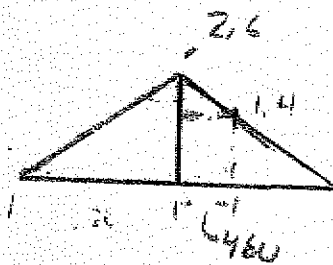
$$C = 1.7$$

$$A = 2.413$$

$$Q = (1.7)(1.5)(2.413)$$

$$Q = 2.6 \text{ CF}$$

$$17 \text{ min} = 1020 \text{ sec}$$



$$\frac{2.6}{1020} = 10025$$

$$10025 = \frac{2.6 - 1.4}{\text{Time (sec)}} = 480 \text{ sec} = \underline{8 \text{ min}}$$

$$\text{Vol} = (1.5 \times 1020 \times 2.6) + (1.5 \times 480 \times 1.2) + (480 \times 1.4) = \underline{\underline{2286 \text{ cu. ft.}}}$$

$$\text{Vol Needed} = 2286 \text{ cu. ft.}$$

$$\text{Storage Provided} =$$

$$T_{C \text{ Exal}} = 16 \text{ mm}$$

$$\text{For } 16 \text{ mm } C = 2.9 \text{ in/hr}$$

$$Q = C \cdot A$$

$$C = 38$$

$$A = 2.13$$

$$Q = (38)(2.9)(2.13)$$

$$Q = 2.7 \text{ cfs}$$

(POST)

$$T_{\text{up}} = 17 \text{ mm}$$

$$\text{For } 17 \text{ mm } C = 3.0 \text{ in/hr}$$

$$Q = C \cdot A$$

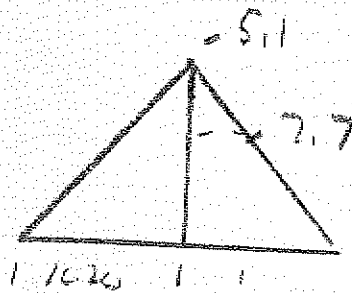
$$C = 7$$

$$A = 2.13$$

$$Q = (7)(3)(2.13)$$

$$Q = 5.1 \text{ cfs}$$

$$17 \text{ mm} = 1020 \text{ sec}$$



$$\frac{5.1}{1020} = 1005$$

$$1005 = \frac{5.1 \cdot 2.7}{6.00 \text{ (sec)}} = 480 \text{ sec}$$

$$D.H. = 2.41 \text{ cfs}$$

$$\text{Vol} = (.5 \times 1020 \times 5.1) + (.5 \times 480 \times 2.41) + (480 \times 2.7) = 4470$$

Storage Needed 4470 cu. ft.

$$\text{Vol Provided} = 4800 \text{ cu. ft.}$$

APPEENDIX 'C'
Detention Ponds

Elev	Sq. Ft	Cu. Ft
57.0	= 2514.5	0(2514.5) = 0.0
57.5	= 2807.5	.5(2807.5) = 1403.8
58.0	= 3307.6	.5(3307.6) = 1553.8
58.5	= 3413.6	.5(3413.6) = 1706.8
59.0	= 3725.8	.5(3725.8) = 1862.9
59.5	= 4044.4	.5(4044.4) = 2022.2
60.0	= 4369.2	.5(4369.2) = 2184.6

Storage Provided = 10734 Cu. Ft. 10734.1

Pond R2

51.0	= 542.8	0(542.8) = 0
51.5	= 665.7	.5(665.7) = 332.9
52.0	= 783.3	.5(783.3) = 391.7
52.5	= 974.3	.5(974.3) = 487.15
53.0	= 1201.0	.5(1201.0) = 600.5
53.5	= 1418.0	.5(1418.0) = 709.0
54.0	= 1641.9	.5(1641.9) = 820.95
54.5	= 1879.6	.5(1879.6) = 939.8
55.0	= 2119.2	.5(2119.2) = 1059.6

5341.7

Storage provided = 5341.7 Cu. Ft.

Total Storage Provided = 16076 Cu. Ft.

Common 1 Pond C1

ELEV

sq ft

Cu. Ft

54.5	=	196	=	96
55.0	=	270	=	135
55.5	=	354	=	177
56.0	=	451	=	226
56.5	=	560	=	280
57.0	=	680	=	340
57.5	=	811	=	406
58.0	=	954	=	477
58.5	=	1109	=	555
59.0	=	1272	=	636
59.5	=	1443	=	722
60.0	=	1617	=	808

4869 cu ft

Total Storage = 4869 cu ft.

APPENDIX 'D'
FEMA

To view a copy of the Flood Insurance
Rate Map in regards to this
development
please contact the Community
Development/Planning Division
at 887-2180

Water System Analysis

CONCEPTUAL WATER SYSTEM ANALYSIS

for

MILLS LANDING

Prepared for:

**Landmark Homes
985 Damonte Ranch Pkwy #300
Reno, Nevada 89521**

Prepared by:



CAPITAL ENGINEERING

P.O. Box 3750

(775)882-5630 Fax(775)885-7282

Carson City, NV 89702

October 2006

INTRODUCTION

The Mills Landing at State Street project is approximately 10.1 acres in size and is proposed to consist of approximately 99 residential units located along State Street, and 3 commercial buildings located along U.S. Highway 50 in Carson City NV. (See figure 1.) The water facilities plan will incorporate connections to the existing 8" water main at two existing street locations in State Street (verify 8", not 6"). There will be a connection for the commercial development to the 12" main in U.S. Highway 50.

The Mills Landing water system was analyzed for the worst case scenario to insure that the proposed water system would meet pressure and velocity requirements in accordance Carson City Standards.

EXISTING SYSTEM

As presented on Figure 2, the existing water system consists of one 8" waterline (verify), located in State Street that will be used as a connection point for the proposed residential portion of the development, and a 12" water main that will be used for the commercial portion of the development. This water main is located in U.S. Highway 50 and runs across the front of the proposed commercial property. A double check valve will be required at this location.

The flow test fire hydrants are located on State Street near the corner of State Street and Long Street in Carson City. The flow test data for both of these hydrants was provided by Carson City Utilities and has been included in the appendix for reference. The information for the existing water mains, shown on Figure 2, was obtained from the Carson City Water Maps (page number H11).

PROPOSED SYSTEM

As presented in Figure 2, the infrastructure to be added for Mills Landing at State Street will be 8" water distribution mains. The 8" water main connections to the existing system will be at locations as mentioned above.

The flow tests fire hydrant data is included in the back of this report. From that data, there is an average flow of 3276 gpm at 20 residual psi. The actual test pumping was approximately 1300 gpm with a residual pressure of 85 psi. Static pressure was at 100 psi.

1300 gpm will be utilized to provide conservative residual fire flows, therefore, the calculations for the residential portion allow for an 8" water main throughout the project that connects to the existing 8" water main in State Street. The highest point on the property is 6.9 ft above the existing water main. This is a loss of about 3 psi at that point due to elevation.

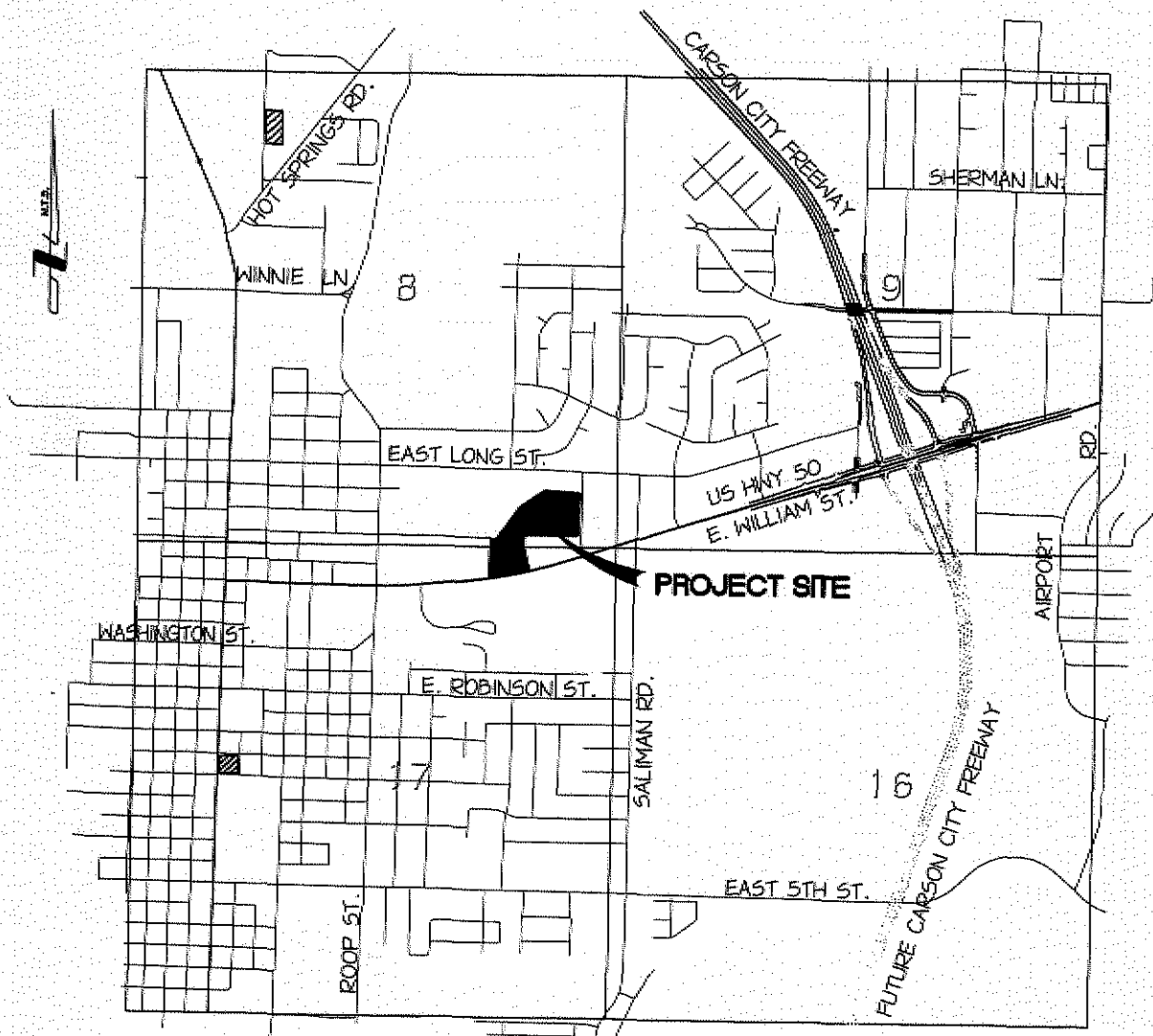
The pipe length to that point is approximately 580 ft of 8" PVC (conservatively assume a single line). Using the Hazen - Williams Equations and Charts, 1300 gpm over 580 ft of PVC produces a friction loss of about 5.5 psi.

The total psi lost through elevation change and pipe length is approximately 8.5 psi for 1300 gpm. The residual pressure of the tested fire hydrant was 85 psi at approximately 1300 gpm. The loss of 8.5 psi will result in a residual pressure of 76 psi at the highest point fire hydrant. This is well above the minimum of 20 psi.

The commercial portion of the project has significantly less distance and less elevation change from the existing 12" water main currently in U.S. Highway 50 to the highest fire hydrant point. Assuming similar flows and pressures in the 12" line as were in the 8" line, the available fire flow of 3267 cfs at 20 psi residual is well in excess of the 1500 gpm required at the commercial site. Total losses at the commercial site will be less than 10 psi for 1500 gmp.

SUMMARY

The analysis of the existing water system with the addition of the proposed Mills Landing development shows that the infill development is located in a well supplied, high pressure area of the Carson City Water System. The project will provide a safe and adequate water system to supply both the residential and commercial portions of the project.



Prepared By:

 **CAPITAL ENGINEERING**

P.O. Box 3750
 Carson City, NV 89702
 (775) 882-5630

**FIGURE 1
 VICINITY MAP
 MILLS LANDING**

APPENDIX "A"

Calculations

PROP - WATER

- From Fire Hydrant Test, have 1300 + gpm available at
Residual Pressure of 85 ps.

$$\text{Pipe Length} = 580'$$

$$\text{ELEV Change} = 6.9'$$

$$\frac{6.9}{2.31} = 2.99 \approx 3 \text{ psi lost in 6.9 ft of FLOW PIPE}$$

- sum of all 1300 gpm within 8' pipe

$$\text{Pipe Length} = 580'$$

$$580' \text{ is } 58\% \text{ of } 1000'$$

$$h_L = 18/1000' = 1.8' \text{ at } L=1000' \quad \left\{ \begin{array}{l} \text{From Hydrant} \\ \text{within} \end{array} \right.$$

- for FVC is higher by 1.46

$$h_{L_T} = 1.46 (.46 \times 100) \times .58 = 12.8'$$

$$h_{L_T} = 12.8'$$

- Again divide by 2.31

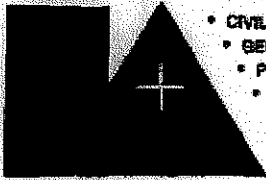
$$\frac{12.8}{2.31} = 5.5 \text{ psi}$$

3 psi + 5.5 psi = 8.5 psi lost due to
Pipe Length and ELEV. (Call + 9 psi)

$$85 \text{ psi} - 9 \text{ psi} = 76 \text{ psi}$$

APPENDIX "B"

Flow Test



- CIVIL ENGINEERING
- GEOTECHNICAL ENGINEERING
- PLANNING
- LANDSCAPE ARCHITECTURE
- SURVEYING • GIS
- CONSTRUCTION SERVICES
- MATERIALS TESTING

LUMOS AND ASSOCIATES, INC.

September 29, 2005

Carson City Development Services
 2621 Northgate Lane
 Carson City, NV 89706-1319
 Attn: Rory Hogen

Re: **Fire Flow Test and Requirements for 5201 Convair Drive,**
Carson City, Nevada *PERMIT 04-1475*

Dear Rory:

This letter is presented to verify that the fire supply water proposed for the above named project complies with applicable standards and code requirements.

Fire flow tests were conducted by Carson City on the fire hydrant located approximately 125' west of the project. The test was performed on June, 25, 2005. This test indicates an available fire flow of 3,049 gpm at 20 psi residual pressure. The proposed building is to be equipped with an internal fire suppression system; therefore, we have assumed the required fire flow to be one half of the 3,750 gpm specified by the Carson City Fire Department for an unsprinklered building. Based on the above test, the available fire flow exceeds the required fire flow and meets the Carson City Fire Marshall's requirements.

If there are any further questions, please don't hesitate to call.

Sincerely,

358
 Tony Siniraglia, E.I.
 Project Designer

Randall Long

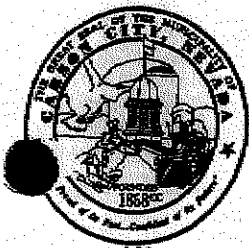
 Randall Long
 Location Principal
12.31.05

Enclosures

Carson City
 800 E. College Parkway
 Carson City, NV 89706
 775.885.7577 • Fax 775.883.7114
 e-mail: cc@lumosengineering.com

Reno
 5401 Longley Lane, Suite 5
 Reno, NV 89511
 775.827.6111 • Fax 775.827.6122
 e-mail: reno@lumosengineering.com

Fallon
 178 S. Main Street
 Fallon, NV 89406
 775.423.2181 • Fax 775.423.5657
 e-mail: info@lumosengineering.com



CARSON CITY, NEVADA

CONSOLIDATED MUNICIPALITY AND STATE CAPITAL

WATER UTILITY DIVISION

FIRE FLOW DATA SHEET

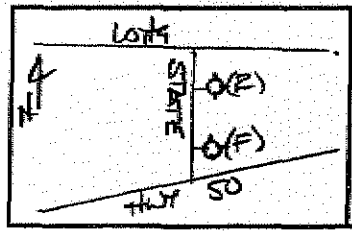
TESTING PERSONNEL: M BRETHAUER / C WELCH
 DATE OF TEST: 2/13/97 TIME OF TEST: 10 Am
 REQUESTED BY: HAUSER & LAUDER

TEST LOCATIONS: (Street & Cross Street or Address)

STATE & Lake

COMMENTS: _____

MAINLINE SIZE: 6"
 PRESSURE: Static (S) 100 PSI
 Residual (R) 85 PSI
 Pitot (P) 64 PSI



LOCATION MAP

EXIT COEFFICIENT (C) .9 EXIT DIAMETER (INCHES) (D) 2.5

Q = FLOW QUANTITY FROM HYDRANT
 $Q = (29.83) \times (C) \times (D^2) \times (\sqrt{P})$
 $Q = (29.83) \times (.9) \times (.625) \times (8)$
 $Q_1 = 1342.3$ Gallons Per Minute

AVAILABLE WATER CALCULATION

$$D_1 = (S) - (R)$$

$$D_1 = 100 - 85 = 15$$

$$Q_1 = Q_2 \sqrt{D_2 / D_1}$$

$$Q_1 = 1342 \sqrt{80 / 15}$$

$$D_2 = (S) - 20 \text{ PSI}$$

$$D_2 = 100 - 20 \text{ PSI} = 80$$

$$Q_2 = 3099.2$$

IF $Q_2 \geq Q_1$, THEN $Q_{AV} = [(Q_2 - Q_1) \cdot .1] + Q_1$

$$Q_{AV} = [(3100 - 1342) \cdot .1] + 1342 = 3275.8$$

$Q_{AV} = 3276$ G.P.M. = Total Available Water At 20 PSI Residual

UTILITIES DEPARTMENT

Environmental Control Authority • 3300 Butti Way, #7 • 89701 • (702) 887-2340
 Wastewater Reclamation Plant • 3320 E. 5th Street • 89701 • (702) 887-2360
 Utility Billing • 2621 Northgate Lane, #66 • 89706 • (702) 887-2370
 Sewer Utility • 3300 Butti Way, #7 • 89701 • (702) 887-2340
 Water Utility • 3300 Butti Way, #8 • 89701 • (702) 887-2355

~~§~~
§

Sewer System Analysis

PRELIMINARY SANITARY SEWER SYSTEM ANALYSIS

FOR

MILLS LANDING

**Prepared For:
Landmark Homes**

Prepared By:



CAPITAL ENGINEERING

P.O. Box 3750

(775)882-5630 Fax(775)885-7282

Carson City, NV 89702

OCTOBER 2006

INTRODUCTION

The Mills Landing at State Street project is approximately 10.1 acres in size and is proposed to consist of approximately 94 residential units located along State Street, and 3 commercial buildings located along U.S. Highway 50 in Carson City NV. (See figure 1.) The water facilities plan will incorporate connections to the existing 8" water main at two existing street locations in State Street (verify 8", not 6"). There will be a connection for the commercial development to the 12" main in U.S. Highway 50.

The Mills Landing water system was analyzed for the worst case scenario to insure that the proposed water system would meet pressure and velocity requirements in accordance Carson City Standards.

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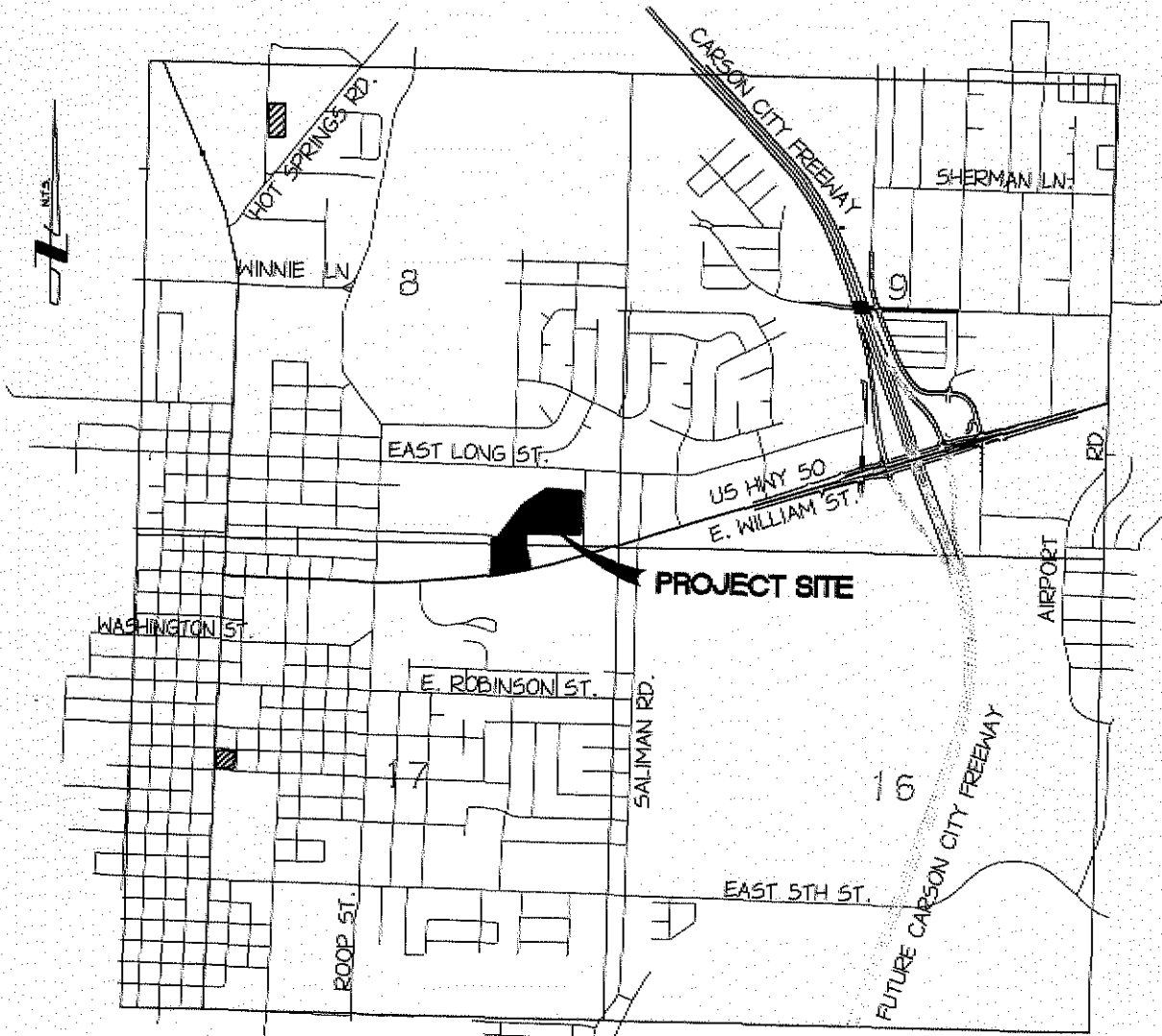
The flow tests fire hydrant data is included in the back of this report. From that data, there is an average flow of 3276 gpm at 20 residual psi. The actual test pumping was approximately 1300 gpm with a residual pressure of 85 psi. Static pressure was at 100 psi.

1300 gpm will be utilized to provide conservative residual fire flows, therefore, the calculations for the residential portion allow for an 8" water main throughout the project that connects to the existing 8" water main in State Street. The highest point on the property is 6.9 ft above the existing water main. This is a loss of about 3 psi at that point due to elevation.

The pipe length to that point is approximately 580 ft of 8" PVC (conservatively assume a single line). Using the Hazen - Williams Equations and Charts, 1300 gpm over 580 ft of PVC produces a friction loss of about 5.5 psi.

The total psi lost through elevation change and pipe length is approximately 8.5 psi for 1300 gpm. The residual pressure of the tested fire hydrant was 85 psi at approximately 1300 gpm. The loss of 8.5 psi will result in a residual pressure of 76 psi at the highest point fire hydrant. This is well above the minimum of 20 psi.

APPENDIX 'A'



Prepared By:

 **CAPITAL ENGINEERING**

P.O. Box 3750
 Carson City, NV 89702
 (775) 882-5630

**FIGURE I
 VICINITY MAP
 MILLS LANDING**

Residential

Assume Peak Flow of 250 GPD

Pipe Radius = 8"

94 houses at 250 = 23500 GPD - Avg

$$\left(\frac{23500}{1 \text{ day}} \right) \left(\frac{1 \text{ day}}{86400 \text{ sec}} \right) \left(\frac{1 \text{ cfs}}{7.48} \right) = 0.36$$

$$\text{Flow from pipe} = 0.36 \text{ cfs} \times 3.5$$

(Peaking factor)

$$= 1.26 \text{ cfs}$$

Assume pipe is half-full, using Mannings Equation, find cfs for half-full pipe,

Check to make sure if half-full pipe will work, then if less than or equal, we know it will work

$$Q = A \times \frac{1.486}{1010} \times R^{4/3} \times S^{1/2}$$

$$A = \pi r^2 = 3.14 \times 3.3^2 = \frac{35}{2} \text{ ft}^2 = 17.5$$

$$R = \frac{1}{2} (\pi \cdot D) = 1.05$$

$$R = 0.34 / 1.05 = 0.33$$

$$S = 0.004 \text{ ft/ft}$$

$$Q = .175 \times 148.6 \times .467 \times 1.0633$$

$$Q = .150 \text{ cfs}$$

15 cfs is much greater than .133 cfs

The pipe will only be 12% full and will have no problem carrying the flow.

Commercial

Assume Avg Commercial flow of $10,000 \frac{\text{GPD}}{\text{Ac}}$
Pipe Size = 8"

$$2.43 \text{ ac} \text{ at } 1000 \text{ GPD} = 2430 \text{ GPD}$$

$$\frac{2430}{1 \text{ hr}} \left(\frac{1 \text{ day}}{24 \text{ hr}} \right) \left(\frac{1 \text{ cut}}{7.4 \text{ days}} \right) = 1003 \text{ cfs}$$

Average flow of 1003 cfs with a peaking factor of 3.5

$$1003 \text{ cfs} \times 3.5 = 3510 \text{ cfs}$$

Similar Assumption as Res 4-101.

$$Q = A \cdot \frac{148.6}{1010} \times R^{2/3} \times S^{1/2}$$

$$A = .175 \text{ ft}^2$$

$$R = .33 \text{ ft}$$

$$S = .004 \text{ ft/ft}$$

$$Q = .175 \times 148.6 \times .417 \times .0033$$

$$Q = .50 \text{ cfs}$$

150 cfs is more than 100 cfs,
This is a minimal increase on
the existing sewer