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GEOTECHNICAL INVESTIGATION REPORT
COLD SPRINGS
WASTEWATER TREATMENT PLANT UPGRADE
WASHOE COUNTY, NEVADA

KLEINFELDER
An employee owned company

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WASHOE COUNTY, NEVADA

August 22, 2003

KLEINFELDER
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File: 33247.01

Kennedy/Jenks Consultants
2828 SW Nation Parkway, Suite 250
Portland, Oregon 97201

Attention: Mr. Harry Ritter, P.E.

SUBJECT: Geotechnical Investigation Report
Cold Springs Wastewater Treatment Plant Upgrades
Washoe County, Nevada

Dear Mr. Ritter:

The attached report presents the results of our geotechnical investigation for the proposed Cold Springs Waste Water Treatment Plant Upgrades to be constructed at the existing treatment plant in Washoe County, Nevada. Additional improvements include the construction of approximately 8,300 linear feet of force main along Diamond Peak Drive, Cold Springs Drive, and Village Parkway and a new lift station immediately south of the Diamond Peak Lift Station, located at the intersection of Diamond Peak Drive and Spicer Lake Court. The purpose of this investigation was to evaluate the feasibility of the proposed construction with respect to the observed subsurface conditions and to provide geotechnical recommendations for project design. Our work consisted of subsurface exploration, laboratory testing, engineering analyses, and report preparation.

Based on our review of the *Reno NW Quadrangle Earthquake Hazards Map* (Szecsody, 1983) and University of Nevada-Reno aerial photos, an active section of the Peterson Mountain fault zone crosses the force main alignment at approximately the intersection of Diamond Peak Drive and Spicer Lake Court. The Peterson Mountain fault zone is estimated to be capable of generating an earthquake of moment magnitude 7.0 (dePolo, et al., 1997). The estimated fault location relative to the pipeline alignment is shown on the Plate 4 in Appendix A of the attached report. If the new lift station is deemed as a "critical" structure, additional exploration is recommended to locate the Peterson Mountain fault with regards to the proposed improvements.

We appreciate this opportunity to be of service to you, and look forward to future endeavors. If you have any questions regarding this report or need additional information or services, please feel free to call one of the undersigned in our Reno office.

Sincerely,

KLEINFELDER, INC.

MARK DOEHRING
12-31-04
CIVIL

8-22-03

Mark Doehring, P.E.
Senior Engineer

Mike Klein

Mike Klein, P.E.
Geotechnical Department Manager

MD:MK:tg

Enclosures: Report (8 Bound, 1 Unbound)

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EXECUTIVE SUMMARY

The following paragraphs summarize some of the primary findings and recommendations. The report should be read in it's entirety.

- A review of the *Mt. Rose NW Quadrangle Geologic Map* (Soeller, et al., 1980) indicates that the treatment plant upgrades are underlain by Flood-plain deposits (Qs) of the Quaternary period. This unit is described as moderately to well sorted fine to very fine sand and sandy clay and mud. The force main crosses four geologic units of the Quaternary period including Flood-plain deposits (Qs) and Beach deposits (Lakeshore deposits (Qb), Forebeach deposits (Qfb), and Lake-floor deposits (Ql). The Beach and Forebeach deposits are predominately granular, medium to coarse sand, sandy pebble gravel. The Lake-floor deposits consist of very thin-bedded clay and sandy mud. The project site and force main alignment are shown with respect to the geologic map on the Plate 3 in Appendix A of the attached report.

- The conditions encountered during our field investigation are in general agreement with the geologic map. We encountered intercalated layers of silt and sand with varying amounts of clay and silt fines. The granular soils generally had a relative density of medium dense to very dense; the fine-grained soils had a consistency of stiff to hard. The exceptions were layers of loose silty sand encountered in borings B8 and B13 at depths of approximately 2 feet and 11 feet, respectively. Heaving sand was encountered in borings B1, B2, and B5 at a depth of approximately 45 feet bgs and at a depth of 15 feet in borings B13 and B14. (Borings B1, B2, and B5 were advanced just west of the existing treatment plant. Borings B13 and B14 were advanced just south of the Diamond Peak Lift Station.)

currently 37'-42'

& increasing (TM 008)

- During our field investigation, groundwater was encountered at depths ranging from 37 to 42 feet below ground surface (bgs) near the existing treatment facility and at depths of eight to nine feet near the Diamond Peak lift station. The borings for the new lift station (B13 and B14), were drilled on July 25 and July 28, 2003 and completed as monitoring wells. Slug tests were performed in these wells on August 13, 2003. At the time of our field testing, groundwater was measured at approximately 5½ feet bgs. The geotechnical investigation for Village Parkway by Pezonella Associates, Inc. (2001) indicates groundwater was encountered at a depth of four feet bgs at the southern end of the alignment. Groundwater is anticipated to intercept construction of the force main and lift station near the intersection of Diamond Peak Drive and Spicer Lake Court. Construction dewatering should be accomplished in such a manner that will preserve the strength of the foundation soils, will not cause instability of trench and building excavations, and will not result in damage to the existing, surrounding improvements. Fluctuations in groundwater levels and soil moisture contents may occur due to variations in precipitation, land use, irrigation, and other factors.

deepest structure ~20', so are assuming no Groundwater condition for permanent facilities. GW depths are rising... any risk in long-term?

RW

- A review of the *Flood Insurance Rate Map* (1994) indicates that the majority of the force main alignment and lift station are located within the 100-year floodplain, Zone A. (A Zone A designation indicates that the site is within the 100-year floodplain, but no base flood elevation has been determined.) The upgrades to the wastewater treatment plant are located outside of the 500-year floodplain.

- The project site is located in UBC Seismic Zone 3, a relatively active seismic area. Based on our review of the *Reno NW Quadrangle Earthquake Hazards Map* (Szecsody, 1983) and University of Nevada-Reno aerial photos, an active lineament/fault of the Peterson Mountain fault zone crosses the force main alignment roughly at the intersection of Diamond Peak Drive and Spicer Lake Court. The Peterson Mountain fault zone is estimated to be capable of generating an earthquake of moment magnitude 7.0 (dePolo, et al., 1997). The approximate pipeline alignment with regards to the earthquake hazards map (Szecsody, 1983) is shown on the Plate 4, Appendix A.

- The Peterson Mountain fault was trenched north of Cold Springs Road (Kleinfelder, 1994) and confirmed to be active. A geotechnical investigation by Summit™ Engineering Corporation (2002) for Parcel APN 566-01-08, located east of Diamond Peak states, "A fault location trench was excavated near the southwest corner of the site to determine if this fault crossed the site. Our exploration indicated that this fault did not cross the site." This indicates that the fault is most likely located on the west side of Diamond Peak Drive crossing through the Lake Hill Subdivision, located immediately north of the Diamond Peak Lift Station. However, a report by Pezonella Associates, Inc. (1998) indicates that the Lake Hill Subdivision, Units 1, 2, and 3 was trenched by Summit Engineering Corporation in 1997 and the "presence of the suspected fault was not confirmed..."

- If the pipeline and lift station are deemed "critical" structures, inconsistencies between the referenced geotechnical reports, aerial photographs, and hazards map will need to be resolved. Additional exploration would be required to locate the Peterson Mountain fault with regards to the proposed improvements. If the force main is deemed to cross the fault trace, the final pipeline design could incorporate mitigations against possible ground movement such as flexible connections for the pipeline section crossing the fault or free space to allow for lateral and vertical displacement of the pipeline during a seismic event. A set-back would need to be determined for the new lift station. The *Guidelines for Evaluating Potential Surface Fault Rupture/Land Subsidence Hazards in Nevada* (Price, 1998) states, " Set-back from faults and fissures. State, Federal or local guidelines may dictate minimum standards otherwise, the minimum set-back for occupied structures for Holocene active faults shall be fifty (50) feet. Furthermore, no critical facility shall be placed directly over the trace of a Late Quaternary active fault (a fault that has moved in the last 130,000 years)."

- The project site also lies within the zone of influence of numerous other fault systems in Truckee River Basin, western Nevada, and eastern California. According to the *Reno NW Quadrangle Earthquake Hazards Map (Szecsody, 1983),* upgrades to the existing treatment plant and the majority of the force main alignment are located in an area, which would likely experience moderate severity of shaking (Level III) during a seismic event. As shown on Plate 4 (Appendix A), the southern section of the force main alignment, along Diamond Peak Drive, Cold Springs Drive, and approximately 1,200 linear feet of Village Parkway, crosses areas which would likely experience moderate severity of shaking (Level II) to the greatest severity of shaking (Level I). (Both Level II and Level III on the hazards map are designated as "moderate" severity.) The proposed lift station is located in an area of greatest severity of shaking, which could possibly experience severe liquefaction locally. An analysis for potential liquefaction, is outside of our current scope of work.

- If seismic loadings are evaluated using the 1997 UBC method, we recommend using a seismic zone factor of 0.3 and a soil profile type S_D. The S_D profile is applicable to a stiff soil profile with an average shear wave velocity of 600 to 1,200 feet/second for the upper 100 feet of the underlying stratigraphy.

- Based on our laboratory test results, the native granular soils meet the requirements for structural fill as outlined in Section 304.03 of the current edition of the *Standard Specifications for Public Works Construction.*

- The proposed structures may be supported on conventional shallow spread or mat foundations bearing on granular soils with low plasticity fines. Exterior wall foundations should be embedded a minimum of 24 inches below finished grade for frost protection and confinement. Interior foundations should be embedded a minimum of 12 inches for confinement.

- Chemical testing for sodium sulfates indicates the site soils have negligible reaction to concrete. Based the laboratory results and the requirements of Section 19, Table 19-A-4 of the 1997 *Uniform Building Code* conventional Type I/II cement may be used for site concrete.

- Resistivity testing indicates that the subgrade soils are potentially corrosive. Non-metal pipelines should be used wherever possible. A corrosion engineer should review resistivity and pH testing results to determined corrosion protection for metal elements.

Specific recommendations for project design and construction are presented in Section 4.0 of the attached geotechnical report.

GEOTECHNICAL INVESTIGATION REPORT
COLD SPRINGS
WASTEWATER TREATMENT PLANT UPGRADES
WASHOE COUNTY, NEVADA

1. INTRODUCTION AND SCOPE

1.1 Project Description

This report presents the results of our geotechnical study for the proposed Cold Springs Wastewater Treatment Plant Upgrades to be located at the existing treatment plant in Washoe County, Nevada. The site location is shown on the attached vicinity map (Plate 1). Design is in the conceptual stage and dead and live loads for the individual structures were not available. Loads presented below were roughly estimated based on our experience with similar structures. Final details on some of the features were not available; however, we understand the upgrades will consist of the construction of the following items:

- Sequential Batch Reactors (SBR) and Aerobic Digester: This facility will consist of two SBR's and one aerobic digester. Each SBR is about 70 feet by 90 feet in plan. These structures will extend about 10 to 15 feet below grade, and above the ground by a similar height. They will be constructed of reinforced concrete. Based on the existing structures at the Cold Springs Wastewater Treatment Plant, we anticipate that structural loads will be on the order of 1,500 psf, supported on a mat foundation.

- Screen and Grit Removal Structures: Two options are being considered for these facilities. One would consist of an at-grade above ground structure, about 20 feet high, 20 feet wide, and 50 feet long. The structure will probably be of CMU construction. The second option would be a below grade structure, extending about 25 feet bgs. The size of this structure would be similar to the at-grade option. Estimated vertical structural loads are not expected to exceed 3,500 psf for external foundations and 2,000 psf beneath the grit chamber.

- Mechanical Sludge Dewatering Facility: This structure, if constructed, would consist of a building about 20 feet wide, 40 feet long, and 22 feet high. It would be of masonry construction and would be constructed at-grade. Estimated vertical structural loads are not expected to exceed 3,500 psf for external foundations and 2,000 psf beneath the chamber, supported on a mat foundation.

- Diamond Peak Lift Station: An existing lift station will be replaced with a new lift station. The new facility, located about 1.5 miles south of the treatment plant, would be constructed adjacent to the existing lift station. The facility would consist of a wet well and a dry well. Each unit would be about 10 feet by 10 feet in plan, separated by about five feet, and extend about 25 feet below grade. The construction would consist of cast-in-place concrete or precast units. Mat foundation are anticipated to support gross dead plus live loads of less than 3,500 psf.

- Diamond Peak Force Main: A force main will extend about 8,300 feet from the Diamond Peak lift station to the treatment plant. It is anticipated the pipe will be about 10 inches in diameter, constructed of HDPE, and be placed at a depth of about four feet below grade.

- Paved Areas: Access and areas adjacent to the new facilities will be paved with asphalt.

1.2 Purpose and Scope of Work

The purpose of this study is to evaluate the feasibility of the proposed development with respect to the observed subsurface conditions, and to provide our geotechnical recommendations and opinions as outlined in our proposal dated June 5, 2003, and summarized below.

- General geology of the site as well as geologic hazards that can be identified from review of the site references; (A detailed liquefaction analysis was outside of our scope of services.)

- General soil conditions at the project site and pipeline alignment, with emphasis on how the conditions are expected to affect the proposed construction;

- Suggested specifications for earthwork construction, including site preparation recommendations, a discussion of reuse of existing near surface soils as structural or non-structural fill, and a discussion of remedial earthwork recommendations, if warranted;

- Recommendations for temporary excavations and trench backfill;

- Conventional shallow spread foundation design including soil bearing values, minimum footing depth, resistance to lateral loads and estimated settlements, and Uniform Building Code seismic site coefficient for use in structural design;

- Preliminary structural sections for asphalt concrete pavement based on a laboratory R-value;

- Lateral earth pressures and drainage recommendations for retaining structures;

- Subgrade preparation for slab-on-grade concrete;

- Potential for site soils to corrode steel, or to adversely react with concrete; and

- Groundwater levels and the potential effects on construction and performance of structures. (Groundwater evaluations are qualitative in nature. We have not included quantitative analysis. We have included a discussion of anticipated general groundwater conditions and dewatering methods. We have not included estimating dewatering quantities or detailed recommendations or design for dewatering systems.)

Our scope of services consisted of background review, site reconnaissance, field exploration, laboratory testing, engineering analyses, and preparation of this report. This study did not include site-specific evaluation of seismicity, faulting (fault trenching), or environmental hazards.

1.3 Authorization

Authorization to proceed with our work on this project was provided by Mr. Travis Tormanen, Project Administrator on June 18, 2003 in the form of a signed Kennedy/Jenks Consultants Inc, Master Services Subcontract Agreement.

1.4 References

The following information was provided to Kleinfelder in the course of this study and serves as the basis of our understanding of the project type and scope.

- "Cold Springs Survey Control," Summit Engineering Corporation, May 30, 2002, Job No. 21417. This drawing was the basis for the site plan shown on Plate 2 of this report.
- "Cold Springs Wastewater Facility Plan," Kennedy/Jenks Consultants, Figure 4, July 2002.
- "Process & Control Structure, Bottom Plan, Cold Spring Valley, Waste Water Treatment Plant," Dewante and Stowell Consulting Engineers, January 1996.
- "Process & Control Structure, Foundation, Top & Roof Framing Plans, Cold Spring Valley, Waste Water Treatment Plant," Dewante and Stowell Consulting Engineers, January 1996.
- "Process & Control Structure, Sections, Cold Spring Valley, Waste Water Treatment Plant," Dewante and Stowell Consulting Engineers, January 1996.
- "Raw Sewage Pump Station, Plans and Section, Cold Spring Valley, Waste Water Treatment Plant," Dewante and Stowell Consulting Engineers, January 1996.

In addition, the following published and unpublished references were reviewed during preparation of this report.

- Bell, J.W. Quaternary Fault Map of Nevada - Reno Sheet. Nevada Bureau of Mines and Geology, 1984.
- Bradburn, C., "Geotechnical Investigation Peavine View Estates, Unit 3, Washoe County, Nevada, Summit Engineering Corporation™, September 20, 1995, File No. 21703.1.
- dePolo, C. M., et al. "Earthquake Occurrence in the Reno-Carson City Urban Corridor." Seismological Research Letters. Volume 68, Number 3, May/June 1997.
- dePolo, C. M., "Local Quaternary Faults and Associated Potential Earthquakes in the Reno and Carson City, Urban Areas, Nevada." Final Technical Report National Earthquake Hazards Reduction Program (NEHRP). Nevada Bureau of Mines and Geology, Contract #1434-95-G-2612, Program Element II.4, 1996.
- Firm, Flood Insurance Rate Map, Washoe County, Nevada and Incorporated Areas. National Flood Insurance Program, Federal Emergency Management Agency, Panel 2800 of 3350, Map Number 32031C2800, September 30, 1994.
- Glynn, J. K. "Geotechnical Investigation Cold Springs Fire Station, Washoe County, Nevada, Summit™ Engineering Corporation, December 2, 2002, File No. 25610.
- Hudson, J. K. "Preliminary Geotechnical and Fault Investigation Report Proposed Peavine View Estates Subdivision, Cold Springs Valley, Washoe County, Nevada," Kleinfelder, Inc., June 23, 1994, 30-2284-01.001.
- Hunter D. "Geotechnical Investigation, Peavine View Estates – Unit 2, Reno, Nevada," SEA, June 17, 1994, Project No. 2508-02-1.
- Pezonella, R. M. "Consulting Engineering Services, Woodland Village Phase 4, Proposed Village Parkway, Washoe County, Nevada," Pezonella Associates, Inc., May 16, 2001, Job No. 4624.01-N.
- Pezonella, R. M., "Geotechnical Investigation, Proposed Lake Hills Subdivision, units 1,2, and 3, Washoe County, Nevada," Pezonella Associated, Inc., June 5, 1998, Job No. 4098.03-A.
- Soeller, S.A., and R.L. Nielsen. Reno NW Quadrangle Geologic Map. Nevada Bureau of Mines and Geology, 1980.
- Szecsody, G.C. Reno NW Quadrangle Earthquake Hazards Map. Nevada Bureau of Mines and Geology, 1983.
- University of Nevada-Reno, Aerial Photos, 1-5, 1-6, 2-6, 2-7, 1:12,000 scale, 0726 am, October 1981, Low-Angle Sun.
- University of Nevada-Reno, Aerial Photo, 1-975, 1:4,000 scale, June 20, 1980.

2. METHODS OF STUDY

2.1 Field Exploration

Our selection of field exploration locations was based on the anticipated project layout and site access. The subsurface exploration consisted of drilling 14 borings in the proposed construction area and pipeline alignment using an auger type drill rig. Boring depths ranged from 11.5 to 51.5 feet below the existing ground surface. Locations of the borings and elevations shown on the Site Plan (Plate 2, Appendix A) were surveyed by Kennedy/Jenks Consultants.

Soil conditions encountered are presented on the boring logs, which are included as Plates 5 through 18. (The project location and force main alignment are shown on the Plates 3 and 4 with respect to the geologic and earthquake hazards maps, respectively.) A description of the Unified Soil Classification System used to identify the site soils and a boring log legend are presented on Plates 19 and 20 (Appendix A).

A field engineer logged the soil conditions exposed in the borings and collected bulk samples and relatively undisturbed driven samples for laboratory testing. Soil samples were obtained by driving a 2-1/2-inch OD Modified California sampler, containing thin brass liners, into the bottom of the boring. The number of blows required to drive the last 12 inches of an 18-inch drive with a 140-pound hammer dropping 30 inches is recorded as the blows per foot (Blow Count) on the boring logs. The blow counts presented on the boring logs represent field blow counts and have not been corrected for sampler type, overburden, hammer type, rod length, etc.

When the sampler was withdrawn from the boring, the brass liners containing the samples were removed, examined for logging, labeled and sealed to preserve the natural moisture content for laboratory testing. After borings were completed, they were backfilled with excavated soil using the equipment at hand. Backfill was loosely placed and not compacted to the requirements typically specified for engineered fill.

2.2 Slug Tests

Two of the borings, B13 and B14, were finished as temporary monitoring wells in accordance with Washoe County District Health Department Permit No. WL030163. Slug tests were performed in each of the wells to assess the hydraulic conductivity of the underlying aquifer within the immediate vicinity of the borings. A Hermit 2000 data logger and 10 psi pressure transducer were utilized during testing. A displacement slug constructed of a 6-foot length of PVC was using to rapidly displace a known volume of water in each of the wells. A 2-inch diameter slug was used in the 4-inch diameter wells.

The "slug out" method was used for all slug tests. Each test was performed by first measuring the static water level followed by installation of a pressure transducer and the displacement slug below static water level. When the water level in the well had recovered to a static condition the slug was quickly removed. The resulting drawdown and recovery of the water level in the well was recorded by the data logger at a logarithmically decreasing rate of at least one data point per second.

Test data were analyzed using the solution method of Bouwer and Rice (1976). This solution was chosen since it accounts for variables such as the degree of partial penetration of the well and other well design factors. For this analysis, we assumed that the wells were fully penetrating. In addition, it was developed for unconfined aquifer conditions such as those encountered at the project site. Plots of water levels versus time with linear fits and calculations are shown in Appendix B. Slug test analysis results are summarized in Table 1.

TABLE 1
MONITORING WELL SLUG TESTING RESULTS

Well Number	Hydraulic Conductivity (ft./day)	Hydraulic Conductivity (cm/sec)	Transmissity[1] (ft^2/day)	Lithology
B13/MW2	71	2.5×10^{-2}	1750	Clayey sand (0'-9') Sand with some silt (9'-35')
B14/MW1	26	9.2×10^{-3}	960	Clayey sand (0'-8') Sand with some silt (8'-45') Sandy silt (45'-50')

Note: 1 Transmissivity assessment based on monitor well full penetration into the aquifer.

2.3 Laboratory Testing

Laboratory testing is useful for evaluating both index and engineering properties of soils. We performed laboratory testing on selected soil samples to assess the following:

- Soil Classification (ASTM D422, D1140, and D4318)
- Unit Weight and Moisture Content (ASTM D2937 and D2216)
- Consolidation (ASTM D2435)
- R-Value (Nevada Test Method T115)

In addition, the following analytical tests were performed by Western Environmental Testing (WET) Laboratory:

- Soluble Sulfate Content
- Resistivity and pH

Individual laboratory test results can be found on the boring logs and on Plates 21 through 27, Appendix A, at the end of this report.

3. DISCUSSION

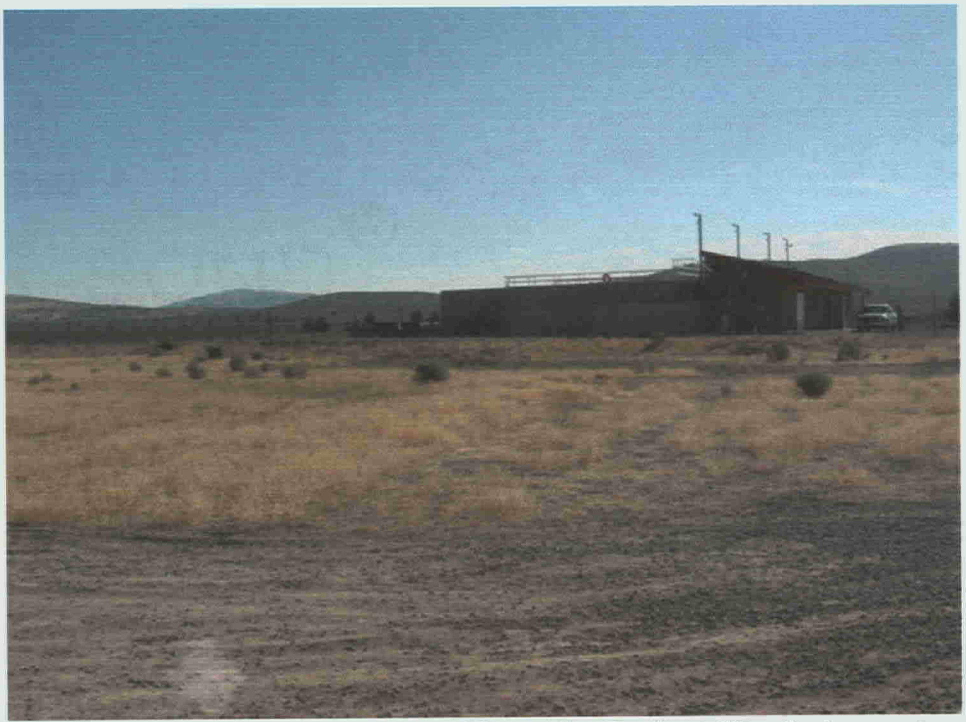
3.1 Site Conditions

The new pump station is located on parcel APN 566-091-25 at the intersection of Diamond Peak Drive and Spicer Lake Court. The new structure will be located approximately 20 feet south of the existing Diamond Peak Lift Station. The site is surrounded by residential housing to the west and north and undeveloped land to the south, and Diamond Peak Drive and undeveloped land to the east. The entire site is fenced with an access gate on Diamond Peak Drive. A photograph of the site taken on August 14, 2003 is provided below.

New Diamond Peak Lift Station Project Site, facing west

The force main alignment travels from the Diamond Peak Lift Station for approximately 1,300 feet north along Diamond Peak Drive, then east along Cold Springs Drive for approximately 600 feet before turning south onto Village Parkway. The alignment then travels for approximately 2,700 feet along Village Parkway and a gravel road (Mud Springs Road) for approximately 3,500 feet before reaching the Cold Springs Wastewater Treatment. Pavements along Diamond Peak Drive, Cold Springs Drive, and Village Parkway are relatively new, having been paved in 1998, 1989, and 2002, respectively.

Upgrades to the Cold Springs Wastewater Treatment Plant basically include the construction of a duplicate plant approximately 50 to 150 feet west of the existing structure. The project site, parcel APN 556-290-04, is undeveloped. Vegetation consisted of a sparse cover of desert brush. The ground surface in the area of the proposed building gently slopes towards the south with a total relief of approximately three foot. Site drainage consists of overland sheet flow down gradient. A photograph of the site taken on August 14, 2003 is provided below.



Cold Springs Treatment Plant Upgrades Project Site, facing east

3.2 Subsurface Conditions

A review of the *Mt. Rose NW Quadrangle Geologic Map* (Soeller, et al., 1980) indicates that the treatment plant upgrades are underlain by Flood-plain deposits (Qs) of the Quaternary period. This unit is described as moderately to well sorted fine to very fine sand and sandy clay and mud. The force main crosses four geologic units of the Quaternary period including Flood-plain deposits (Qs) and Beach deposits (Lakeshore deposits (Qb), Forebeach deposits (Qfb), and Lake-floor deposits (Ql)). The Beach and Forebeach deposits are predominately granular, medium to coarse sand, sandy pebble gravel. The Lake-floor deposits consist of very thin-bedded clay and sandy mud. The project site and force main alignment is shown with regards to the geologic map on Plate 3, Appendix A.

The following paragraphs summarize the results of our field exploration. The boring logs should be reviewed for a more detailed description of the subsurface conditions at the locations explored.

The conditions encountered during our field investigation are in general agreement with the geologic map. We encountered intercalated layers of silt and sand with varying amounts of clay and silt fines. The granular soils generally had a relative density of medium dense to very dense; the fine-grained soils had a consistency of stiff to hard. The exceptions were layers of loose silty sand encountered in borings B8 and B13 at depths of approximately two feet and 11 feet, respectively. Heaving sand was encountered in borings B1, B2, and B5 at a depth of approximately 45 feet bgs and at a depth of 15 feet in borings B13 and B14. (Borings B1, B2, and B5 were advanced just west of the existing treatment plant. Borings B13 and B14 were advanced just south of the Diamond Peak lift station.)

During our field investigation, groundwater was encountered at depths ranging from 37 to 42 feet below ground surface (bgs) near the existing treatment facility and at depths of eight to nine feet near the Diamond Peak lift station. The borings for the new lift station (B13 and B14), were drilled on July 25 and July 28, 2003 and completed as monitoring wells. Slug tests were performed in these wells on August 13, 2003. At the time of our field testing, groundwater was measured at approximately 5½ feet bgs. During the field investigation for Village Parkway by Pezonella Associates, Inc. (2001), groundwater was encountered at a depth of four feet bgs at the southern end of the alignment. Groundwater is anticipated to intercept construction of the force main and lift station near the intersection of Diamond Peak Drive and Spicer Lake Court. Fluctuations in groundwater levels and soil moisture contents may occur due to variations in precipitation, land use, irrigation, and other factors.

3.3 Regional Geology and Faulting

The project site lies within the western portion of the Basin and Range Geomorphic Province. The Basin and Range province was formed by numerous north-south trending normal faults, which displaced to form the horst and graben morphology present throughout most of Nevada. The mountain ranges in western Nevada are primarily composed of Mesozoic or Early Tertiary intrusive and volcanic rocks. The intervening basins consist of deep accumulations of Quaternary age alluvium

The project site is located in UBC Seismic Zone 3, a relatively active seismic area. Based on our review of the Reno NW Quadrangle Earthquake Hazards Map (Szecsody, 1983) and University of Nevada-Reno aerial photos, an active lineament/fault of the Peterson Mountain fault zone crosses the force main alignment roughly at the intersection of Diamond Peak Drive and Spicer Lake Court. (An active fault is a fault with evidence indicating movement during the last 10,000 years (Price, 1998)). The Peterson Mountain fault zone is estimated to be capable of generating an earthquake of moment magnitude 7.0 (dePolo, et al., 1997). The approximate pipeline alignment with regards to the earthquake hazards map (Szecsody, 1983) is shown on the Plate 4, Appendix A.

The Peterson Mountain fault was trenched north of Cold Springs Road (Kleinfelder, 1994) and confirmed to be active. A geotechnical investigation by Summit™ Engineering Corporation (2002) for Parcel APN 566-01-08, located east of Diamond Peak states, "A fault location trench was excavated near the southwest corner of the site to determine if this fault crossed the site. Our exploration indicated that this fault did not cross the site." This indicates that the fault is most likely located on the west side of Diamond Peak Drive crossing through the Lake Hill Subdivision, located immediately north of the Diamond Peak Lift Station. However, a report by Pezonella Associates, Inc. (1998) indicates that the Lake Hill Subdivision, Units 1, 2, and 3 was trenched by Summit ™ Engineering Corporation in 1997 and the "presence of the suspected fault was not confirmed…" The reason for the inconsistencies between the referenced geotechnical reports, aerial photographs, and hazards map is unknown.

The project site also lies within the zone of influence of numerous other fault systems in Truckee River Basin, western Nevada, and eastern California. Should a seismic event occur, the site could be significantly affected by ground shaking. According to the *Reno NW Quadrangle Earthquake Hazards Map* (Szecsody, 1983), upgrades to the existing treatment plant and the majority of the force main alignment are located in an area, which would likely experience moderate severity of shaking (Level III) during a seismic event. As shown on Plate 4 (Appendix A), the southern section of the force main alignment, along Diamond Peak Drive, Cold Springs Drive, and approximately 1,200 linear feet of Village Parkway, crosses areas which would likely experience moderate severity of shaking (Level II) to the greatest severity of shaking (Level I). The proposed lift station is located in an area of greatest severity of shaking, which could possibly experience severe local liquefaction.

4. RECOMMENDATIONS

4.1 Site Clearing and Preparation

The locations of the proposed Cold Springs Wastewater Treatment Plant upgrades (the sequential batch reactors and aerobic digester, screen and grit removal structures, and mechanical sludge dewatering facility) and the new Diamond Peak Lift Station have been previously rough graded; however, a sparse cover vegetation is currently present at each site. (Please, see the site photographs provided in Section 3.1 of this report.) The majority of the force main alignment is located beneath paved roadway sections (Diamond Peak Drive, Cold Springs Drive, and Village Parkway) with the exception of approximately 3,500 linear feet located beneath Mud Springs Road, a gravel road.

Prior to construction, any surface vegetation and organic soils at should be stripped and removed from the site or stockpiled for use in landscape areas as approved by the Owner. It appears four inches can be used as a reasonable estimate for average depth of stripping. Deeper stripping/grubbing of organic soils, roots, etc., may be required in localized areas. The resulting voids backfilled with adequately compacted backfill soil. All man-made debris including structures, pavements, dump fills, and trash should be removed from the site.

The geotechnical engineer should be present during stripping and site preparation operations to observe stripping and grubbing depths, and to evaluate whether buried obstacles such as underground utilities, are present. Special care should be exercised in evaluating whether loose utility backfills exist which could adversely affect the planned structures. Excavations resulting from removal operations should be cleaned of all loose material and widened as necessary to permit access to compaction equipment.

Dust control will be the responsibility of the contractor. A dust control plan should be prepared by the owner, civil engineer, or contractor prior to the start of grading.

4.2 Earthwork

4.2.1 General Site Grading

Site preparation and grading should conform to the requirements contained in this report and in the suggested specifications, which are provided as Appendix C of this report. We anticipate that site grading can be performed with conventional earthmoving equipment.

Where fill is necessary, materials should meet the gradation and plasticity requirements listed for "structural fill" in Appendix C. It appears that the existing site soils will generally be capable of meeting recommended requirements for structural fill. Exceptions include near surface silt/clay layers encountered in borings B1, B3, B5, and B12 at a depth of approximately four feet bgs and at the surface in borings B9 and B10.

Fill placement and compaction requirements presented in Appendix C should be followed. Prior to fill placement, the exposed native soils should be scarified to a minimum depth of six inches, moisture conditioned as necessary, and compacted to a minimum of 90% relative compaction in accordance with the ASTM D1557 compaction test method.

No fill material should be placed, spread, or rolled on frozen subgrade. Areas to receive fill should be blanketed with a layer of loose fill (typically four to six inches thick) at the end of each workday to protect the ground from freezing. No fill soils should be moisture conditioned or placed when the atmospheric temperature is below 35 degrees Fahrenheit.

4.2.2 Temporary Unconfined Excavations

We understand that deep cuts of up to 15 and 25 feet are proposed to construct of the sequential batch reactors and screen and grit removal structures as part of the upgrades to the Cold Springs Wastewater Treatment Plant. In additional, excavations for the construction of the new Diamond Peak Lift Station are anticipated to extend approximately 25 feet bgs.

The use of steepened, temporary cut slopes will be needed to construct below grade structures. Excavations for upgrades at the treatment plant and lift station should comply with current OSHA safety requirements for Type C soils, with maximum inclinations of 1½:1 (horizontal to vertical). The above layback assumes complete dewatering of the soils and is a suggested guideline, which may require modification in the field after the start of construction.

The stability of slopes below the groundwater table will be a function of the method and degree to which the soils are dewatered. The contractor is ultimately responsible for the safety of workers and should strictly observe federal and local OSHA requirements for excavation shoring and safety. Due to the granular nature of the surface soils, some ravelling of temporary cut slopes should be anticipated. During wet weather, runoff water should be prevented from entering excavations.

4.2.3 Temporary Trench Excavation and Backfill

It appears that temporary confined excavations for subject project can be readily made with either a conventional backhoe or excavator. We understand that the force main will be placed at a depth of about four feet below grade. We expect that the utility trench walls will stand nearly vertical without significant sloughing in the areas north of Cold Springs Drive. The excavations south of Cold Springs Drive, particularly south of Spicer Lake Court, are anticipated to become unstable due to saturated conditions and flowing sand. The need for shoring or sloping of trench walls to protect personnel and provide temporary stability should be anticipated. Confined excavations within the native soils should comply with current OSHA safety requirements for Type C soils (Federal Register 29 CFR, Part 1926).

The contractor is ultimately responsible for the safety of workers and should be evaluated to verify their stability prior to occupation by construction personnel. All excavations should strictly observe federal and local OSHA requirements for excavation shoring and safety.

During wet weather, runoff water should be prevented from entering excavations. Water should be collected and disposed of outside the construction limits. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance of one-third the slope height from the top of any excavation.

The native soils are anticipated to generally meet the Class E backfill criteria as outline in Section 200.03.06 of the *Standard Specification for Public Works Construction* (2001) sponsored by Washoe County. Backfills for trenches or other excavations within pavement areas, beneath slabs, and adjacent to foundations should be compacted in six- to eight-inch layers with mechanical tampers. Jetting and flooding should not be permitted. We recommend all backfill be compacted to a minimum compaction of 90% of the maximum dry density as determined by ASTM D1557. The moisture content of compacted granular backfill soils should be within two percent of optimum. Poor compaction in utility trench backfill may cause excessive settlements resulting in damage to the pavement structural section or other overlying improvements. Compaction of trench backfill outside of improvement areas should be a minimum of 85% relative compaction.

As an alternative, slurry backfill could be used in lieu of bedding and pipe zone backfill. If used, slurry backfill should meet the requirements outlined in Section 202.02 of the *Standard Specifications* (2001).

For any utilities placed in existing Washoe County streets, the street has to be repaired to the satisfaction of the County Engineer. As a minimum, this requires fill depth removal and replacement of asphalt for half the width, or replacement of the pavement with a non-woven reinforcing fabric with a 2-inch asphalt overlay for half the street width. Type II slurry seal is required for the entire width of the street. Full width street improvements may be required if the proposed utility location is located too close to the centerline of the existing street.

4.2.4 Pipe Thrust Blocks

Pipe thrust blocks may be designed using an equivalent fluid passive pressure of 350 pcf. Concrete should be placed directly on undisturbed, native soil under the direction of a qualified field inspector.

4.2.5 Construction Dewatering

During our field investigation, groundwater was encountered at depths ranging from eight to nine feet near the Diamond Peak Lift Station. During field slug tests in borings B13 and B14, groundwater was measured at approximately 5½ feet bgs. We understand the lift station will extend up to 25 bgs.

Dewatering will be required so that free water does not interfere with construction. To prevent unstable trench wall conditions and to provide a firm, unyielding subgrade for construction, groundwater should be lowered about two feet below the bottom of the excavation and below any utility excavations.

The dewatering system should be a contractor-designed system. Control of groundwater should be accomplished in such a manner that will preserve the strength of the foundation of soils, will not cause instability of excavated slopes, and will not result in damage to existing structures. Where necessary, the water should be lowered in advance of any excavation by deep wells, well points, or other methods.

Our slug test results indicated that the site soils at the lift station have relatively high hydraulic conductivity values ranging from 2.5×10^{-2} to 9.2×10^{-3} (cm/sec). Based on conversations with Mr. Gary Rambosek of Department of Public Works, Engineering Department, County of Washoe, we understand that dewatering was successfully completed for the existing the sewer line at Diamond Peak and Spicer Lake Court using two 24-inch dewatering wells that were left in-place. Mr. Rambosek also stated that he believed at least 40 well points were used to lower the groundwater table during the construction of the original Diamond Peak Lift Station. It should be noted that heaving sand (clean sands that flowed vertically into the drill stem) was encountered during our investigation of the site for the proposed lift station.

Open pumping should not be permitted if it results in boils, loss of fines, unacceptable settlement of existing structures, or causes construction slope instability. Water should not be allowed to pool and remain in the excavated area over an extended period of time. General lowering of the groundwater can result in settlement of nearby structures. This should be taken into account in the contractor's design of the dewatering system. Any nearby structures should be monitored for settlement and any signs of instability during dewatering operations. It may be desirable to examine the cost-effectiveness of using sheet piling to limit the effects of dewatering on the surrounding improvements. The majority of the surrounding improvements, including residential structures were not in-place during dewatering for the construction of the initial lift station.

Discharge should be arranged to meet the necessary local governmental requirements. Discharge should be arranged to facilitate sampling by the engineer of record.

4.2.6 Subgrade Stabilization

Soft subgrade conditions should be anticipated in the bottom of excavation for the pump station and utility trenches, which extend below or near the groundwater surface. These soils may be unstable and deflect (pump) under construction equipment loads. Saturated, pumping subgrade materials will not be suitable for placement of structural fill or structures and will need to be stabilized. Over-excavation and placement of drain rock or similar materials in conjunction with geogrid or geotextile should be included in the construction documents. For preliminary planning purposes we recommend a minimum depth of 18 inches of drain rock with geogrid (Tensar^{TM} BX1200 or equivalent) placed on the subgrade and mid-center of the rock layer. Individual sheets of geogrid should overlap by at least 12 inches. Light, track-mounted construction equipment should be anticipated in excavations for the pump station to help prevent destabilizing the subgrade soils and causing "pumping" conditions.

4.3 Foundations

4.3.1 General

The proposed structures may be supported by conventional spread footings and/or mat foundations bearing on non-expansive native soil or compacted imported fill. Any loose soil in the bottom of footing excavations should be recompacted to at least 90% relative compaction or removed to expose firm, unyielding material. The design engineer should provide reinforcing steel requirements for foundations.

The allowable bearing capacities provided in the sections below may be increased by one-third for total loading conditions, including wind and seismic forces. The allowable bearing pressures are net values; therefore, the weight of the foundation and backfill may be neglected when computing dead loads.

The site is located in UBC Seismic Zone3. If seismic loadings are evaluated using the 1997 UBC method, we recommend using a seismic zone factor of 0.3 and a Soil Profile Type S_D, as outlined in Tables 16-I and 16-J of the UBC.

4.3.2 Sequential Batch Reactors and Aerobic Digester (above the groundwater table)

The sequential batch reactors and aerobic digester can be supported on a mat foundation. In order to limit post-construction settlement to less than 1-inch, mat foundations may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot for dead loads plus long-term live loads.

We recommend that SBRs and aerobic digester be filled to their design water levels prior to hooking up utilities/plumbing to allow immediate settlement beneath the structures to occur.

4.3.3 Screen and Grit Removal Structures and Mechanical Sludge Dewatering Facility (above the ground water table)

Two options are being considered for the screen and grit removal structures. One would consist of an at-grade above ground structure. The second option would be a below grade structure, extending about 25 feet below grade. The mechanical sludge dewatering facility will be constructed at-grade.

For the structures at-grade, foundations may be designed for an allowable bearing capacity of 3,000 pounds per square foot for dead and live loads. Exterior foundations should be embedded a minimum of 24 inches below lowest adjacent exterior finish grade for frost protection and confinement. Interior footings should be bottomed at least 12 inches below lowest adjacent finish grade for confinement. Wall foundation dimensions should satisfy the requirements listed in the latest edition of the Uniform Building Code.

For the second option for the screen and grit removal structures (constructed 25 feet below grade), foundations may be designed for an allowable bearing capacity of 4,500 pounds per square foot for dead and live loads.

We estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be less than one inch, with approximate differential settlement of ½ inches or less between adjacent similarly loaded isolated footings.

4.3.4 Lift Station (below the groundwater table)

The foundations for the new lift station will be located beneath the groundwater table. These foundations should be supported on a minimum of 10 inches of compacted gravel over stabilized subgrade. The compacted gravel should be graded gravel, which meets requirements for Class C or Class D backfill listed in Section 200.03 of the Standard Specifications or other material clean gravel approved by the Geotechnical Engineer. The rock should be vibrated into place to a minimum relative compaction of 90 percent or a relative density of 70 percent.

Foundations designed and constructed in accordance with the recommendations of this geotechnical report may be designed for an allowable bearing capacity of 2,500 pounds per square foot.

We estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be less than one inch, with approximate differential settlement of ½ inches or less between adjacent similarly loaded isolated footings.

4.4 Concrete Slab-on-Grade Construction

All concrete floor slabs should have a minimum thickness of four inches. Slab thickness and structural reinforcing requirements within the slab should be determined by the design engineer. Specific design recommendations for floor slabs located above and below the groundwater table are provide in the following paragraphs.

Floor Slabs Above the Water Table (Upgrades to the Cold Springs Wastewater Treatment Plant)

Prior to constructing concrete slabs, patios, sidewalks, or other slabs-on-grade, the upper six inches of slab subgrade should be scarified, moisture conditioned to within 2% of optimum, and uniformly compacted to at least 90% of maximum dry density as determined by ASTM D1557. Scarification and compaction will not be required if floor slabs are to be placed directly on undisturbed compacted structural fill.

At least four inches of Type 2 aggregate base should be placed beneath slab-on-grade floors to provide uniform support. The aggregate base should be compacted to a minimum of 95% relative compaction. We recommend that the base course be placed within three to five days (depending on the time of year) after moisture conditioning and compaction of the subgrade soil. The subgrade should be protected against drying until the concrete slab is placed.

In floor slab areas where moisture sensitive floor coverings are planned, an impermeable membrane should be used to help reduce the migration of moisture vapor through the concrete slabs from external sources (e.g. landscape irrigation). The impermeable membrane should either be a minimum of 8-mil thick polyethylene and protected by two inches of fine, moist sand placed both above and below the membrane or a minimum, or a minimum of 10-mil thick polyethylene and placed beneath the slab subgrade and aggregate base section, protected by two inches of overlying sand. The sand cover should be moistened and tamped prior to slab placement. Care should be taken not to damage the vapor during construction. These recommendations are generic in nature and the manufacture's recommendations for installation and protection of any floor covering should ultimately take precedence.

During the winter months, concrete should be placed and protected in accordance with the recommendations provided in the American Concrete Institute, ACI 306R, *Cold Weather Concreting.*

Floor Slabs Below the Groundwater Table (Diamond Peak Lift Station)

For concrete floor slabs below the groundwater table support should be provided by a 10-inch layer of compacted gravel over stabilized subgrade. The compacted gravel should be graded gravel, which meets requirements for Class C or Class D backfill listed in Section 200.03 of the Standard Specifications or other material clean gravel approved by the Geotechnical Engineer. The rock should be vibrated into place to 90 percent relative compaction or a relative density of 70 percent.

Based on the hydraulic conductivities estimated from field slug tests (Section 2.2) and conversations with Mr. Gary Rambosek of Department of Public Works, Engineering Department, County of Washoe with regards to construction dewatering in the area, we anticipate that a permanent subdrain system for reducing hydrostatic pressure beneath the floor of the lift station is impractical. We anticipate that the structure will need to be waterproofed and designed to resist uplift forces. We recommend using a design static water level of four feet bgs in calculating hydrostatic uplift forces.

4.5 Retaining Structures

Lateral earth pressures will be imposed on all subterranean structures, including retaining walls and foundations. Table 2 presents a list of soil parameters, which we recommend for design of these structures assuming a level backfill above the water table. Table 3 presents a list of soil parameters, which we recommend for the design of structures below the water table with level backfill. Hydrostatic forces will need to be considered for design of the lift station at the intersection of Diamond Peak Drive and Spicer Lake Court. We recommend assuming a design groundwater level of four feet bgs at the lift station.

TABLE 2

LATERAL EARTH PRESSURE COEFFICIENTS

Earth Pressure	Equivalent Fluid Density (pcf)
Active	35
At-rest	55
Passive	350
Friction Coefficient	0.4

Where backfill is placed against structures, we recommend that non-expansive, free-draining materials meeting filter criteria be used in the zone immediately adjacent to the structure to reduce hydrostatic forces. Alternately, the use of pre-manufactured drainage panels should be considered. Furthermore, adequate drainage of the backfill in the form of subdrains and/or weepholes should be provided at the base of the wall.

TABLE 3

LATERAL EARTH PRESSURE COEFFICIENTS WITH HYDROSTATIC PRESSURE

Earth Pressure	Equivalent Fluid Density (pcf)
Active	75
At-rest	85
Passive	215
Friction Coefficient	0.4

Recommended minimum factors of safety against sliding, overturning, and bearing failure are listed in Table 4, below.

TABLE 4

RECOMMENDED MINIMUM FACTORS OF SAFETY

Factor of safety against sliding	1.5
Factor of safety against overturning	2
Factor of safety against bearing failure	3

If both passive and frictional resistances are assumed to act concurrently, we recommend a minimum safety factor of 2 be used for design against sliding.

The at-rest case is applicable for braced walls where rotational movement is confined to less than 0.001H. If greater movement is possible, the active case applies. A wall movement of about 0.01H is required to develop the full pressure.

Lateral pressures computed using the values in Tables 2 and 3 assume that the non-expansive native backfill will extend laterally at least one-half of the wall height. If this condition does not apply, the design values may require revision. This backfill should be compacted to 90% of maximum dry density and within 2% of the optimum moisture content as determined by ASTM D1557. Over-compaction should be avoided, as the increased compactive effort will result in lateral pressures higher than those recommended above. Heavy compaction equipment or other loads should not be allowed in close proximity to the wall unless planned for in the structural design.

4.6 Pavement Sections

Pavement sections below are presented for site development only and are not recommendations for dedicated pavements. The recommended pavement structural sections for the project were calculated using the Caltrans method for flexible pavement design. Structural sections for several traffic loadings and their approximate ESALs are presented in Table 5. Traffic loadings should be verified prior to construction. A minimum R-value of 70 was used to represent the aggregate base material and a minimum R-value of 50 for select subbase. A minimum R-value of 12 was used for the native subgrade.

TABLE 5
PAVEMENT STRUCTURAL SECTIONS

Traffic Index	Approximate ESAL (18 kip single axle load)	Recommended Minimum Structural Section
5.0	7,500	3.5 inches of asphalt concrete 7.5 inches of aggregate base
6.0	34,000	4 inches of asphalt concrete 10 inches of aggregate base or 4 inches of asphalt concrete 4 inches of select subbase 6.5 inches of aggregate base
7.0	123,00	5 inches of asphalt concrete 12 inches of aggregate base or 5 inches of asphalt concrete 4.5 inches of select subbase 8 inches of aggregate base

Placement and compaction procedures for materials and construction should conform to the suggested specifications contained in Appendix C of this report. The sections presented in Table 5 are based on a single R-value test performed on a select sample obtained during our investigation and should be considered preliminary in nature. We recommend verification of soil conditions as construction progresses so that appropriate revisions can be made if necessary.

The pavement structural sections presented in Table 5 are designed for the assumed traffic loadings. However, based on our experience in the Reno area, environmental aspects such as freeze-thaw cycles and thermal cracking will probably govern the life of AC pavements. Thermal cracking of the asphalt pavements allows more water to enter the pavement section, which promotes deterioration and increases maintenance costs. A yearly maintenance program of asphalt concrete crack sealing is recommended.

4.7 Site Drainage

Final elevations at the site should be planned so that drainage is directed away from all foundations. Parking areas should be sloped and drainage gradients maintained to carry all surface water off the site.

4.8 Steel and Concrete Reactivity

Analytical testing of selected soil samples was performed to assess the potential for adverse reactivity with concrete and corrosivity with steel. Soluble sulfate tests were performed to evaluate potential sulfate attack against Portland Cement Concrete. Soluble sulfate contents were observed to be less than 15 ppm. Therefore, the potential for sulfate attack appears to be negligible and conventional Type I/II cement may be used for site concrete, according to the data furnished by WET Labs and the requirements of Section 19, Table 19-A-4 of the 1997 *Uniform Building Code.*

Resistivity tests are used as an indication of possible corrosion activity. Generally, the lower the native resistivity of the soils, the more likely that galvanic currents may occur and corrosion result. Resistivity values for the near-surface native soils are on the order of 4,000 to 25,000 ohm-cm; therefore, have a low to moderate corrosion potential. Non-metal pipelines should be used wherever possible. A corrosion engineer should review resistivity and pH testing results to determined corrosion protection for buried metal elements.

5. ADDITIONAL SERVICES

5.1 Project Bid Documents

It has been our experience during the bidding process, that contractors often contact us to discuss the geotechnical aspects of the project. Informal contacts between Kleinfelder and an individual contractor could result in incorrect or incomplete information being provided to the contractor. Therefore, we recommend a pre-bid meeting be held to answer any questions about the report prior to submittal of bids. If this is not possible, questions or clarifications regarding this report should be directed to the project Owner or his designated representative. After consultation with Kleinfelder, the project Owner (or his representative) should provide clarifications or additional information to all contractors bidding the job.

5.2 Construction Observation/Testing and Plan Review

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation and earthwork.
- Observation of footing trench excavations.
- Observation and testing of construction materials.
- Consultation as may be required during construction.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

The review of plans and specifications and the field observation and testing by Kleinfelder are an integral part of the conclusions and recommendations made in this report. If we are not retained for these services, the Client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during construction.

6. LIMITATIONS

Recommendations contained in this report are based on our field explorations, laboratory tests, and our understanding of the proposed construction. The study was performed using a mutually agreed upon scope of work. It is our opinion that this study was a cost-effective method to evaluate the subject site and evaluate some of the potential geotechnical concerns. More detailed, focused, and/or thorough investigations can be conducted. Further studies will tend to increase the level of assurance, however, such efforts will result in increased costs. If the Client wishes to reduce the uncertainties beyond the level associated with this study, Kleinfelder should be contacted for additional consultation.

The soils data used in the preparation of this report were obtained from borings made for this investigation. It is possible that variations in soils exist between the points explored. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at this site which are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to our recommendations. In addition, if the scope of the proposed project, locations of structures, or building loads change from the description given in this report, our firm should be notified.

This report has been prepared for design purposes for specific application to the Cold Springs Waste Water Treatment Plant Upgrades Project in accordance with the generally accepted standards of practice at the time the report was written. No warranty, express or implied, is made.

Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference," as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on- and off-site), or other factors including advances in man's understanding of applied science may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issue. Kleinfelder should be notified if the project is delayed by more than 24 months from the date of this report so that a review of site conditions can be made, and recommendations revised if appropriate.

It is the CLIENT'S responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk. Any party other than the Client who wishes to use this report shall notify Kleinfelder of such intended use by executing the "Application for Authorization to Use" which follows this document as an appendix. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

sand encountered in borings B8 and B13 at depths of approximately two feet and 11 feet, respectively. Heaving sand was encountered in borings B1, B2, and B5 at a depth of approximately 45 feet bgs and at a depth of 15 feet in borings B13 and B14. (Borings B1, B2, and B5 were advanced just west of the existing treatment plant. Borings B13 and B14 were advanced just south of the Diamond Peak lift station.)

During our field investigation, groundwater was encountered at depths ranging from 37 to 42 feet below ground surface (bgs) near the existing treatment facility and at depths of eight to nine feet near the Diamond Peak lift station. The borings for the new lift station (B13 and B14), were drilled on July 25 and July 28, 2003 and completed as monitoring wells. Slug tests were performed in these wells on August 13, 2003. At the time of our field testing, groundwater was measured at approximately 5½ feet bgs. During the field investigation for Village Parkway by Pezonella Associates, Inc. (2001), groundwater was encountered at a depth of four feet bgs at the southern end of the alignment. Groundwater is anticipated to intercept construction of the force main and lift station near the intersection of Diamond Peak Drive and Spicer Lake Court. Fluctuations in groundwater levels and soil moisture contents may occur due to variations in precipitation, land use, irrigation, and other factors.

3.3 Regional Geology and Faulting

The project site lies within the western portion of the Basin and Range Geomorphic Province. The Basin and Range province was formed by numerous north-south trending normal faults, which displaced to form the horst and graben morphology present throughout most of Nevada. The mountain ranges in western Nevada are primarily composed of Mesozoic or Early Tertiary intrusive and volcanic rocks. The intervening basins consist of deep accumulations of Quaternary age alluvium

The project site is located in UBC Seismic Zone 3, a relatively active seismic area. Based on our review of the Reno NW Quadrangle Earthquake Hazards Map (Szecsody, 1983) and University of Nevada-Reno aerial photos, an active lineament/fault of the Peterson Mountain fault zone crosses the force main alignment roughly at the intersection of Diamond Peak Drive and Spicer Lake Court. (An active fault is a fault with evidence indicating movement during the last 10,000 years (Price, 1998)). The Peterson Mountain fault zone is estimated to be capable of generating an earthquake of moment magnitude 7.0 (dePolo, et al., 1997). The approximate pipeline alignment with regards to the earthquake hazards map (Szecsody, 1983) is shown on the Plate 4, Appendix A.

The Peterson Mountain fault was trenched north of Cold Springs Road (Kleinfelder, 1994) and confirmed to be active. A geotechnical investigation by Summit™ Engineering Corporation (2002) for Parcel APN 566-01-08, located east of Diamond Peak states, "A fault location trench was excavated near the southwest corner of the site to determine if this fault crossed the site. Our exploration indicated that this fault did not cross the site." This indicates that the fault is most likely located on the west side of Diamond Peak Drive crossing through the Lake Hill Subdivision, located immediately north of the Diamond Peak Lift Station. However, a report by Pezonella Associates, Inc. (1998) indicates that the Lake Hill Subdivision, Units 1, 2, and 3 was trenched by Summit ™ Engineering Corporation in 1997 and the "presence of the suspected fault was not confirmed…" The reason for the inconsistencies between the referenced geotechnical reports, aerial photographs, and hazards map is unknown.

The project site also lies within the zone of influence of numerous other fault systems in Truckee River Basin, western Nevada, and eastern California. Should a seismic event occur, the site could be significantly affected by ground shaking. According to the *Reno NW Quadrangle Earthquake Hazards Map* (Szecsody, 1983), upgrades to the existing treatment plant and the majority of the force main alignment are located in an area, which would likely experience moderate severity of shaking (Level III) during a seismic event. As shown on Plate 4 (Appendix A), the southern section of the force main alignment, along Diamond Peak Drive, Cold Springs Drive, and approximately 1,200 linear feet of Village Parkway, crosses areas which would likely experience moderate severity of shaking (Level II) to the greatest severity of shaking (Level I). The proposed lift station is located in an area of greatest severity of shaking, which could possibly experience severe local liquefaction.

4. RECOMMENDATIONS

4.1 Site Clearing and Preparation

The locations of the proposed Cold Springs Wastewater Treatment Plant upgrades (the sequential batch reactors and aerobic digester, screen and grit removal structures, and mechanical sludge dewatering facility) and the new Diamond Peak Lift Station have been previously rough graded; however, a sparse cover vegetation is currently present at each site. (Please, see the site photographs provided in Section 3.1 of this report.) The majority of the force main alignment is located beneath paved roadway sections (Diamond Peak Drive, Cold Springs Drive, and Village Parkway) with the exception of approximately 3,500 linear feet located beneath Mud Springs Road, a gravel road.

Prior to construction, any surface vegetation and organic soils at should be stripped and removed from the site or stockpiled for use in landscape areas as approved by the Owner. It appears four inches can be used as a reasonable estimate for average depth of stripping. Deeper stripping/grubbing of organic soils, roots, etc., may be required in localized areas. The resulting voids backfilled with adequately compacted backfill soil. All man-made debris including structures, pavements, dump fills, and trash should be removed from the site.

The geotechnical engineer should be present during stripping and site preparation operations to observe stripping and grubbing depths, and to evaluate whether buried obstacles such as underground utilities, are present. Special care should be exercised in evaluating whether loose utility backfills exist which could adversely affect the planned structures. Excavations resulting from removal operations should be cleaned of all loose material and widened as necessary to permit access to compaction equipment.

Dust control will be the responsibility of the contractor. A dust control plan should be prepared by the owner, civil engineer, or contractor prior to the start of grading.

4.2 Earthwork

4.2.1 General Site Grading

Site preparation and grading should conform to the requirements contained in this report and in the suggested specifications, which are provided as Appendix C of this report. We anticipate that site grading can be performed with conventional earthmoving equipment.

Where fill is necessary, materials should meet the gradation and plasticity requirements listed for "structural fill" in Appendix C. It appears that the existing site soils will generally be capable of meeting recommended requirements for structural fill. Exceptions include near surface silt/clay layers encountered in borings B1, B3, B5, and B12 at a depth of approximately four feet bgs and at the surface in borings B9 and B10.

Fill placement and compaction requirements presented in Appendix C should be followed. Prior to fill placement, the exposed native soils should be scarified to a minimum depth of six inches, moisture conditioned as necessary, and compacted to a minimum of 90% relative compaction in accordance with the ASTM D1557 compaction test method.

No fill material should be placed, spread, or rolled on frozen subgrade. Areas to receive fill should be blanketed with a layer of loose fill (typically four to six inches thick) at the end of each workday to protect the ground from freezing. No fill soils should be moisture conditioned or placed when the atmospheric temperature is below 35 degrees Fahrenheit.

4.2.2 Temporary Unconfined Excavations

We understand that deep cuts of up to 15 and 25 feet are proposed to construct of the sequential batch reactors and screen and grit removal structures as part of the upgrades to the Cold Springs Wastewater Treatment Plant. In additional, excavations for the construction of the new Diamond Peak Lift Station are anticipated to extend approximately 25 feet bgs.

The use of steepened, temporary cut slopes will be needed to construct below grade structures. Excavations for upgrades at the treatment plant and lift station should comply with current OSHA safety requirements for Type C soils, with maximum inclinations of 1½:1 (horizontal to vertical). The above layback assumes complete dewatering of the soils and is a suggested guideline, which may require modification in the field after the start of construction.

The stability of slopes below the groundwater table will be a function of the method and degree to which the soils are dewatered. The contractor is ultimately responsible for the safety of workers and should strictly observe federal and local OSHA requirements for excavation shoring and safety. Due to the granular nature of the surface soils, some ravelling of temporary cut slopes should be anticipated. During wet weather, runoff water should be prevented from entering excavations.

4.2.3 Temporary Trench Excavation and Backfill

It appears that temporary confined excavations for subject project can be readily made with either a conventional backhoe or excavator. We understand that the force main will be placed at a depth of about four feet below grade. We expect that the utility trench walls will stand nearly vertical without significant sloughing in the areas north of Cold Springs Drive. The excavations south of Cold Springs Drive, particularly south of Spicer Lake Court, are anticipated to become unstable due to saturated conditions and flowing sand. The need for shoring or sloping of trench walls to protect personnel and provide temporary stability should be anticipated. Confined excavations within the native soils should comply with current OSHA safety requirements for Type C soils (Federal Register 29 CFR, Part 1926).

The contractor is ultimately responsible for the safety of workers and should be evaluated to verify their stability prior to occupation by construction personnel. All excavations should strictly observe federal and local OSHA requirements for excavation shoring and safety.

During wet weather, runoff water should be prevented from entering excavations. Water should be collected and disposed of outside the construction limits. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance of one-third the slope height from the top of any excavation.

The native soils are anticipated to generally meet the Class E backfill criteria as outline in Section 200.03.06 of the *Standard Specification for Public Works Construction* (2001) sponsored by Washoe County. Backfills for trenches or other excavations within pavement areas, beneath slabs, and adjacent to foundations should be compacted in six- to eight-inch layers with mechanical tampers. Jetting and flooding should not be permitted. We recommend all backfill be compacted to a minimum compaction of 90% of the maximum dry density as determined by ASTM D1557. The moisture content of compacted granular backfill soils should be within two percent of optimum. Poor compaction in utility trench backfill may cause excessive settlements resulting in damage to the pavement structural section or other overlying improvements. Compaction of trench backfill outside of improvement areas should be a minimum of 85% relative compaction.

As an alternative, slurry backfill could be used in lieu of bedding and pipe zone backfill. If used, slurry backfill should meet the requirements outlined in Section 202.02 of the *Standard Specifications* (2001).

For any utilities placed in existing Washoe County streets, the street has to be repaired to the satisfaction of the County Engineer. As a minimum, this requires fill depth removal and replacement of asphalt for half the width, or replacement of the pavement with a non-woven reinforcing fabric with a 2-inch asphalt overlay for half the street width. Type II slurry seal is required for the entire width of the street. Full width street improvements may be required if the proposed utility location is located too close to the centerline of the existing street.

4.2.4 Pipe Thrust Blocks

Pipe thrust blocks may be designed using an equivalent fluid passive pressure of 350 pcf. Concrete should be placed directly on undisturbed, native soil under the direction of a qualified field inspector.

4.2.5 Construction Dewatering

During our field investigation, groundwater was encountered at depths ranging from eight to nine feet near the Diamond Peak Lift Station. During field slug tests in borings B13 and B14, groundwater was measured at approximately 5½ feet bgs. We understand the lift station will extend up to 25 bgs.

Dewatering will be required so that free water does not interfere with construction. To prevent unstable trench wall conditions and to provide a firm, unyielding subgrade for construction, groundwater should be lowered about two feet below the bottom of the excavation and below any utility excavations.

The dewatering system should be a contractor-designed system. Control of groundwater should be accomplished in such a manner that will preserve the strength of the foundation of soils, will not cause instability of excavated slopes, and will not result in damage to existing structures. Where necessary, the water should be lowered in advance of any excavation by deep wells, well points, or other methods.

Our slug test results indicated that the site soils at the lift station have relatively high hydraulic conductivity values ranging from 2.5×10^{-2} to 9.2×10^{-3} (cm/sec). Based on conversations with Mr. Gary Rambosek of Department of Public Works, Engineering Department, County of Washoe, we understand that dewatering was successfully completed for the existing the sewer line at Diamond Peak and Spicer Lake Court using two 24-inch dewatering wells that were left in-place. Mr. Rambosek also stated that he believed at least 40 well points were used to lower the groundwater table during the construction of the original Diamond Peak Lift Station. It should be noted that heaving sand (clean sands that flowed vertically into the drill stem) was encountered during our investigation of the site for the proposed lift station.

Open pumping should not be permitted if it results in boils, loss of fines, unacceptable settlement of existing structures, or causes construction slope instability. Water should not be allowed to pool and remain in the excavated area over an extended period of time. General lowering of the groundwater can result in settlement of nearby structures. This should be taken into account in the contractor's design of the dewatering system. Any nearby structures should be monitored for settlement and any signs of instability during dewatering operations. It may be desirable to examine the cost-effectiveness of using sheet piling to limit the effects of dewatering on the surrounding improvements. The majority of the surrounding improvements, including residential structures were not in-place during dewatering for the construction of the initial lift station.

Discharge should be arranged to meet the necessary local governmental requirements. Discharge should be arranged to facilitate sampling by the engineer of record.

4.2.6 Subgrade Stabilization

Soft subgrade conditions should be anticipated in the bottom of excavation for the pump station and utility trenches, which extend below or near the groundwater surface. These soils may be unstable and deflect (pump) under construction equipment loads. Saturated, pumping subgrade materials will not be suitable for placement of structural fill or structures and will need to be stabilized. Over-excavation and placement of drain rock or similar materials in conjunction with geogrid or geotextile should be included in the construction documents. For preliminary planning purposes we recommend a minimum depth of 18 inches of drain rock with geogrid (Tensar^{TM} BX1200 or equivalent) placed on the subgrade and mid-center of the rock layer. Individual sheets of geogrid should overlap by at least 12 inches. Light, track-mounted construction equipment should be anticipated in excavations for the pump station to help prevent destabilizing the subgrade soils and causing "pumping" conditions.

4.3 Foundations

4.3.1 General

The proposed structures may be supported by conventional spread footings and/or mat foundations bearing on non-expansive native soil or compacted imported fill. Any loose soil in the bottom of footing excavations should be recompacted to at least 90% relative compaction or removed to expose firm, unyielding material. The design engineer should provide reinforcing steel requirements for foundations.

The allowable bearing capacities provided in the sections below may be increased by one-third for total loading conditions, including wind and seismic forces. The allowable bearing pressures are net values; therefore, the weight of the foundation and backfill may be neglected when computing dead loads.

The site is located in UBC Seismic Zone3. If seismic loadings are evaluated using the 1997 UBC method, we recommend using a seismic zone factor of 0.3 and a Soil Profile Type S_D, as outlined in Tables 16-I and 16-J of the UBC.

4.3.2 Sequential Batch Reactors and Aerobic Digester (above the groundwater table)

The sequential batch reactors and aerobic digester can be supported on a mat foundation. In order to limit post-construction settlement to less than 1-inch, mat foundations may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot for dead loads plus long-term live loads.

We recommend that SBRs and aerobic digester be filled to their design water levels prior to hooking up utilities/plumbing to allow immediate settlement beneath the structures to occur.

4.3.3 Screen and Grit Removal Structures and Mechanical Sludge Dewatering Facility (above the ground water table)

Two options are being considered for the screen and grit removal structures. One would consist of an at-grade above ground structure. The second option would be a below grade structure, extending about 25 feet below grade. The mechanical sludge dewatering facility will be constructed at-grade.

For the structures at-grade, foundations may be designed for an allowable bearing capacity of 3,000 pounds per square foot for dead and live loads. Exterior foundations should be embedded a minimum of 24 inches below lowest adjacent exterior finish grade for frost protection and confinement. Interior footings should be bottomed at least 12 inches below lowest adjacent finish grade for confinement. Wall foundation dimensions should satisfy the requirements listed in the latest edition of the Uniform Building Code.

For the second option for the screen and grit removal structures (constructed 25 feet below grade), foundations may be designed for an allowable bearing capacity of 4,500 pounds per square foot for dead and live loads.

We estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be less than one inch, with approximate differential settlement of ½ inches or less between adjacent similarly loaded isolated footings.

4.3.4 Lift Station (below the groundwater table)

The foundations for the new lift station will be located beneath the groundwater table. These foundations should be supported on a minimum of 10 inches of compacted gravel over stabilized subgrade. The compacted gravel should be graded gravel, which meets requirements for Class C or Class D backfill listed in Section 200.03 of the Standard Specifications or other material clean gravel approved by the Geotechnical Engineer. The rock should be vibrated into place to a minimum relative compaction of 90 percent or a relative density of 70 percent.

Foundations designed and constructed in accordance with the recommendations of this geotechnical report may be designed for an allowable bearing capacity of 2,500 pounds per square foot.

We estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be less than one inch, with approximate differential settlement of ½ inches or less between adjacent similarly loaded isolated footings.

4.4 Concrete Slab-on-Grade Construction

All concrete floor slabs should have a minimum thickness of four inches. Slab thickness and structural reinforcing requirements within the slab should be determined by the design engineer. Specific design recommendations for floor slabs located above and below the groundwater table are provide in the following paragraphs.

Floor Slabs Above the Water Table (Upgrades to the Cold Springs Wastewater Treatment Plant)

Prior to constructing concrete slabs, patios, sidewalks, or other slabs-on-grade, the upper six inches of slab subgrade should be scarified, moisture conditioned to within 2% of optimum, and uniformly compacted to at least 90% of maximum dry density as determined by ASTM D1557. Scarification and compaction will not be required if floor slabs are to be placed directly on undisturbed compacted structural fill.

At least four inches of Type 2 aggregate base should be placed beneath slab-on-grade floors to provide uniform support. The aggregate base should be compacted to a minimum of 95% relative compaction. We recommend that the base course be placed within three to five days (depending on the time of year) after moisture conditioning and compaction of the subgrade soil. The subgrade should be protected against drying until the concrete slab is placed.

In floor slab areas where moisture sensitive floor coverings are planned, an impermeable membrane should be used to help reduce the migration of moisture vapor through the concrete slabs from external sources (e.g. landscape irrigation). The impermeable membrane should either be a minimum of 8-mil thick polyethylene and protected by two inches of fine, moist sand placed both above and below the membrane or a minimum, or a minimum of 10-mil thick polyethylene and placed beneath the slab subgrade and aggregate base section, protected by two inches of overlying sand. The sand cover should be moistened and tamped prior to slab placement. Care should be taken not to damage the vapor during construction. These recommendations are generic in nature and the manufacture's recommendations for installation and protection of any floor covering should ultimately take precedence.

During the winter months, concrete should be placed and protected in accordance with the recommendations provided in the American Concrete Institute, ACI 306R, *Cold Weather Concreting*.

Floor Slabs Below the Groundwater Table (Diamond Peak Lift Station)

For concrete floor slabs below the groundwater table support should be provided by a 10-inch layer of compacted gravel over stabilized subgrade. The compacted gravel should be graded gravel, which meets requirements for Class C or Class D backfill listed in Section 200.03 of the Standard Specifications or other material clean gravel approved by the Geotechnical Engineer. The rock should be vibrated into place to 90 percent relative compaction or a relative density of 70 percent.

Based on the hydraulic conductivities estimated from field slug tests (Section 2.2) and conversations with Mr. Gary Rambosek of Department of Public Works, Engineering Department, County of Washoe with regards to construction dewatering in the area, we anticipate that a permanent subdrain system for reducing hydrostatic pressure beneath the floor of the lift station is impractical. We anticipate that the structure will need to be waterproofed and designed to resist uplift forces. We recommend using a design static water level of four feet bgs in calculating hydrostatic uplift forces.

4.5 Retaining Structures

Lateral earth pressures will be imposed on all subterranean structures, including retaining walls and foundations. Table 2 presents a list of soil parameters, which we recommend for design of these structures assuming a level backfill above the water table. Table 3 presents a list of soil parameters, which we recommend for the design of structures below the water table with level backfill. Hydrostatic forces will need to be considered for design of the lift station at the intersection of Diamond Peak Drive and Spicer Lake Court. We recommend assuming a design groundwater level of four feet bgs at the lift station.

TABLE 2

LATERAL EARTH PRESSURE COEFFICIENTS

Earth Pressure	Equivalent Fluid Density (pcf)
Active	35
At-rest	55
Passive	350
Friction Coefficient	0.4

Where backfill is placed against structures, we recommend that non-expansive, free-draining materials meeting filter criteria be used in the zone immediately adjacent to the structure to reduce hydrostatic forces. Alternately, the use of pre-manufactured drainage panels should be considered. Furthermore, adequate drainage of the backfill in the form of subdrains and/or weepholes should be provided at the base of the wall.

TABLE 3

LATERAL EARTH PRESSURE COEFFICIENTS WITH HYDROSTATIC PRESSURE

Earth Pressure	Equivalent Fluid Density (pcf)
Active	75
At-rest	85
Passive	215
Friction Coefficient	0.4

Recommended minimum factors of safety against sliding, overturning, and bearing failure are listed in Table 4, below.

TABLE 4

RECOMMENDED MINIMUM FACTORS OF SAFETY

Factor of safety against sliding	1.5
Factor of safety against overturning	2
Factor of safety against bearing failure	3

If both passive and frictional resistances are assumed to act concurrently, we recommend a minimum safety factor of 2 be used for design against sliding.

The at-rest case is applicable for braced walls where rotational movement is confined to less than 0.001H. If greater movement is possible, the active case applies. A wall movement of about 0.01H is required to develop the full pressure.

Lateral pressures computed using the values in Tables 2 and 3 assume that the non-expansive native backfill will extend laterally at least one-half of the wall height. If this condition does not apply, the design values may require revision. This backfill should be compacted to 90% of maximum dry density and within 2% of the optimum moisture content as determined by ASTM D1557. Over-compaction should be avoided, as the increased compactive effort will result in lateral pressures higher than those recommended above. Heavy compaction equipment or other loads should not be allowed in close proximity to the wall unless planned for in the structural design.

4.6 Pavement Sections

Pavement sections below are presented for site development only and are not recommendations for dedicated pavements. The recommended pavement structural sections for the project were calculated using the Caltrans method for flexible pavement design. Structural sections for several traffic loadings and their approximate ESALs are presented in Table 5. Traffic loadings should be verified prior to construction. A minimum R-value of 70 was used to represent the aggregate base material and a minimum R-value of 50 for select subbase. A minimum R-value of 12 was used for the native subgrade.

TABLE 5

PAVEMENT STRUCTURAL SECTIONS

Traffic Index	Approximate ESAL (18 kip single axle load)	Recommended Minimum Structural Section
5.0	7,500	3.5 inches of asphalt concrete 7.5 inches of aggregate base
6.0	34,000	4 inches of asphalt concrete 10 inches of aggregate base or 4 inches of asphalt concrete 4 inches of select subbase 6.5 inches of aggregate base
7.0	123,00	5 inches of asphalt concrete 12 inches of aggregate base or 5 inches of asphalt concrete 4.5 inches of select subbase 8 inches of aggregate base

Placement and compaction procedures for materials and construction should conform to the suggested specifications contained in Appendix C of this report. The sections presented in Table 5 are based on a single R-value test performed on a select sample obtained during our investigation and should be considered preliminary in nature. We recommend verification of soil conditions as construction progresses so that appropriate revisions can be made if necessary.

The pavement structural sections presented in Table 5 are designed for the assumed traffic loadings. However, based on our experience in the Reno area, environmental aspects such as freeze-thaw cycles and thermal cracking will probably govern the life of AC pavements. Thermal cracking of the asphalt pavements allows more water to enter the pavement section, which promotes deterioration and increases maintenance costs. A yearly maintenance program of asphalt concrete crack sealing is recommended.

4.7 Site Drainage

Final elevations at the site should be planned so that drainage is directed away from all foundations. Parking areas should be sloped and drainage gradients maintained to carry all surface water off the site.

4.8 Steel and Concrete Reactivity

Analytical testing of selected soil samples was performed to assess the potential for adverse reactivity with concrete and corrosivity with steel. Soluble sulfate tests were performed to evaluate potential sulfate attack against Portland Cement Concrete. Soluble sulfate contents were observed to be less than 15 ppm. Therefore, the potential for sulfate attack appears to be negligible and conventional Type I/II cement may be used for site concrete, according to the data furnished by WET Labs and the requirements of Section 19, Table 19-A-4 of the 1997 *Uniform Building Code.*

Resistivity tests are used as an indication of possible corrosion activity. Generally, the lower the native resistivity of the soils, the more likely that galvanic currents may occur and corrosion result. Resistivity values for the near-surface native soils are on the order of 4,000 to 25,000 ohm-cm; therefore, have a low to moderate corrosion potential. Non-metal pipelines should be used wherever possible. A corrosion engineer should review resistivity and pH testing results to determined corrosion protection for buried metal elements.

5. ADDITIONAL SERVICES

5.1 Project Bid Documents

It has been our experience during the bidding process, that contractors often contact us to discuss the geotechnical aspects of the project. Informal contacts between Kleinfelder and an individual contractor could result in incorrect or incomplete information being provided to the contractor. Therefore, we recommend a pre-bid meeting be held to answer any questions about the report prior to submittal of bids. If this is not possible, questions or clarifications regarding this report should be directed to the project Owner or his designated representative. After consultation with Kleinfelder, the project Owner (or his representative) should provide clarifications or additional information to all contractors bidding the job.

5.2 Construction Observation/Testing and Plan Review

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation and earthwork.
- Observation of footing trench excavations.
- Observation and testing of construction materials.
- Consultation as may be required during construction.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

The review of plans and specifications and the field observation and testing by Kleinfelder are an integral part of the conclusions and recommendations made in this report. If we are not retained for these services, the Client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during construction.

6. LIMITATIONS

Recommendations contained in this report are based on our field explorations, laboratory tests, and our understanding of the proposed construction. The study was performed using a mutually agreed upon scope of work. It is our opinion that this study was a cost-effective method to evaluate the subject site and evaluate some of the potential geotechnical concerns. More detailed, focused, and/or thorough investigations can be conducted. Further studies will tend to increase the level of assurance, however, such efforts will result in increased costs. If the Client wishes to reduce the uncertainties beyond the level associated with this study, Kleinfelder should be contacted for additional consultation.

The soils data used in the preparation of this report were obtained from borings made for this investigation. It is possible that variations in soils exist between the points explored. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at this site which are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to our recommendations. In addition, if the scope of the proposed project, locations of structures, or building loads change from the description given in this report, our firm should be notified.

This report has been prepared for design purposes for specific application to the Cold Springs Waste Water Treatment Plant Upgrades Project in accordance with the generally accepted standards of practice at the time the report was written. No warranty, express or implied, is made.

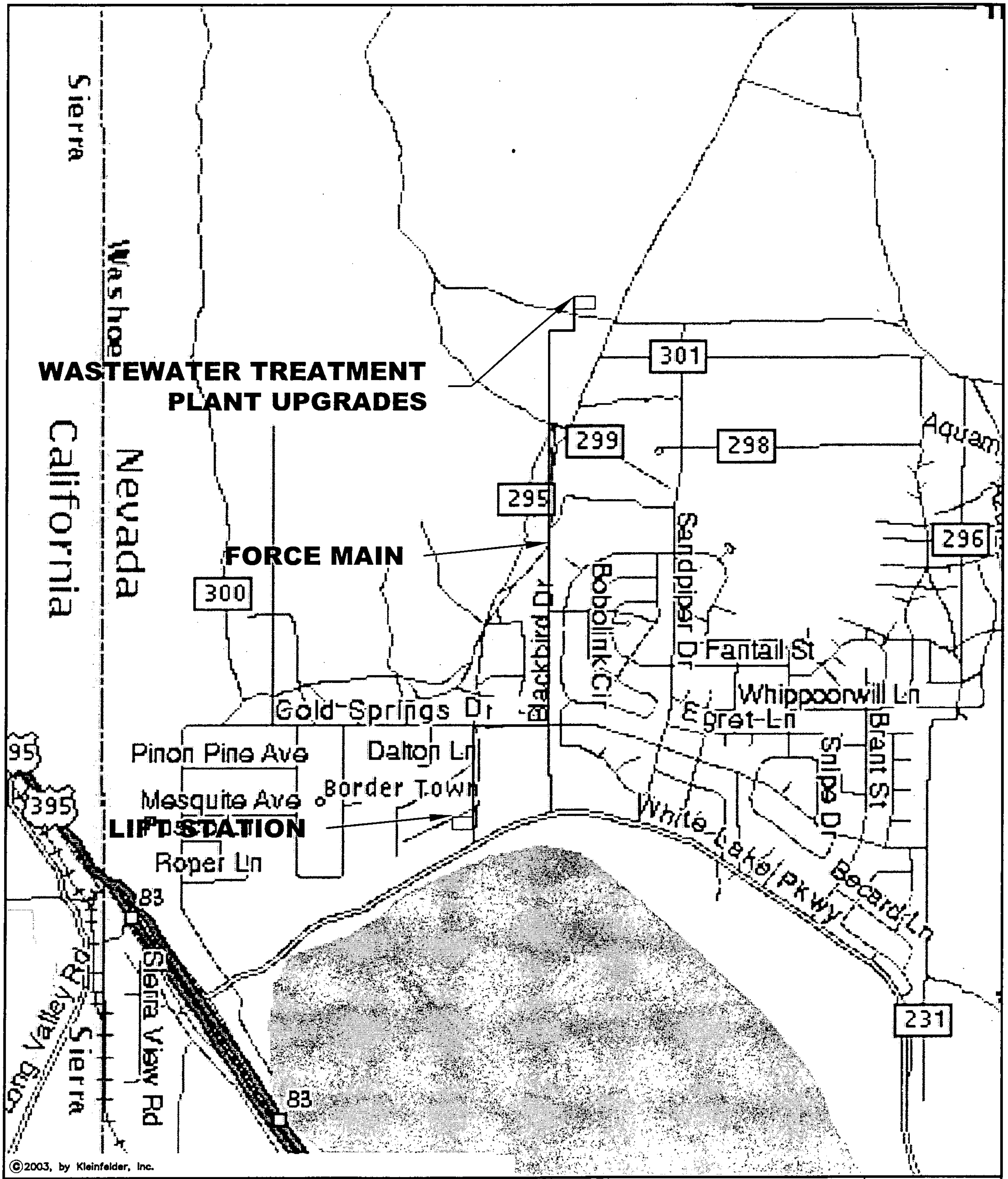
Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference," as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on- and off-site), or other factors including advances in man's understanding of applied science may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issue. Kleinfelder should be notified if the project is delayed by more than 24 months from the date of this report so that a review of site conditions can be made, and recommendations revised if appropriate.

It is the CLIENT'S responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk. Any party other than the Client who wishes to use this report shall notify Kleinfelder of such intended use by executing the "Application for Authorization to Use" which follows this document as an appendix. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

APPENDIX A

Plates



KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 33247.01

VICINITY MAP

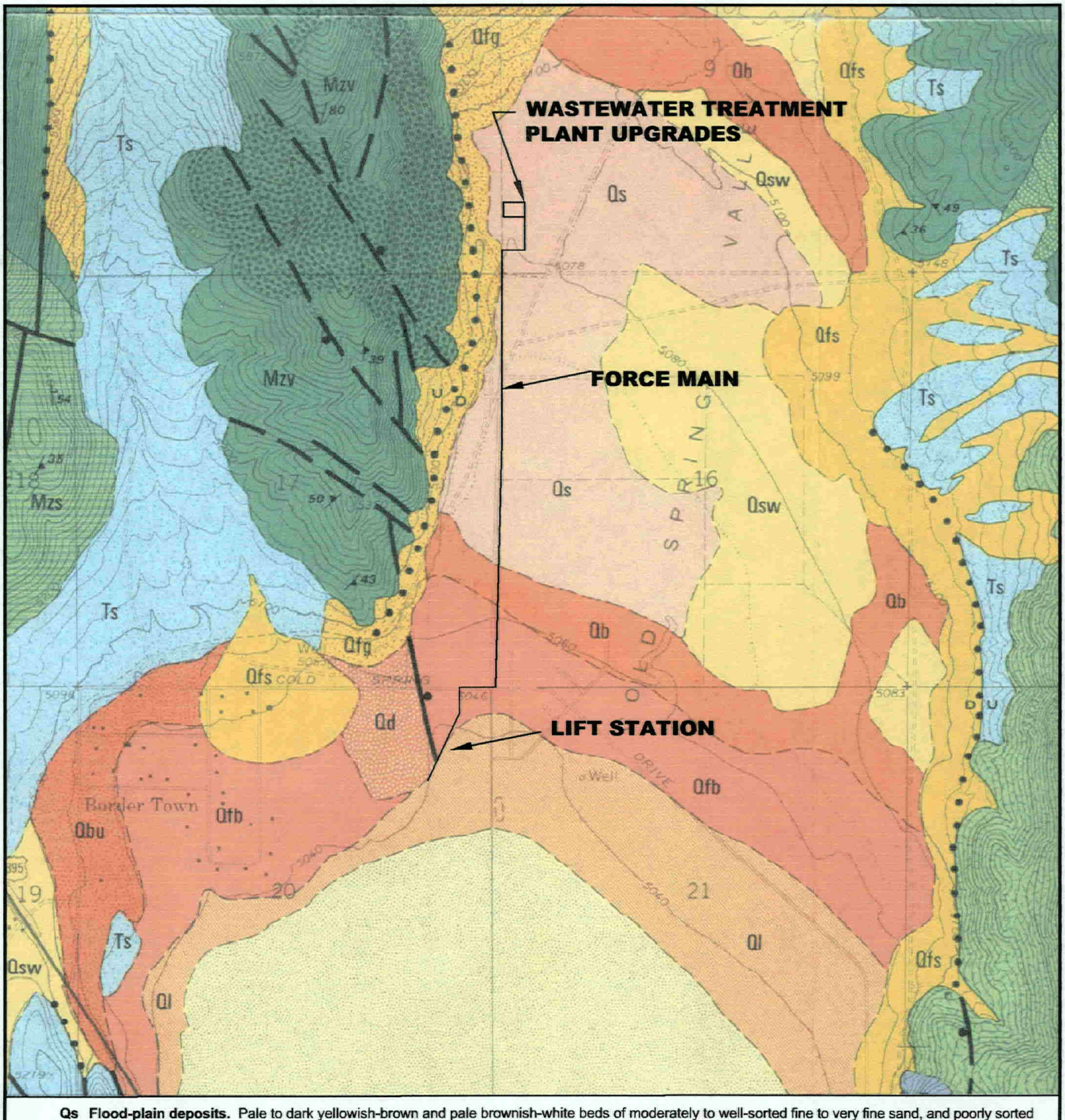
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

PLATE

1

CAD FILE: L:\2003\DRAFTING\33247\33247-MAPS.DWG



Qs **Flood-plain deposits.** Pale to dark yellowish-brown and pale brownish-white beds of moderately to well-sorted fine to very fine sand, and poorly sorted sandy clay and mud.

Qb **Beach deposits.** Lakeshore deposits of pale yellowish-brown to pale yellowish-white, granular medium to coarse sand, sandy pebble gravel, and granule gravel.

Qfb **Beach deposits.** Forebeach deposits of pale yellowish-brown to grayish-orange, pebbly to granular coarse to medium sand, and sandy granule gravel. Grades laterally into beach deposits.

Ql **Beach deposits.** Lake-floor deposits of yellowish-gray to dark-gray and black, very thin-bedded clay and sandy mud.

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RENO, NEVADA 89502
Tel. (775) 689-7800

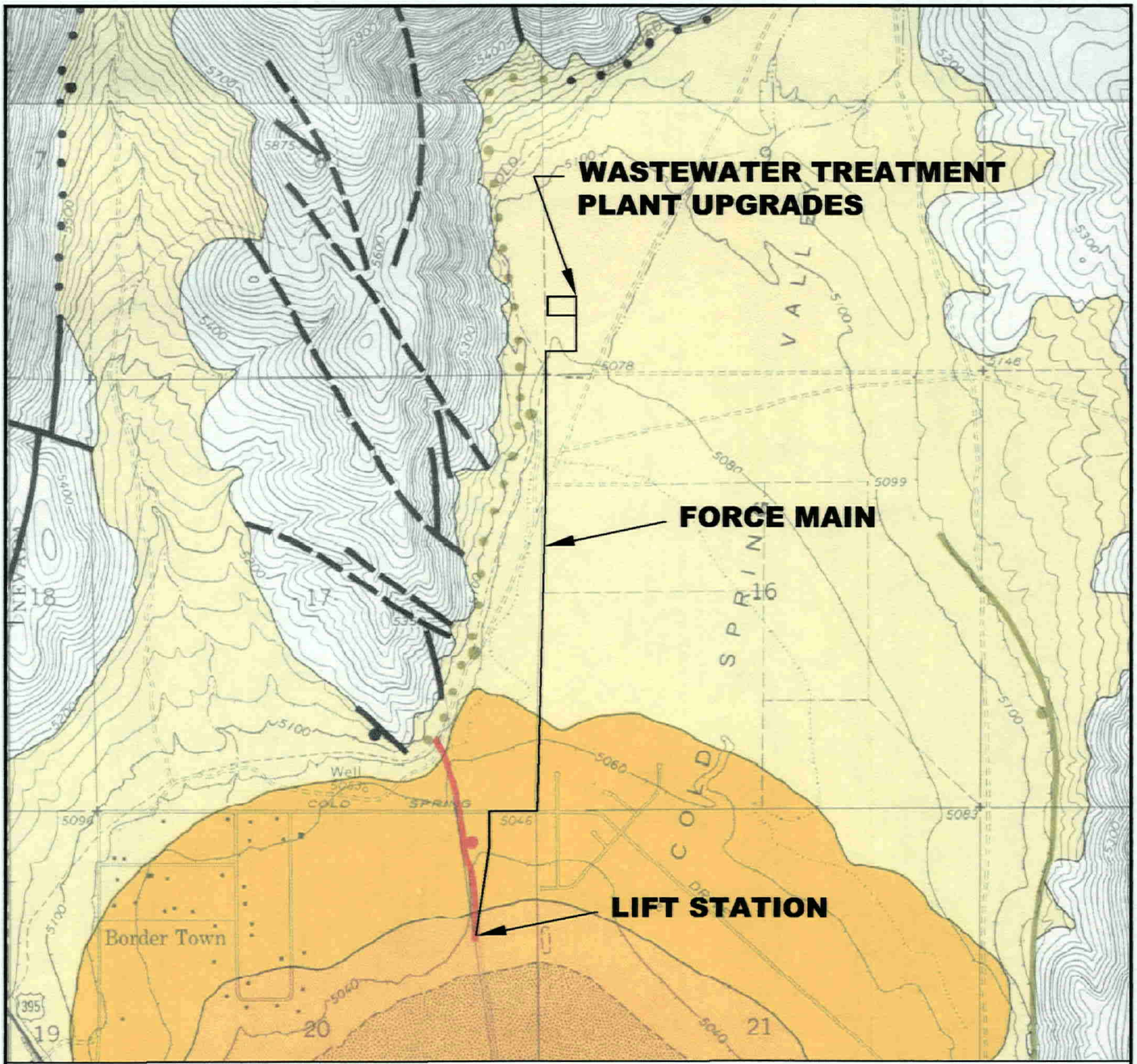
PROJECT NO. 33247.01

GEOLOGIC MAP

COLD SPRINGS WASTEWATER TREATMENT PLANT
COLD SPRINGS, NEVADA

PLATE
3

CAD FILE: L:\2003\DRAFTING\33247\33247-MAPS.DWG



Potential For Ground Shaking During Earthquakes

I	Greatest severity of shaking. Depth to ground water less than 3 m (10 ft). Unconsolidated deposits with low rigidity. Possible severe liquefaction locally
II	Moderate severity of shaking. Includes units from I where depth to ground water is greater than 3 m (10 ft); also includes unconsolidated deposits with moderate to moderately high rigidity where depth to ground water is less than 10 m (33 ft). May be subject to liquefaction
III	Moderate severity of shaking. Includes unconsolidated deposits with moderate to moderately indurated deposits with moderately high rigidity where depth to ground water is greater than 10 m (33 ft); also includes moderately indurated deposits with moderately high rigidity where depth to ground water is less than 10 m (33 ft)

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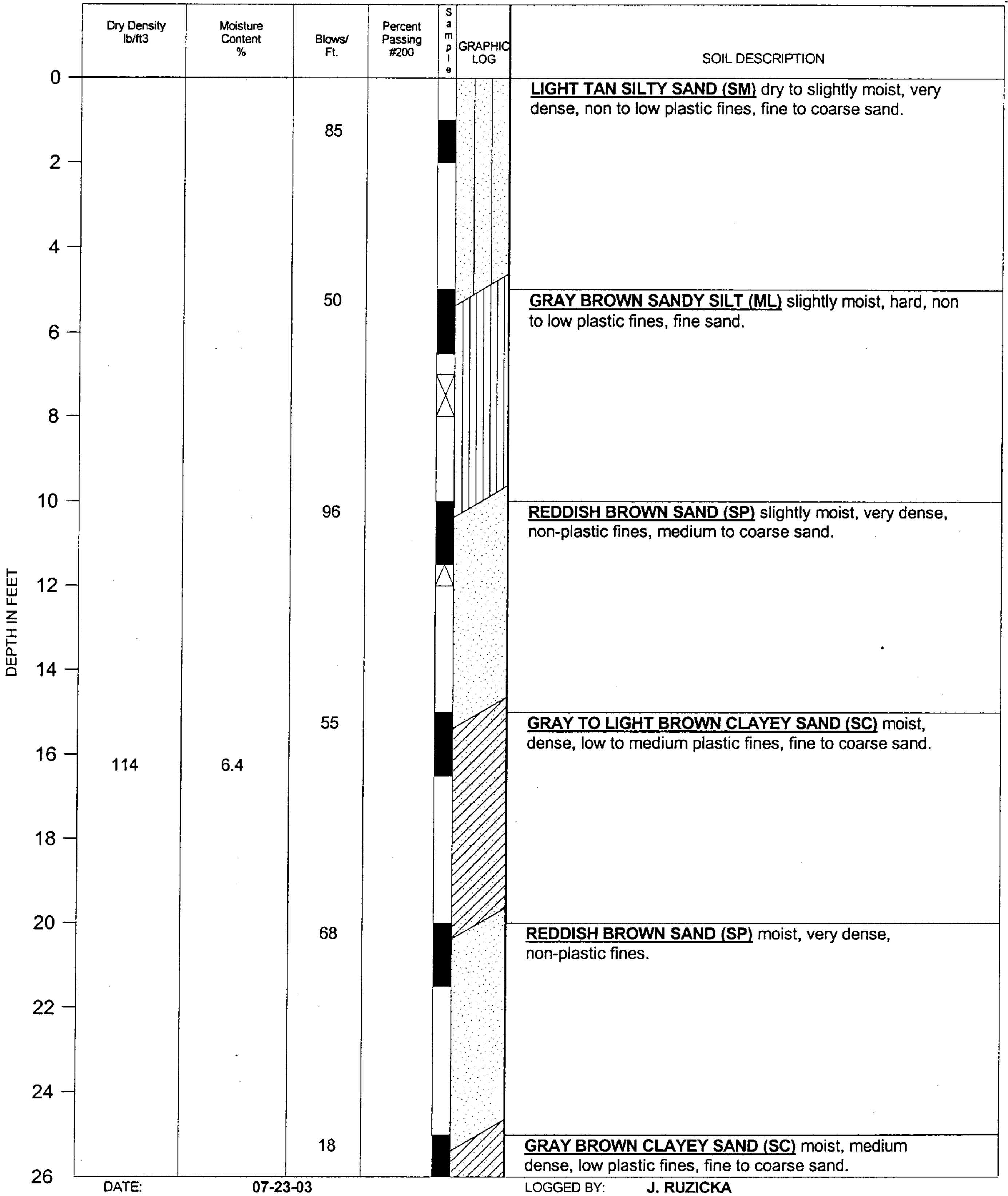
EARTHQUAKE HAZARDS MAP

COLD SPRINGS WASTEWATER TREATMENT PLANT
COLD SPRINGS, NEVADA

PLATE
4

CAD FILE: L:\2003\DRAFTING\33247\33247-MAPS.DWG



LIGHT TAN SILTY SAND (SM) dry to slightly moist, very dense, non to low plastic fines, fine to coarse sand.

GRAY BROWN SANDY SILT (ML) slightly moist, hard, non to low plastic fines, fine sand.

REDDISH BROWN SAND (SP) slightly moist, very dense, non-plastic fines, medium to coarse sand.

GRAY TO LIGHT BROWN CLAYEY SAND (SC) moist, dense, low to medium plastic fines, fine to coarse sand.

REDDISH BROWN SAND (SP) moist, very dense, non-plastic fines.

GRAY BROWN CLAYEY SAND (SC) moist, medium dense, low plastic fines, fine to coarse sand.

DATE: **07-23-03**
TOTAL DEPTH: **50.0 feet**
DIAMETER: **6 inch**

LOGGED BY: **J. RUZICKA**
EQUIPMENT: **CME 55 AUGER**
ELEVATION **Approx. 5075 feet**

KLEINFELDER

PROJECT NO. **33247.01**

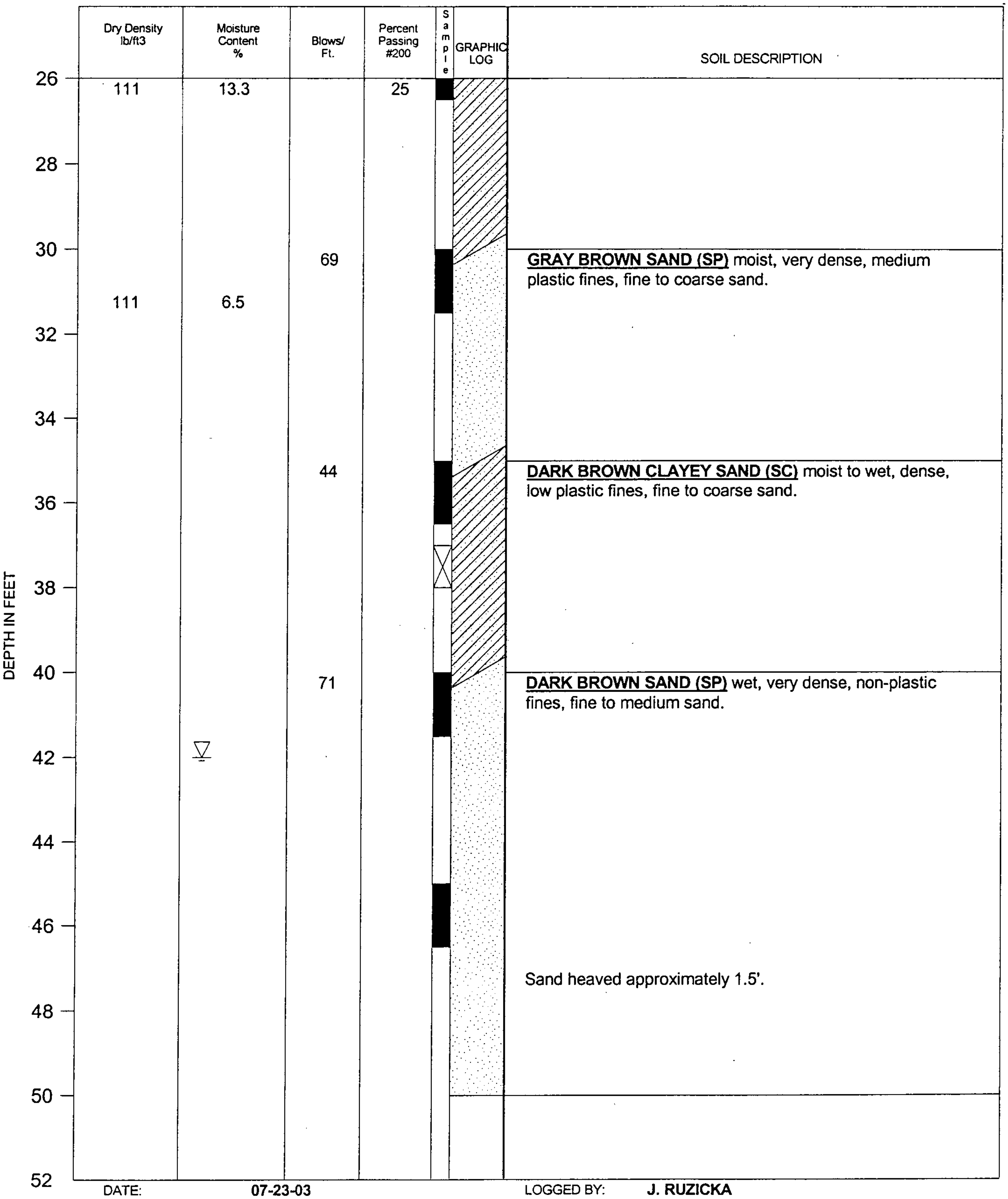
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-1

PLATE

5



GRAY BROWN SAND (SP) moist, very dense, medium plastic fines, fine to coarse sand.

DARK BROWN CLAYEY SAND (SC) moist to wet, dense, low plastic fines, fine to coarse sand.

DARK BROWN SAND (SP) wet, very dense, non-plastic fines, fine to medium sand.

Sand heaved approximately 1.5'.

DATE:	07-23-03	LOGGED BY:	J. RUZICKA
TOTAL DEPTH:	50.0 feet	EQUIPMENT:	CME 55 AUGER
DIAMETER:	6 inch	ELEVATION	Approx. 5075 feet

KLEINFELDER

PROJECT NO. 33247.01

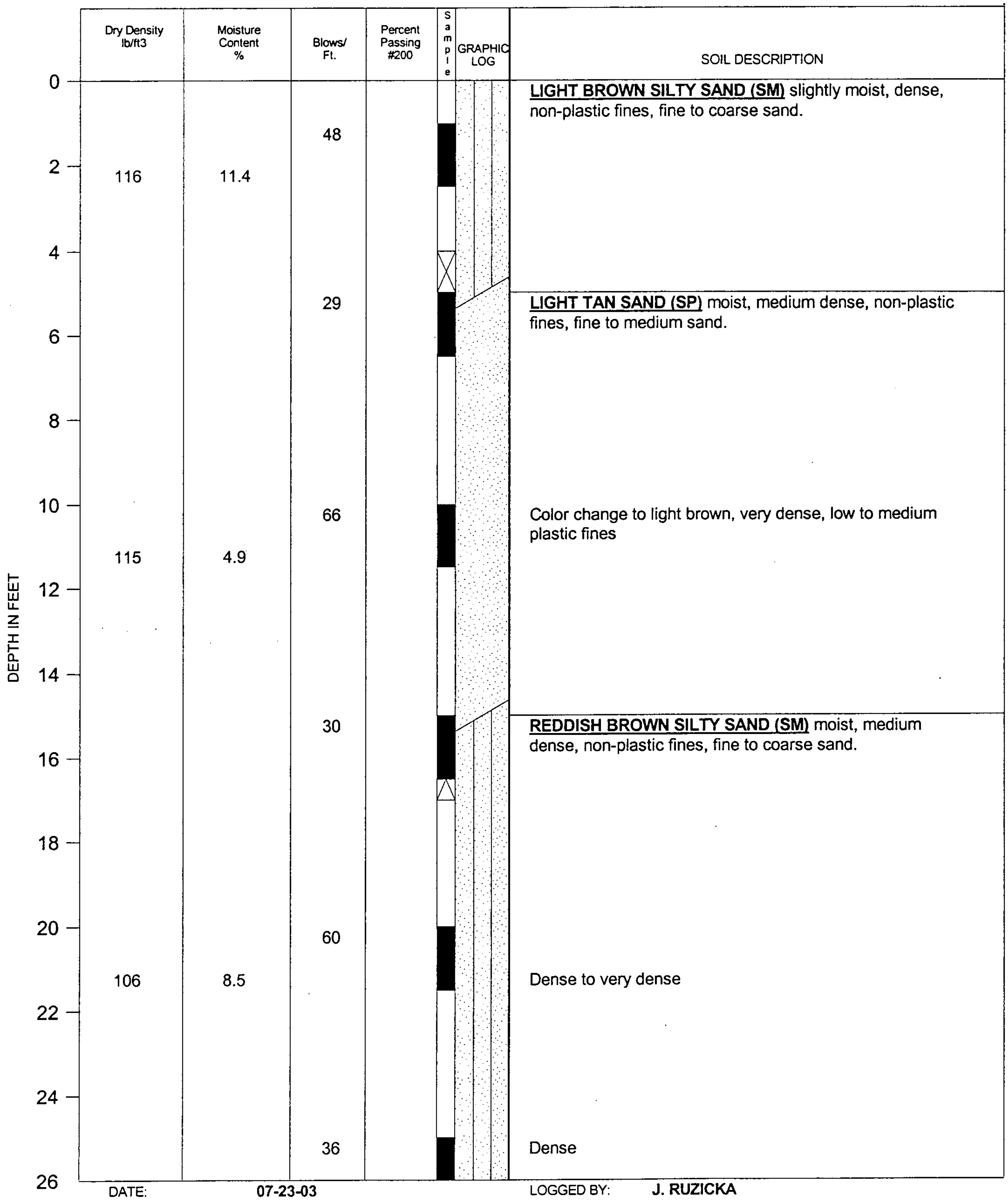
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-1

PLATE

5



LIGHT BROWN SILTY SAND (SM) slightly moist, dense, non-plastic fines, fine to coarse sand.

LIGHT TAN SAND (SP) moist, medium dense, non-plastic fines, fine to medium sand.

Color change to light brown, very dense, low to medium plastic fines

REDDISH BROWN SILTY SAND (SM) moist, medium dense, non-plastic fines, fine to coarse sand.

Dense to very dense

Dense

DATE: 07-23-03
TOTAL DEPTH: 46.5 feet
DIAMETER: 6 inch

LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 5077 feet

KLEINFELDER

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-2

PLATE
6

PROJECT NO. **33247.01**

DEPTH IN FEET	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	GRAPHIC LOG	SOIL DESCRIPTION
26							
28							
30			80				**REDDISH BROWN SAND WITH SILT (SP-SM)** moist, very dense, non-plastic fines, fine to coarse sand.
	119	7.4		11			
32							
34							
			24				**BROWN CLAYEY SAND (SC)** moist, medium dense, low to medium plastic fines, fine to coarse sand.
36	110	18.2		33			
		∇					
38							
40			58				**BROWN SAND (SP)** wet, very dense, non-plastic fines, fine to medium sand.
42							Fat clay with fine to coarse sand at 42'
44							
							Color change to dark brown
46							Sand heaved 5'
48							
50							
52							

DATE: **07-23-03**
TOTAL DEPTH: **46.5 feet**
DIAMETER: **6 inch**

LOGGED BY: **J. RUZICKA**
EQUIPMENT: **CME 55 AUGER**
ELEVATION **Approx. 5077 feet**

KLEINFELDER

COLD SPRINGS WASTEWATER TREATMENT PLANT

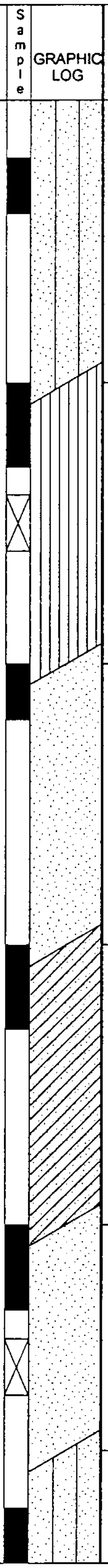
COLD SPRINGS, NEVADA

LOG OF B-2

PROJECT NO. **33247.01**

PLATE

6



DEPTH IN FEET	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
0			85		**LIGHT BROWN SILTY SAND (SM)** slightly moist, very dense, low plastic fines, fine to coarse sand.
5	91	9.3	55		**GRAY TO TAN SANDY SILT (ML)** slightly moist, hard, non-plastic fines, fine sand.
10			73		**REDDISH BROWN SAND SOME GRAVEL (SP)** moist, very dense, non-plastic fines, fine to coarse sand, gravel up to 1" in diameter.
15	101	15.5	44		**REDDISH BROWN CLAYEY SAND (SC)** moist, dense, low plastic fines, fine to coarse sand.
20			99		**RED BROWN SAND WITH GRAVEL (SP)** moist, very dense, non-plastic fines, fine to coarse sand, gravel up to 1/2" in diameter.
24	113	6.1	40		**BROWN SILTY SAND (SM)** moist, dense, non-plastic fines, fine to coarse sand.

DATE:	**07-24-03**	LOGGED BY:	**J. RUZICKA**
TOTAL DEPTH:	**44.0 feet**	EQUIPMENT:	**CME 55 AUGER**
DIAMETER:	**6 inch**	ELEVATION	**Approx. 5076 feet**

KLEINFELDER

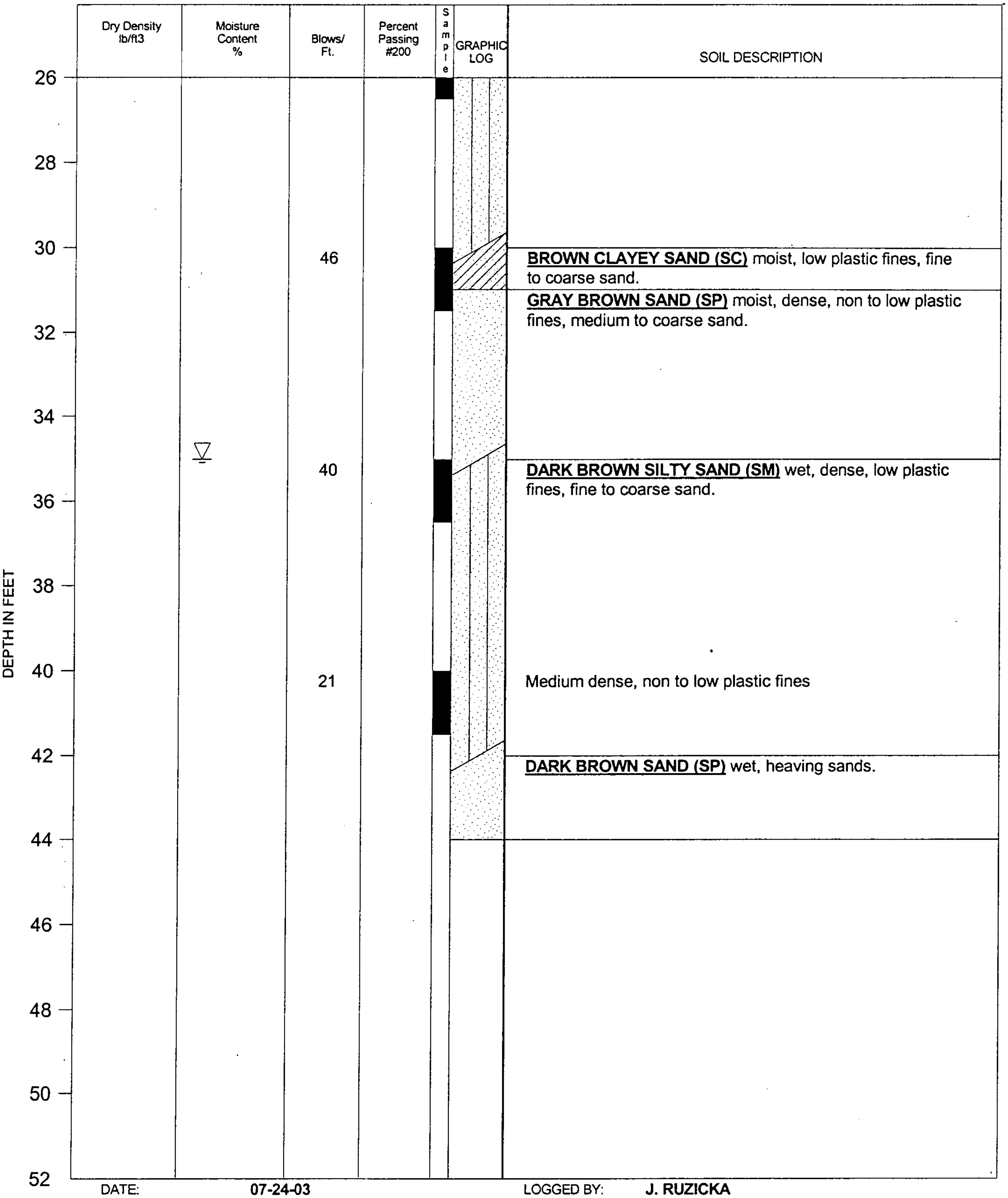
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-3

PLATE
7

PROJECT NO. **33247.01**



DEPTH IN FEET	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
30			46		**BROWN CLAYEY SAND (SC)** moist, low plastic fines, fine to coarse sand.
31					**GRAY BROWN SAND (SP)** moist, dense, non to low plastic fines, medium to coarse sand.
35		∇	40		**DARK BROWN SILTY SAND (SM)** wet, dense, low plastic fines, fine to coarse sand.
40			21		Medium dense, non to low plastic fines
42					**DARK BROWN SAND (SP)** wet, heaving sands.

DATE: 07-24-03
TOTAL DEPTH: 44.0 feet
DIAMETER: 6 inch

LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION: Approx. 5076 feet

KLEINFELDER

PROJECT NO. 33247.01

COLD SPRINGS WASTEWATER TREATMENT PLANT

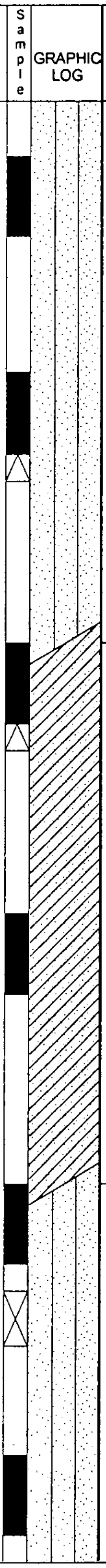
COLD SPRINGS, NEVADA

LOG OF B-3

PLATE

7

Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
				DARK BROWN SILTY SAND (SM) moist, medium dense, low plastic fines, fine to coarse sand.
		27		
120	11.1			
		25		With lense of low to medium plasticity clay
		37		**DARK BROWN CLAYEY SAND (SC)** moist, dense, low to medium plastic fines, fine to coarse sand.
109	11.3			
113	12.6	35		
		37		**DARK BROWN SILTY SAND (SM)** moist, dense, non-plastic fines, lenses of low to medium plasticity.
		13		Medium dense
106	9.4		25	



DATE: **07-24-03**
TOTAL DEPTH: **51.5 feet**
DIAMETER: **6 inch**

LOGGED BY: **J. RUZICKA**
EQUIPMENT: **CME 55 AUGER**
ELEVATION **Approx. 5079 feet**

KLEINFELDER

PROJECT NO. **33247.01**

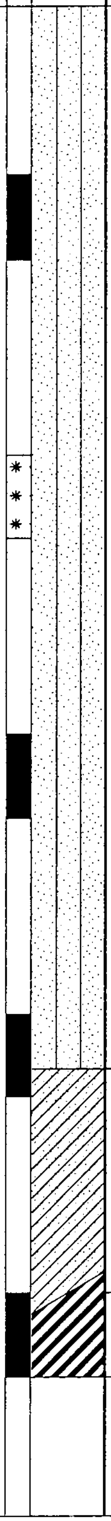
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-4

PLATE

8



DEPTH IN FEET

Depth	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	SOIL DESCRIPTION
28						
30	108	11.7	37		■	Dense
32						
34						
36			26		* * *	Medium dense, gravel grinding at 37'
38						
40		▽	72		■	Wet, very dense, fine to medium sand
42						
44						
46			91		■	**DARK BROWN CLAYEY SAND (SC)** wet, very dense, low to medium plastic fines, fine to coarse sand.
48						
50			31		■	**DARK BROWN TO RED FAT CLAY (CH)** wet, very stiff, medium to highly plastic fines, fine to medium sand. pp = 1.25 tsf (pocket penetrometer)
52						
54						

DATE: 07-24-03
TOTAL DEPTH: 51.5 feet
DIAMETER: 6 inch

LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 5079 feet

KLEINFELDER

PROJECT NO. 33247.01

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

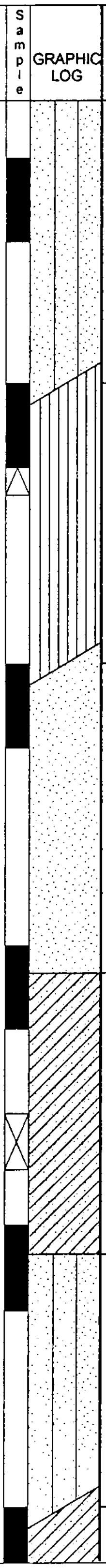
LOG OF B-4

PLATE

8

DEPTH IN FEET: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26

Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
		29		**DARK BROWN SILTY SAND (SM)** moist, medium dense, low plastic fines, fine to coarse sand.
90	6.4	18		**LIGHT TAN SANDY SILT (ML)** moist, very stiff, non-plastic fines, fine sand.
		68		**RED BROWN SAND (SP)** moist, very dense, non-plastic fines, medium to coarse sand.
98	22.4	26		**GRAY TO DARK BROWN CLAYEY SAND (SC)** moist, medium dense, low to medium plastic fines, fine to coarse sand.
		33		**LIGHT BROWN SILTY SAND (SM)** moist, medium dense, non-plastic fines, fine to medium sand.
83	20.5	35		**REDDISH BROWN CLAYEY SAND (SC)** moist, dense, low plastic fines, fine to medium sand.



DATE: 07-24-03
TOTAL DEPTH: 46.5 feet
DIAMETER: 6 inch
LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 5078 feet

KLEINFELDER

PROJECT NO. 33247.01

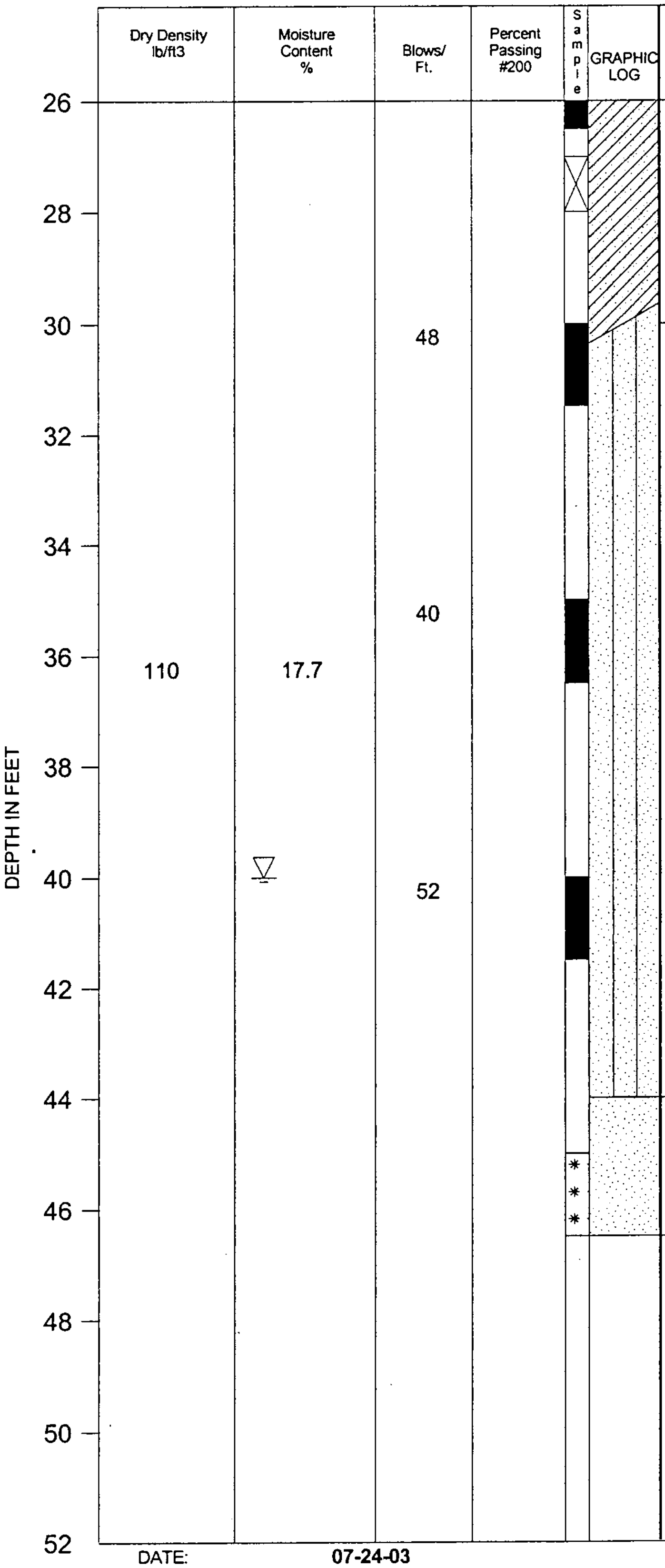
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-5

PLATE

9



SOIL DESCRIPTION

DARK BROWN SILTY SAND (SM) moist, dense, non-plastic fines, fine to medium sand, lense of low plasticity clay.

DARK BROWN SAND (SP) wet, non-plastic fines, fine to coarse sand, heaving sand.

DATE: 07-24-03
TOTAL DEPTH: 46.5 feet
DIAMETER: 6 inch

LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 5078 feet

KLEINFELDER

PROJECT NO. 33247.01

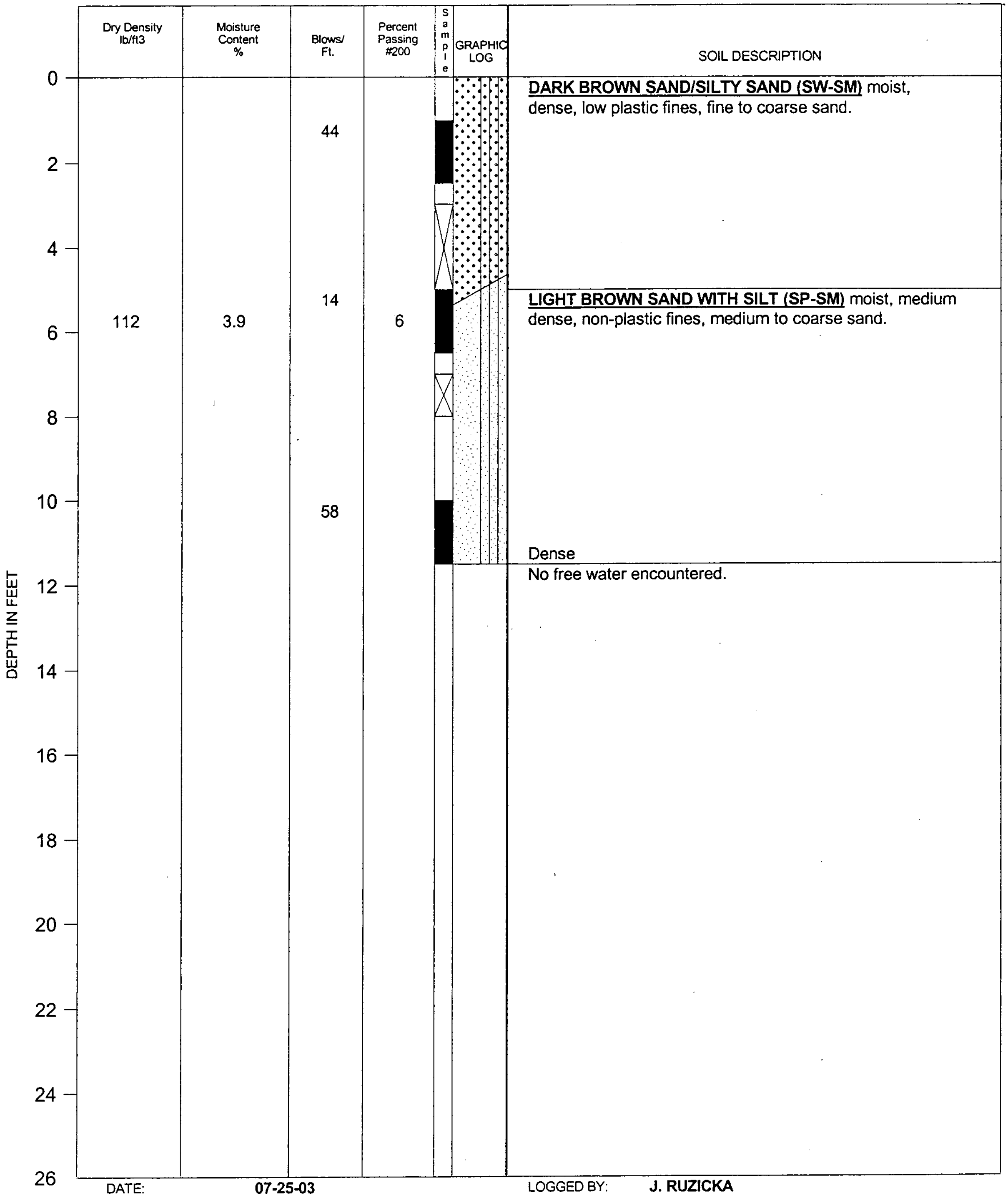
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-5

PLATE

9



Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
				DARK BROWN SAND/SILTY SAND (SW-SM) moist, dense, low plastic fines, fine to coarse sand.
		44		
		14		**LIGHT BROWN SAND WITH SILT (SP-SM)** moist, medium dense, non-plastic fines, medium to coarse sand.
112	3.9		6	
		58		
				Dense
				No free water encountered.

DATE: **07-25-03**
TOTAL DEPTH: **11.5 feet**
DIAMETER: **6 inch**

LOGGED BY: **J. RUZICKA**
EQUIPMENT: **CME 55 AUGER**
ELEVATION **Approx. 5045 feet**

KLEINFELDER

PROJECT NO. **33247.01**

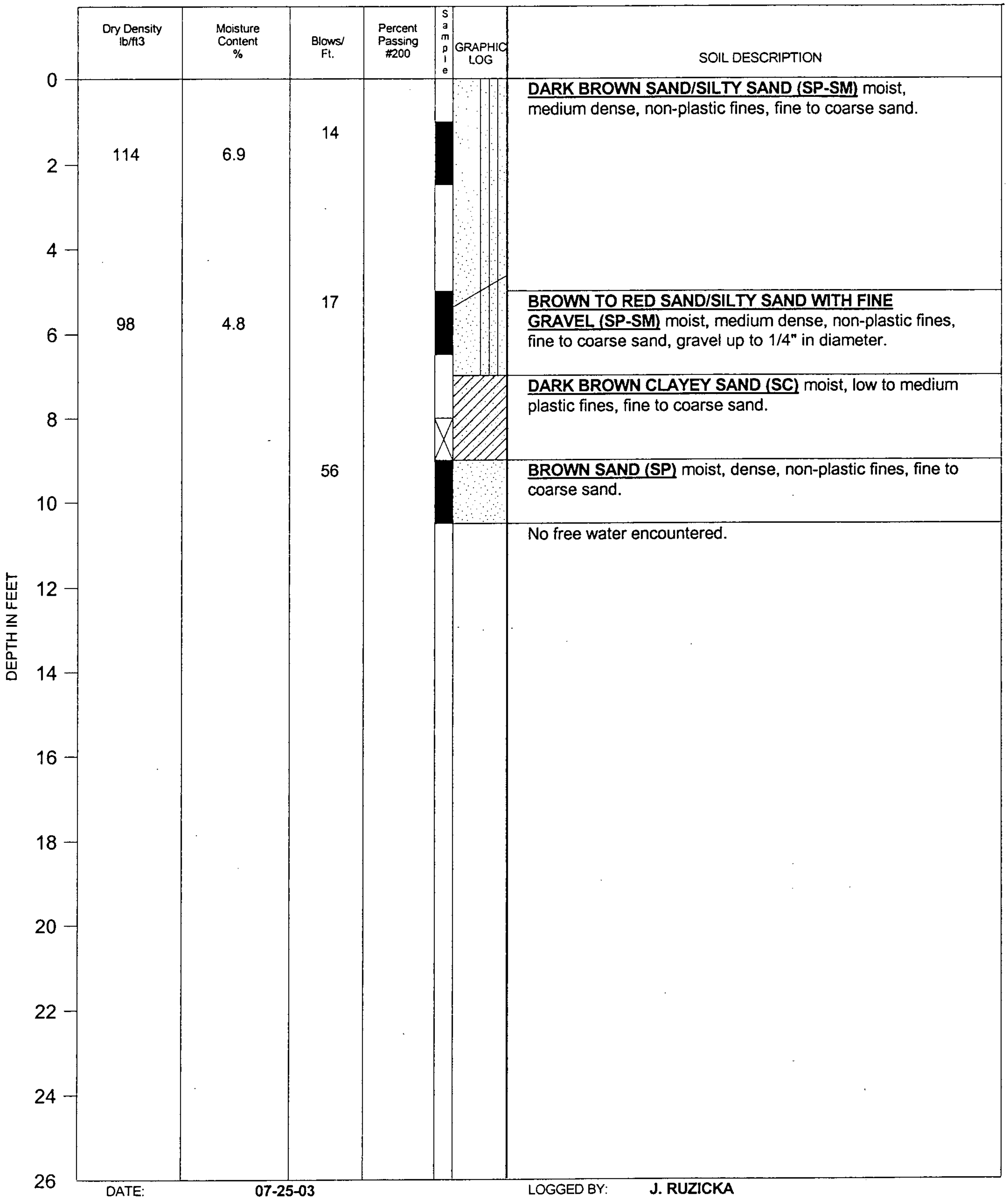
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-6

PLATE

10



Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
114	6.9	14		**DARK BROWN SAND/SILTY SAND (SP-SM)** moist, medium dense, non-plastic fines, fine to coarse sand.
98	4.8	17		**BROWN TO RED SAND/SILTY SAND WITH FINE GRAVEL (SP-SM)** moist, medium dense, non-plastic fines, fine to coarse sand, gravel up to 1/4" in diameter.
				DARK BROWN CLAYEY SAND (SC) moist, low to medium plastic fines, fine to coarse sand.
		56		**BROWN SAND (SP)** moist, dense, non-plastic fines, fine to coarse sand.
				No free water encountered.

DATE:	07-25-03	LOGGED BY:	J. RUZICKA
TOTAL DEPTH:	10.5 feet	EQUIPMENT:	CME 55 AUGER
DIAMETER:	6 inch	ELEVATION	Approx. 5047 feet

KLEINFELDER

PROJECT NO. 33247.01

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-7

PLATE

11

Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
				LIGHT TAN SAND (SP) moist, loose, non-plastic fines, fine to coarse sand.
		10		
98	2.5	14	2.5	Medium dense
		27		**DARK BROWN SANDY CLAY (CL)** moist, very stiff, medium plastic fines, fine to medium sand.
				No free water encountered.

DEPTH IN FEET

DATE: **07-25-03**
TOTAL DEPTH: **10.5 feet**
DIAMETER: **6 inch**

LOGGED BY: **J. RUZICKA**
EQUIPMENT: **CME 55 AUGER**
ELEVATION **Approx. 5065 feet**

KLEINFELDER

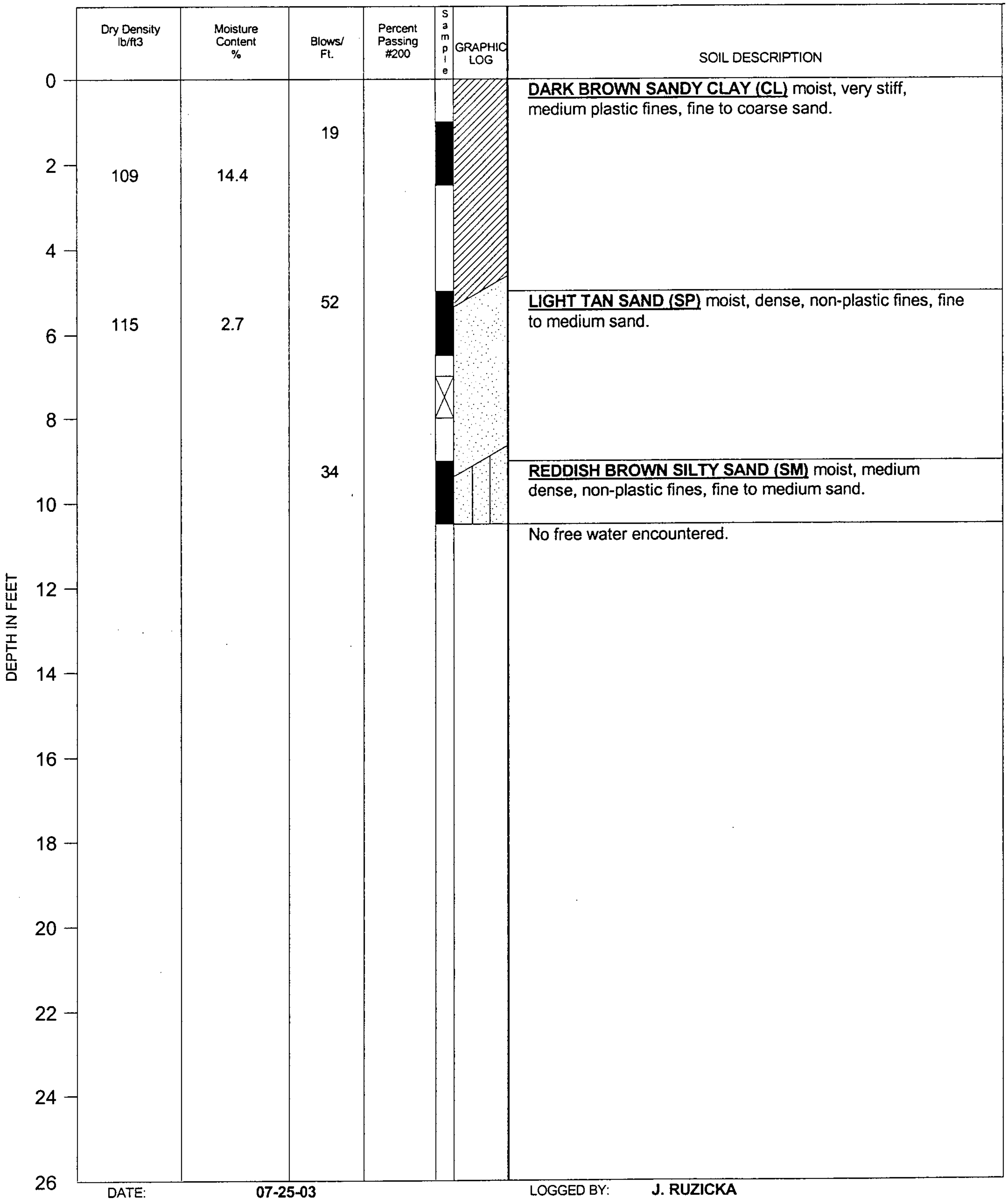
PROJECT NO. **33247.01**

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-8

PLATE
12



Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200
		19	
109	14.4		
		52	
115	2.7		
		34	

SOIL DESCRIPTION

DARK BROWN SANDY CLAY (CL) moist, very stiff, medium plastic fines, fine to coarse sand.

LIGHT TAN SAND (SP) moist, dense, non-plastic fines, fine to medium sand.

REDDISH BROWN SILTY SAND (SM) moist, medium dense, non-plastic fines, fine to medium sand.

No free water encountered.

DATE: **07-25-03**
TOTAL DEPTH: **10.5 feet**
DIAMETER: **6 inch**

LOGGED BY: **J. RUZICKA**
EQUIPMENT: **CME 55 AUGER**
ELEVATION **Approx. 5066 feet**

KLEINFELDER

PROJECT NO. **33247.01**

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

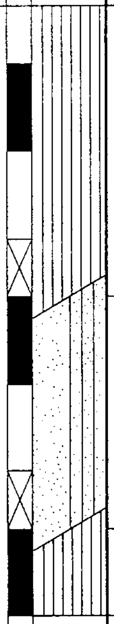
LOG OF B-9

PLATE
13

Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	GRAPHIC LOG	SOIL DESCRIPTION
		43				**LIGHT BROWN SANDY SILT (ML)** moist, hard, non to low plastic fines, fine sand.
114	2.9	15	8			**LIGHT BROWN SAND/SILTY SAND (SP-SM)** moist, medium dense, non-plastic fines, fine to coarse sand.
		40				**LIGHT BROWN SANDY SILT (ML)** moist, very stiff, low plastic fines, fine sand.
						No free water encountered.

DEPTH IN FEET

0 2 4 6 8 10 12 14 16 18 20 22 24 26



DATE: 07-25-03

TOTAL DEPTH: 10.5 feet

DIAMETER: 6 inch

LOGGED BY: J. RUZICKA

EQUIPMENT: CME 55 AUGER

ELEVATION Approx. 5066 feet

KLEINFELDER

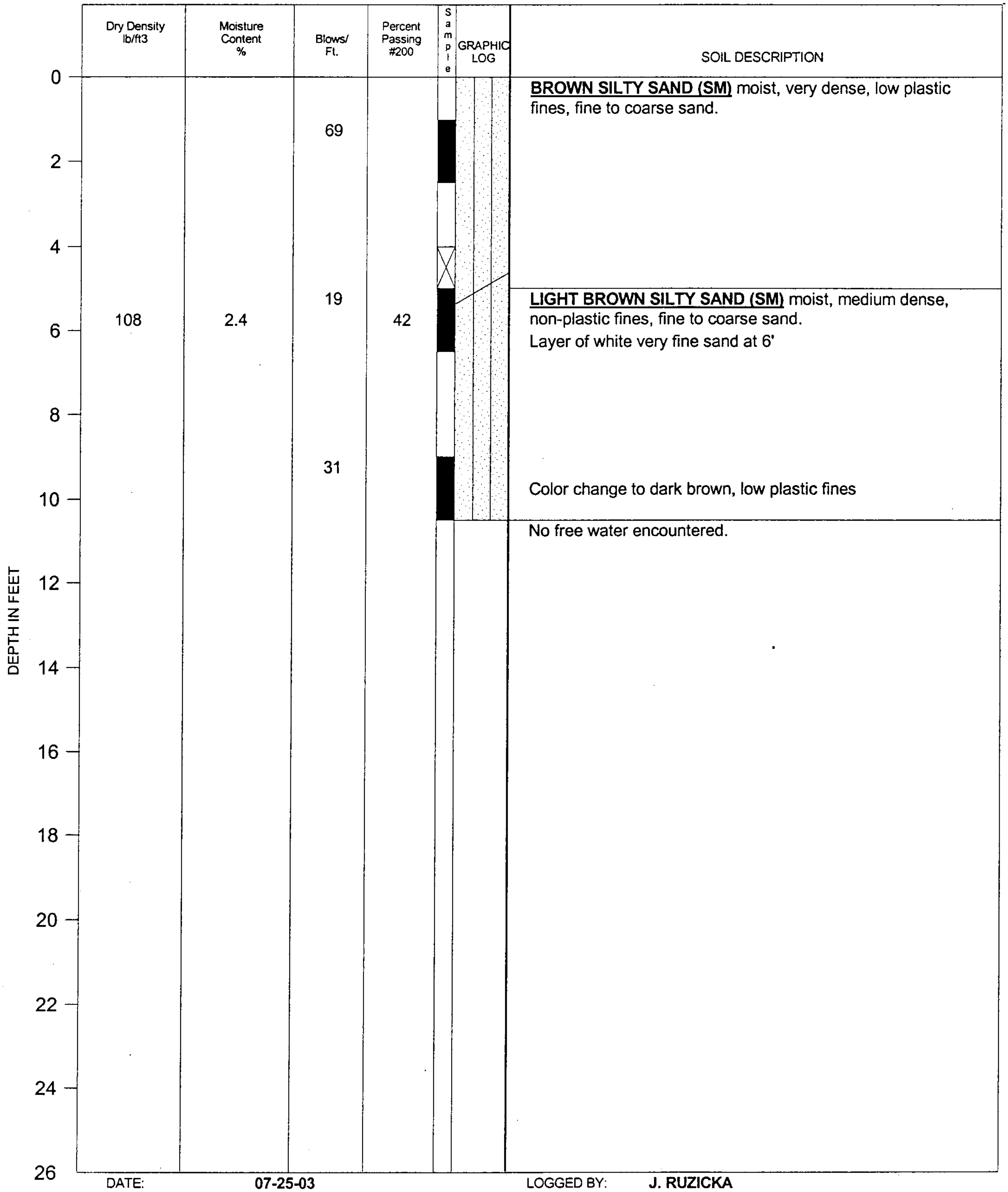
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-10

PLATE **14**

PROJECT NO. 33247.01



Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
				BROWN SILTY SAND (SM) moist, very dense, low plastic fines, fine to coarse sand.
		69		
		19		**LIGHT BROWN SILTY SAND (SM)** moist, medium dense, non-plastic fines, fine to coarse sand.
108	2.4		42	Layer of white very fine sand at 6'
		31		
				Color change to dark brown, low plastic fines
				No free water encountered.

DATE: 07-25-03
TOTAL DEPTH: 10.5 feet
DIAMETER: 6 inch

LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 5069 feet

KLEINFELDER

PROJECT NO. 33247.01

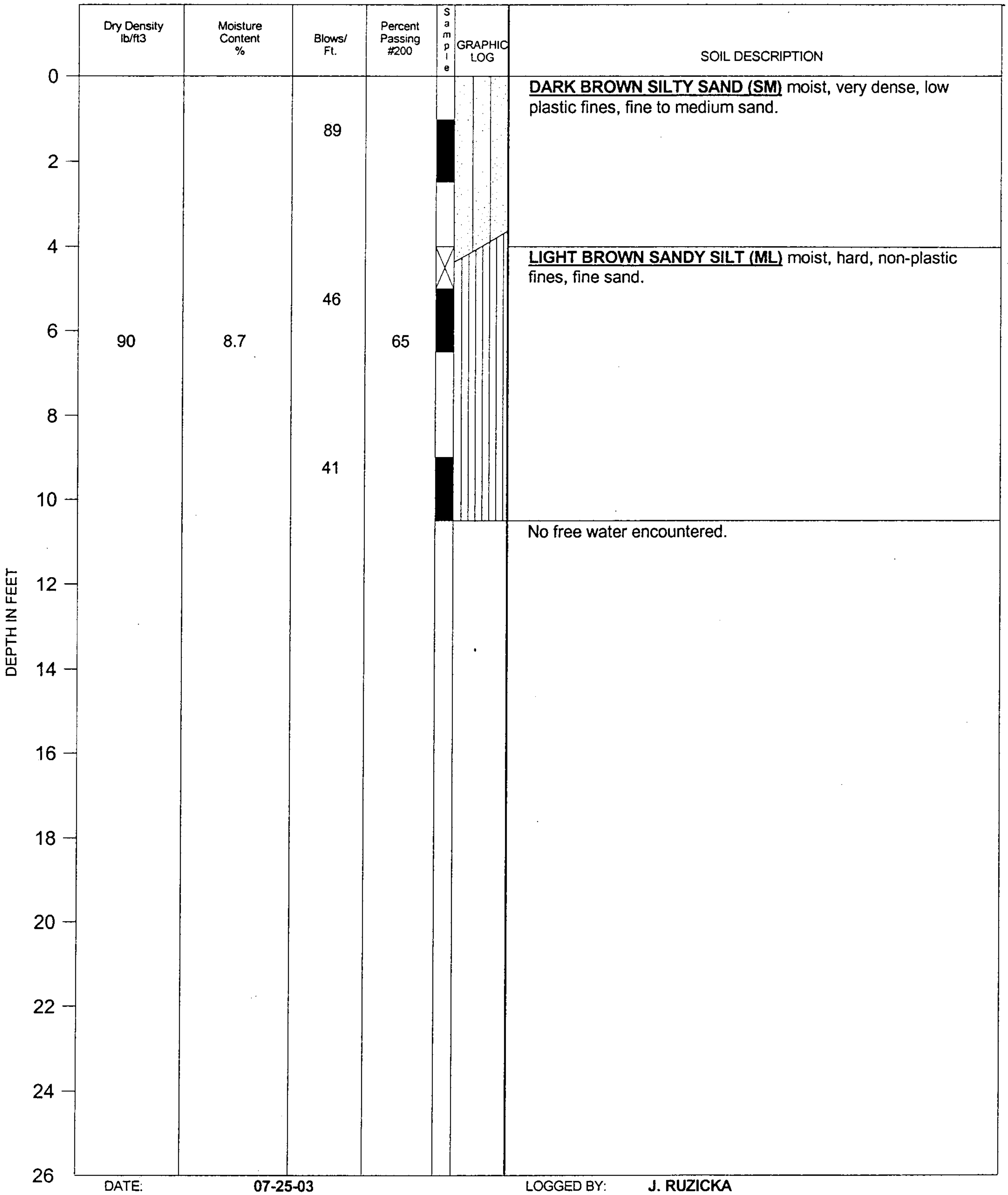
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-11

PLATE

15



DEPTH IN FEET	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
0					**DARK BROWN SILTY SAND (SM)** moist, very dense, low plastic fines, fine to medium sand.
1.3			89		
4					**LIGHT BROWN SANDY SILT (ML)** moist, hard, non-plastic fines, fine sand.
5.3			46		
6	90	8.7		65	
9.3			41		
10.5					No free water encountered.

DATE: 07-25-03
TOTAL DEPTH: 10.5 feet
DIAMETER: 6 inch

LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 5072 feet

KLEINFELDER

PROJECT NO. 33247.01

COLD SPRINGS WASTEWATER TREATMENT PLANT

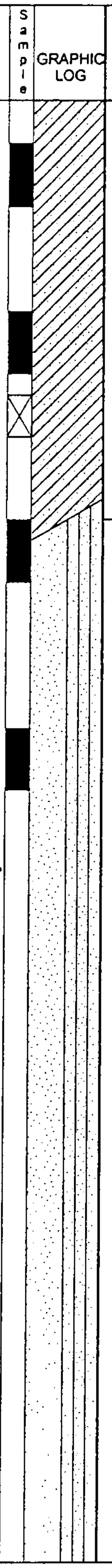
COLD SPRINGS, NEVADA

LOG OF B-12

PLATE

16

DEPTH IN FEET	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
0					**LIGHT BROWN TO GRAY CLAYEY SAND (SC)** dense, low to medium plastic fines, fine to coarse sand.
			39		
			16		
6	101	19.0			Medium dense
		∇			
10			12		**GRAY BROWN SAND/SILTY SAND (SP-SM)** wet, loose, non-plastic fines, fine to coarse sand.
	116	12.9		10	
					Heaving sand up to 3'
16	124	12.2		5	



DATE: 07-25-03
TOTAL DEPTH: 35.0 feet
DIAMETER: 6 inch

LOGGED BY: J. RUZICKA
EQUIPMENT: CME 55 AUGER
ELEVATION Approx. 5035 feet

KLEINFELDER

PROJECT NO. 33247.01

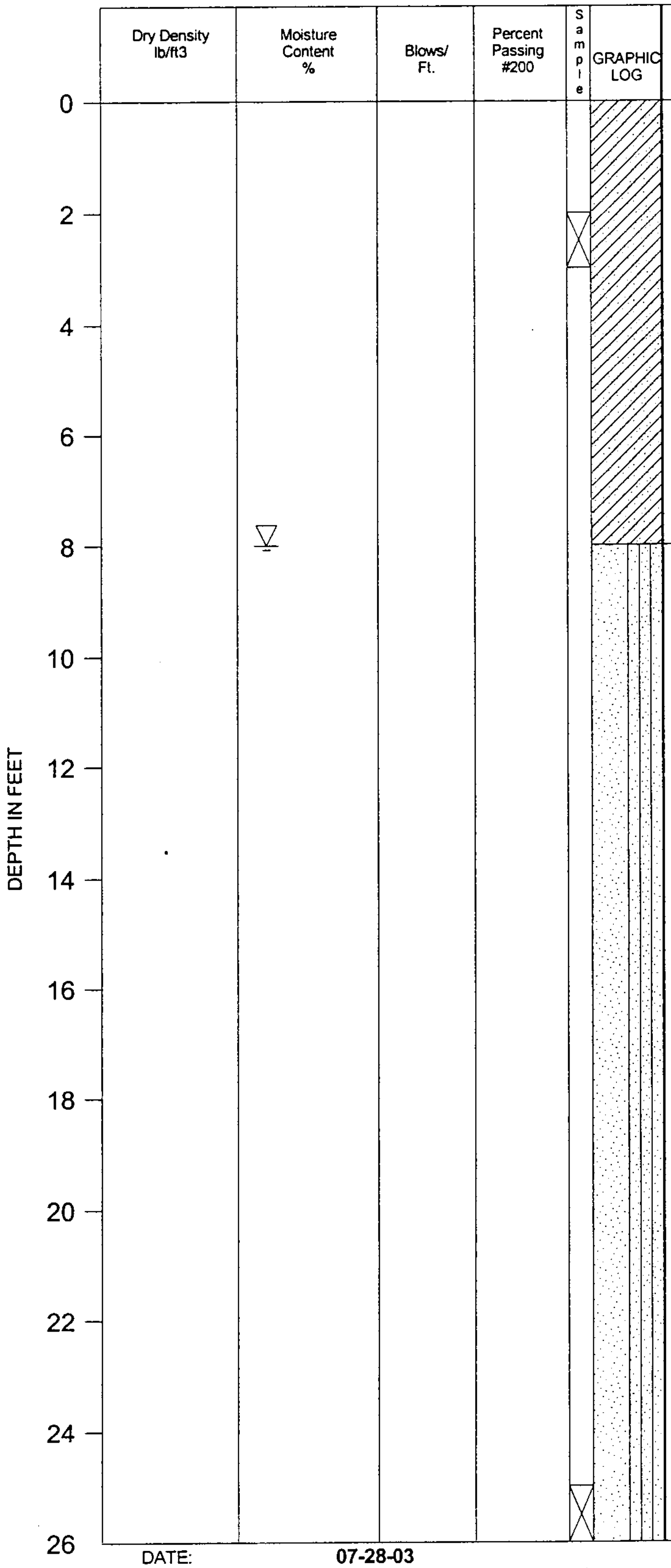
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-13

PLATE

17



SOIL DESCRIPTION

BROWN CLAYEY SAND (SC) moist, low plastic fines, fine to coarse sand.

BROWN SAND/SILTY SAND (SP-SM) wet, non-plastic fines, fine to coarse sand.

DATE:	07-28-03	LOGGED BY:	J. RUZICKA
TOTAL DEPTH:	50.0 feet	EQUIPMENT:	CME 55 AUGER
DIAMETER:	6 inch	ELEVATION	Approx. 5036 feet

KLEINFELDER

COLD SPRINGS WASTEWATER TREATMENT PLANT

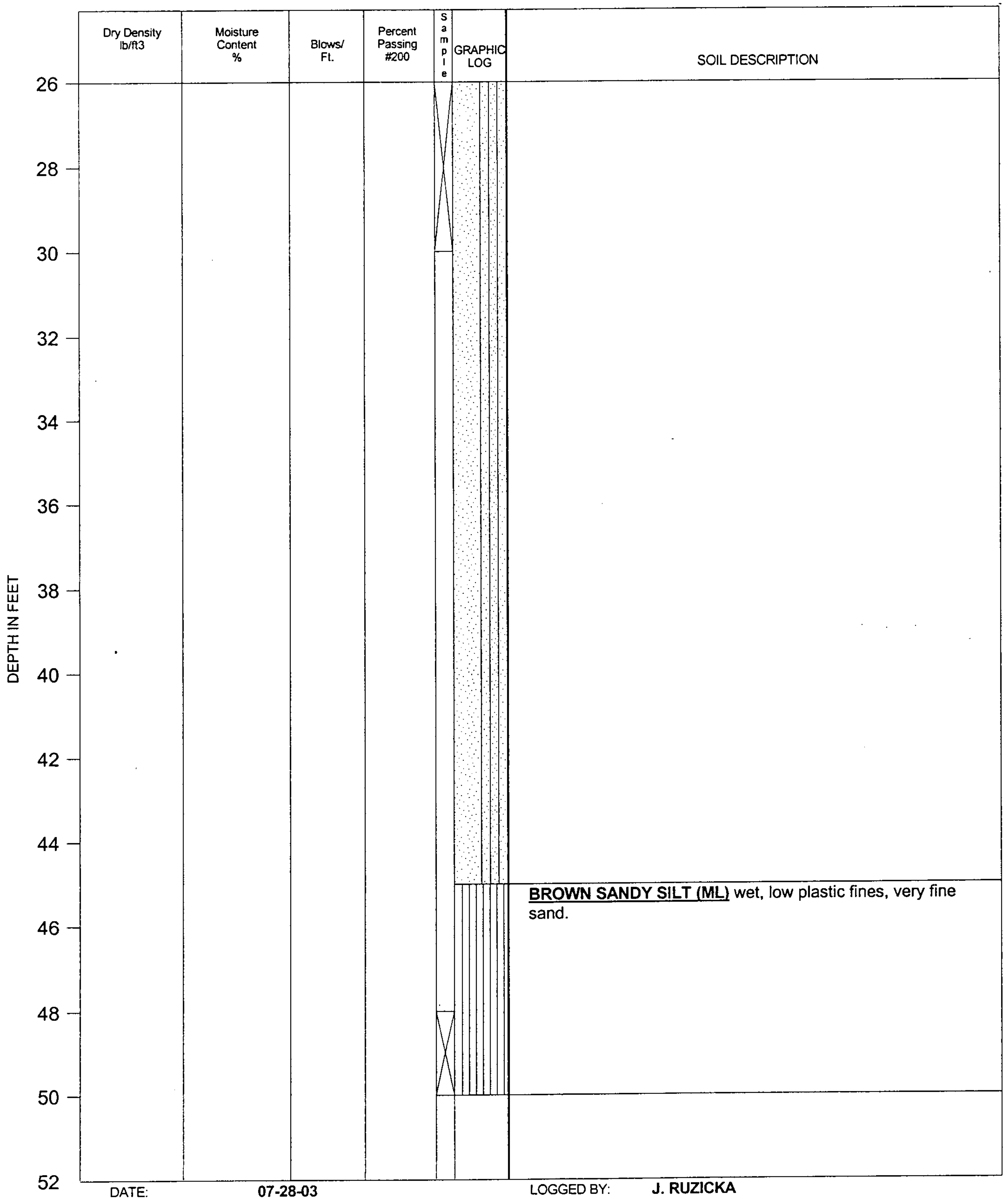
COLD SPRINGS, NEVADA

LOG OF B-14

PLATE

18

PROJECT NO. **33247.01**



BROWN SANDY SILT (ML) wet, low plastic fines, very fine sand.

DATE:	07-28-03	LOGGED BY:	J. RUZICKA
TOTAL DEPTH:	50.0 feet	EQUIPMENT:	CME 55 AUGER
DIAMETER:	6 inch	ELEVATION	Approx. 5036 feet

KLEINFELDER

PROJECT NO. 33247.01

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

LOG OF B-14

PLATE

18

THE UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOIL More than 50% of the material is LARGER than the No. 200 sieve.	GRAVELS More than 50% of coarse part is LARGER than the No. 4 Sieve.	CLEAN GRAVELS Less than 5% finer than No. 200 Sieve.	PI<4	**GW**	Well graded gravels, gravel – sand mixtures, little or no fines, Cu>4 & 1<Cc>3
			PI>7	**GP**	Poorly graded gravels or gravel – sand mixtures, little or no fines Cu<4 or 1>Cc<3
		GRAVEL More than 12% finer than No. 200 Sieve.		**GM**	Silty gravels, gravel – sand – silt mixtures
				GC	Clayey gravels, gravel – sand – clay mixtures
	SANDS More than 50% of coarse part is SMALLER than the No. 4 Sieve.	CLEAN SANDS Less than 5% finer than No. 200 Sieve.		**SW**	Well graded sands, gravelly sands, little or no or no fines, Cu>6 & 1<Cc>3
				SP	Poorly graded sands or gravelly sands, little or no fines Cu<6 or 1>Cc<3
		SAND More than 12% finer than No. 200 Sieve.	PI<4	**SM**	Silty sands, sand – silt mixtures
			PI>7	**SC**	Clayey sands, sand – clay mixtures
FINE GRAINED SOIL More than 50% of the material is SMALLER than the No. 200 sieve.	SILTS AND CLAYS Liquid limit LESS than 50		PI–Below A–Line	**ML**	Inorganic silts, rock flour, or clayey silts of low plasticity
			PI–Above A–Line	**CL**	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts & organic clays of low plasticity
	SILTS AND CLAYS Liquid limit GREATER than 50		PI–Below A–Line	**MH**	Inorganic silts, clayey silts, or silts of high plasticity
			PI–Above A–Line	**CH**	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				**PT**	Peat & other highly organic soils

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		Coarse	Fine	Coarse	Medium	Fine		

12" 3" 3/4" #4 #10 #40 #200 0.002 mm

DESCRIPTIVE TERMS USED WITH SOILS

CONSISTENCY & APPARENT DENSITY	SILTS & CLAYS	SANDS & GRAVELS
Strongest	Hard	Very Dense
	Very Stiff	Dense
	Stiff	Medium Dense
	Medium Stiff	Loose
Weakest	Soft	Very Loose
	Very Soft	

MOISTURE CONTENT	
Wettest	Wet
	Very Moist
	Moist
	Slightly Moist
Driest	Dry

∇ – Water Level Observed During Exploration

▼ – Water Level Observed After Exploration

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 33247.01

KEY TO SOIL CLASSIFICATION AND TERMS

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

PLATE
19

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SYMBOLS

☒	Disturbed Bag or Bulk Sample
◩	Standard Penetration Sample (1.4 inch I.D., 2.0 inch O.D.)
▮	Modified California (Porter) Sample (2.0 inch I.D., 2.56 inch O.D.)
*	No Sample Recovery
▽	Water Level Observed During Drilling
▼	Water Level Observed After Drilling

COMMENTS

NOTE: Blow count represents the number of blows required to drive a sampler through the last 12 inches of an 18 inch penetration. A standard 140 pound hammer with a 30.4 inch free fall is used to drive the sampler.

NOTE: The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings.

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

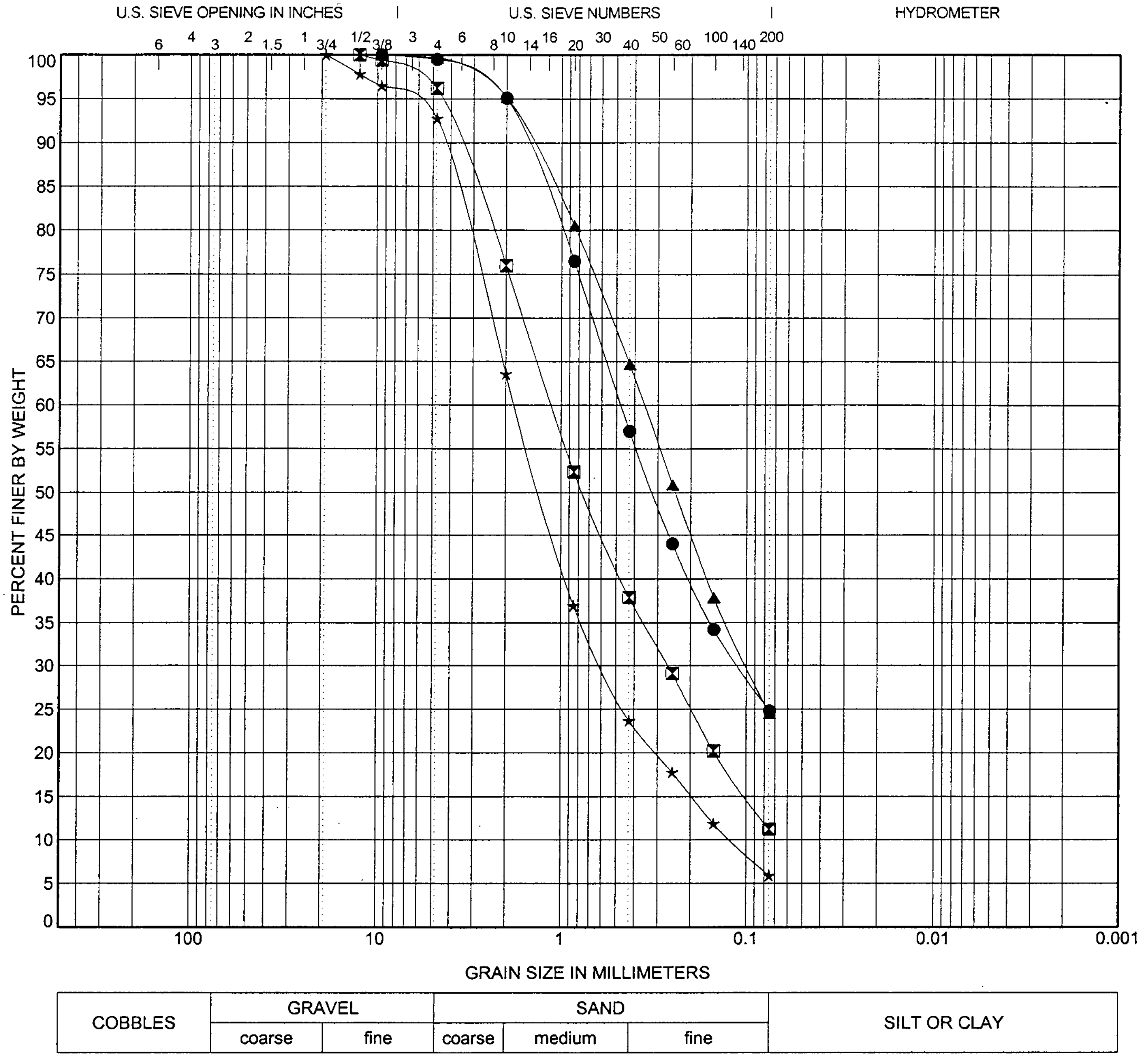
PROJECT NO. 33247.01

KEY TO BORING LOGS

COLD SPRINGS WASTEWATER TREATMENT PLANT
COLD SPRINGS, NEVADA

PLATE
20

CAD FILE: L:\2003\DRAFTING\33247\33247-USCS.dwg



	Boring	Depth (ft.)	Description - ASTM Classification	LL	PL	PI	Cc	Cu
●	B-1	25.5	Gray Clayey Sand (SC)					
☒	B-2	30.5	Brown Red Silty Sand (SP-SM)				0.91	16.37
▲	B-4	25.5	Dark Brown Silty Sand (SM)					
★	B-6	5.5	Light Brown Silty Sand (SW-SM)				1.62	14.79

	Boring	Depth (ft.)	D100	D60	D30	D10	% Gravel	% Sand	% Silt	% Clay
●	B-1	25.5	9.5	0.473	0.11		0.5	74.7	24.8	
☒	B-2	30.5	12.5	1.12	0.264		3.8	85.0	11.2	
▲	B-4	25.5	9.5	0.355	0.1		0.3	75.2	24.5	
★	B-6	5.5	19	1.781	0.59	0.12	7.2	86.9	5.9	

KLEINFELDER

PROJECT NUMBER: 33247.01

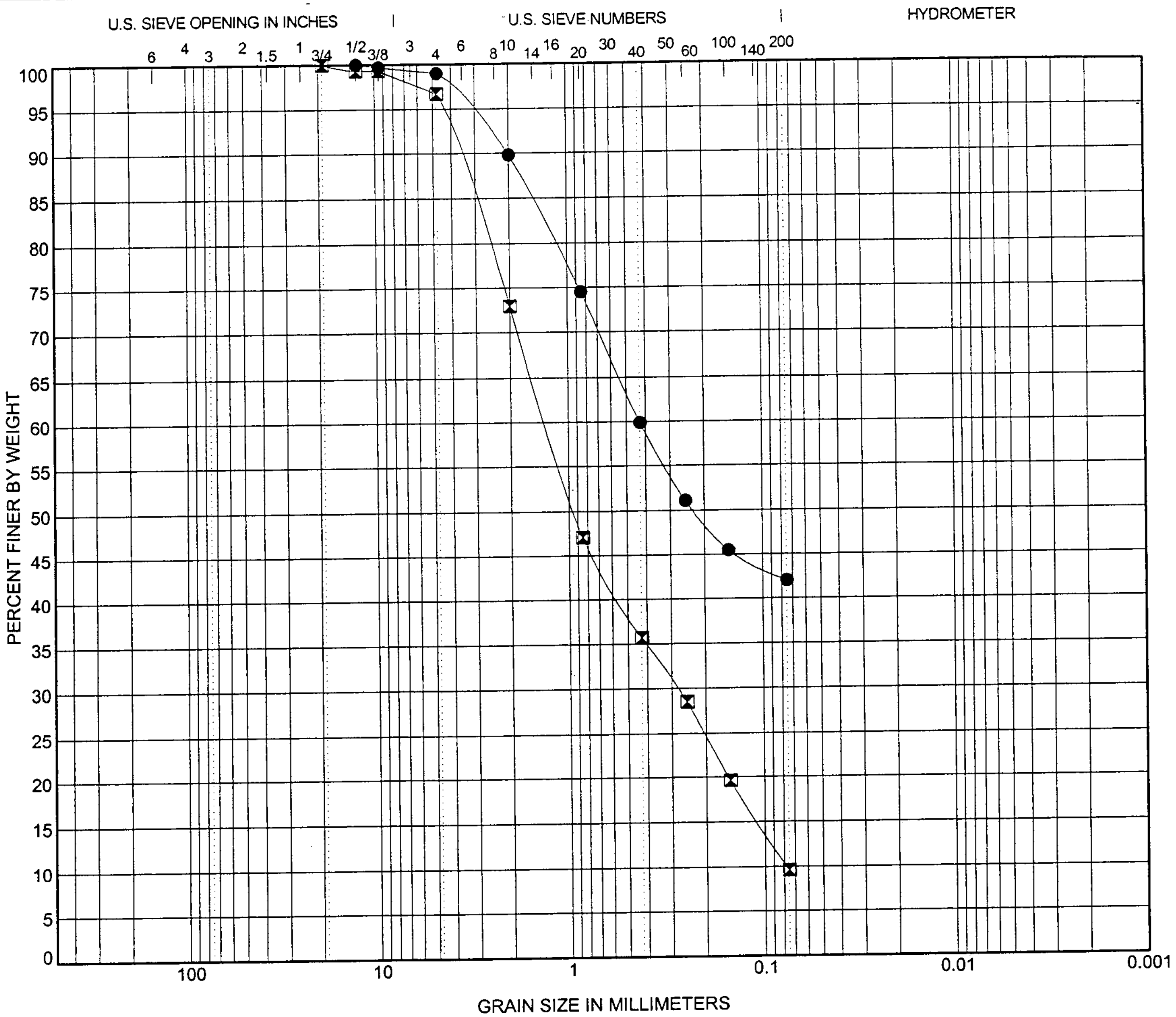
COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

GRAIN SIZE ANALYSES

PLATE

21



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Depth (ft.)	Description - ASTM Classification	LL	PL	PI	Cc	Cu
●	**B-11**	**5.5**	**Light Brown Silty Sand (SM)**					
☒	**B-13**	**10.5**	**Gray Brown Silty Sand (SP-SM)**				**0.78**	**16.91**

	Boring	Depth (ft.)	D100	D60	D30	D10	% Gravel	% Sand	% Silt	% Clay
●	**B-11**	**5.5**	**12.5**	**0.427**			**1.0**	**57.0**	**42.0**	
☒	**B-13**	**10.5**	**19**	**1.304**	**0.279**	**0.077**	**3.3**	**87.1**	**9.6**	

KLEINFELDER

PROJECT NUMBER: 33247.01

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

GRAIN SIZE ANALYSES

PLATE

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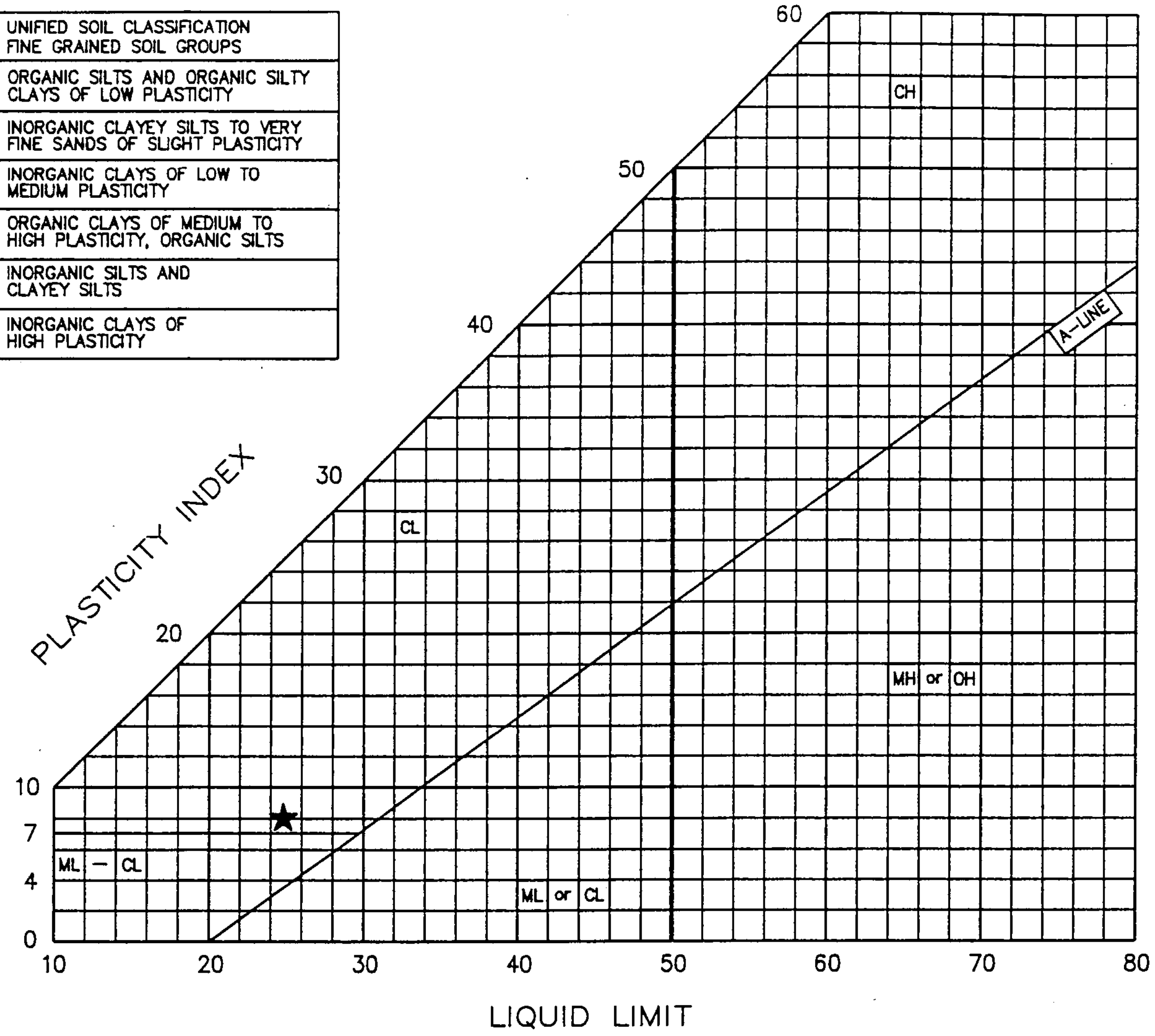
GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILTS
CH	INORGANIC CLAYS OF HIGH PLASTICITY



TEST SYMBOL	SAMPLE NO.	SAMPLE (DEPTH)	LIQUID LIMIT	PLASTICITY INDEX	CLASSIFICATION
★	B-1	25.5-26'	25	8	Gray Clayey Sand (SC)
⬟	B-2	30.5-31'	NP	NP	Brown Red Silty Sand (SP-SM)
●	B-4	25.5-26'	NP	NP	Dark Brown Silty Sand (SM)
▲	B-12	5.5-6'	NP	NP	Light Brown Sandy Silt (ML)

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4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

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PLASTICITY INDEX

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PLATE

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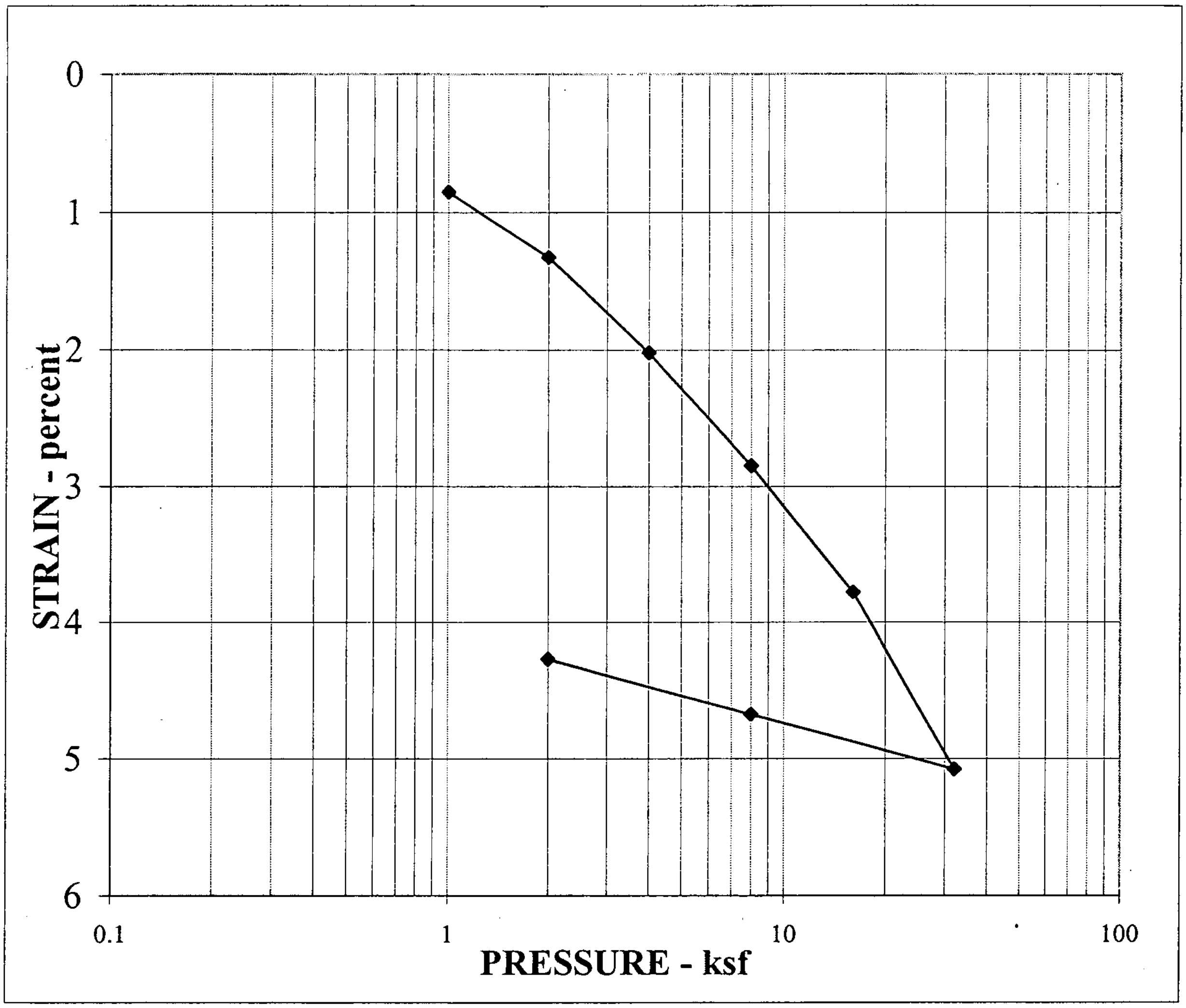

BORING NO.	B-3	DEPTH:	25.5 ft.
SAMPLE DESCRIPTION: Brown Silty Sand			
OVERBURDEN PRESSURE, psf:			3,000
PRECONSOLIDATION PRESSURE, psf :			7,000

	INITIAL	FINAL
DRY DENSITY - pcf	113.2	118.1
WATER CONTENT - %	6.1	15.0
VOID RATIO	0.4607	0.4018
DEGREE OF SATURATION, %	35.00	99.00
SAMPLE HEIGHT - inches	1.0000	0.9573

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CONSOLIDATION TEST

COLD SPRINGS WASTEWATER TREATMENT PLANT
COLD SPRINGS, NEVADA

PLATE
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STEEL CORROSION POTENTIAL OF SOILS*

Corrosion Resistance	Resistivity (ohm-cm)
Excellent	6,000 to 10,000
Good	4,500 to 6,000
Fair	2,000 to 4,500
Bad	0 to 2,000

LABORATORY TEST RESULTS

Soil Type	Source	Resistivity (ohm-cm)	pH**
SANDY SILT (ML)	B-1 @ 5.5-6 FT	4,300	8.44
SAND (SP)	B-1 @ 20.5-21 FT	10,000	8.23
SILTY SAND (SP-SM)	B-6 @ 5-5.5 FT	25,000	7.52
SILTY SAND (SP-SM)	B-10 @ 5-5.5 FT	4,000	7.88
SILTY SAND (SP-SM)	B-13 @ 15-15.5 FT	4,400	7.88

* Reference: "Accelerated Corrosion Tests for Buried Metal Structures", by Paul Lieberman, Ph.D., in Pipeline and Gas Journal October, 1996, Pg.51

** Note: Corrosion potential of soils generally increases as pH decreases below 7.

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STEEL CORROSION POTENTIAL OF SOIL

COLD SPRINGS WASTEWATER TREATMENT PLANT
COLD SPRINGS, NEVADA

PLATE
25

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POTENTIAL REACTIVITY OF SOLUBLE SULFATES IN SOIL OR GROUNDWATER WITH PORTLAND CEMENT CONCRETE

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

SULFATE EXPOSURE	WATER-SOLUBLE SULFATE (SO4) IN SOIL, PERCENTAGE BY WEIGHT	SULFATE (SO4) IN WATER, ppm	CEMENT TYPE	MAXIMUM WATER-CEMENTITIOUS MATERIALS RATIO, BY WEIGHT, NORMAL-WEIGHT AGGREGATE CONCRETE (1)	MINIMUM NORMAL-WEIGHT AND LIGHTWEIGHT AGGREGATE CONCRETE, psi x 0.00689 for MP
Negligible	0.00-0.10	0-150	(Negligible..............................	...........Sulfate...........	..........................Reaction)
Moderate (2)	0.10-0.20	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00	Over 10,000	V plus pozzolan (3)	0.45	4,500

(1) A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing.

(2) Seawater.

(3) Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Reference: 1997 Uniform Building Code

SAMPLE IDENTIFICATION	B-1 @ 5.5-6 FT	B-1 @ 20.5-21 FT	B-6 @ 5-5.5 FT	B-10 @ 5-5.5 FT	B-13 @ 15-15.5 FT
SAMPLE DESCRIPTION	SANDY SILT (ML)	SAND (SP)	SILTY SAND (SP-SM)	SILTY SAND (SP-SM)	SILTY SAND (SP-SM)
SOLUBLE SULFATE (ppm)	<15	<15	<15	<15	<15
COMMENTS	NEGLIGIBLE SULFATE REACTION	NEGLIGIBLE SULFATE REACTION	NEGLIGIBLE SULFATE REACTION	NEGLIGIBLE SULFATE REACTION	NEGLIGIBLE SULFATE REACTION

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POTENTIAL REACTIVITY

COLD SPRINGS WASTEWATER TREATMENT PLANT
COLD SPRINGS, NEVADA

PLATE
26

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Kennedy/Jenks Consultants
Lynn Orphan
5190 Neil Road, Suite 210
Reno, NV 89502

Date: August 20, 2003
Job #: 33247
Phase: 01

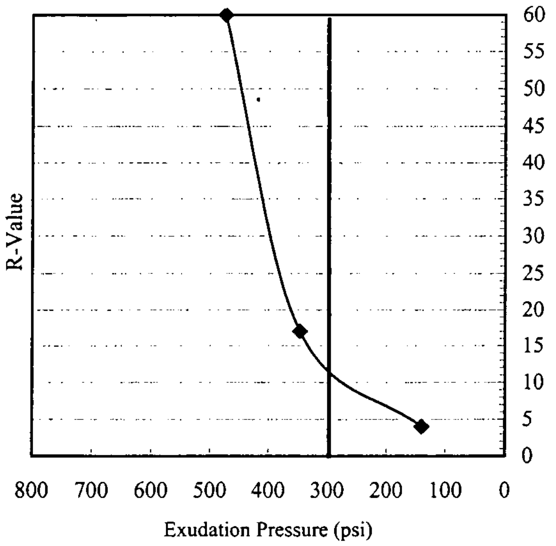
SAMPLE IDENTIFICATION:

Lab No.: 6442
Date Sampled: 12-Aug-03
Date Received: 13-Aug-03
Project: Cold Springs Waste Water Treatment Plant
Material Supplier: Native
Material Type: Native Subgrade
Sample Location: TP-4 0-1 ft.

Sampled By: Jesse Ruzicka
Received By: John Ruckman
Tested By: Nathan Morian
Date Tested: 15-Aug-03
Material Source: Native

TEST RESULTS:

EXUDATION PRESSURE CHART



Specimen	A	B	C
Exudation Pressure, psi	475	347	140
Expansion Dial (.0001")	0.0000	0.0000	0.0000
Expansion Pressure, psf	0	0	0
Resistance Value, R	60	17	4
% Moisture at Test	10.4	11.5	12.5
Dry Density at Test, pcf	124.9	123.7	120.8
R-Value at 300 psi Exudation		12	

Submitted By: Nathan Morian, E.I.
Reviewed By: Mark Doehring, P.E.
Date: 8/20/2003

As a mutual protection to our clients, the public, and ourselves, all reports are submitted as the confidential property of our clients, and authorization of statements, conclusions, or extracts from or regarding our reports pending our written approval. Samples will be disposed of after testing is completed unless prior arrangements are agreed to in writing.

R-VALUE

COLD SPRINGS WASTEWATER TREATMENT PLANT
COLD SPRINGS, NEVADA

PLATE
27

APPENDIX B

Slug Test Results and Calculations

1) BOUWER & RICE EQUATION

$$K = \frac{Rc*Rc * \ln(Re/Rw)}{2*Le} * \frac{1}{T} * \ln(Y1/Y2)$$

A) For partially penetrating wells the term ln(Re/Rw) is:

$$\ln(Re/Rw) = \frac{1}{\left(\frac{1.1}{\ln(Lw/Rw)} + \frac{A + B * \ln\{(H - Lw)/Rw\}}{Le/Rw} \right)}$$

B) For fully penetrating wells the term ln(Re/Rw) is:

$$\ln(Re/Rw) = \frac{1}{\left(\frac{1.1}{\ln(Lw/Rw)} + \frac{C}{Le/Rw} \right)}$$

where K = hydraulic conductivity
Rc = radius of well/screen casing
Re = effective radial distance over which delta-y is dissipated
Rw = borehole radius
H = saturated thickness of aquifer
A = the Bouwer and Rice 'A' parameter
B = the Bouwer and Rice 'B' parameter
C = the Bouwer and Rice 'C' parameter
Lw = depth below water table to bottom of screen
Le = length of wetted screen
Y1 = drawdown (or up) at time T1
Y2 = drawdown (or up) at time T2
T = time between T1 and T2

2) U.S. NAVY EQUATION FOR A CASED BOREHOLE WITH SCREEN

$$K = \frac{Rc * Rc}{2 * Le * T} * \ln(Le/Rw) * \ln(Y1/Y2)$$

3) HVORSLEV EQUATION FOR A CASED BOREHOLE WITH SCREEN

$$K = \frac{Rc * Rc}{2 * Le * T} * \ln\left(\frac{h1}{h2}\right) * \ln\left(\frac{m * Le}{Rw}\right) \quad \text{for} \quad \frac{m * Le}{2 * Rw} > 4$$

m = square root of horizontal to vertical hydraulic conductivity ratio

If static water level occurs in screened interval Rc is adjusted:

$$Rc = \mathrm{SQRT}[(1-n)*Rc*RC + (n)*Rw*Rw]$$

where n is specific yield of sand/gravel pack in well annulus

```
VARIABLES for well:  1

                   Rc =   0.375  ft,  4.50 inches
                   Rw =   0.725  ft,  8.70 inches
        Screen Length =  23.25   ft
   Static water depth =   5.73   ft
    Screen base depth =  43.25   ft
   Aquifer base depth =  43.25   ft
   Aquifer sat. thk H =  37.521  ft
    Bouwer and Rice A =   2.553 (dimensionless)
    Bouwer and Rice B =   0.379 (dimensionless)
            ln(Re/Rw) =   3.565 (partially penetrating)
    Bouwer and Rice C =   1.978 (dimensionless)
            ln(Re/Rw) =   2.938 (fully penetrating)
                   Lw =  37.520  ft
                   Le =  23.250  ft
                   T1 =   0.1000 min
                   Y1 =   1.3390 ft
                   T2 =   0.8000 min
                   Y2 =   0.3280 ft
              DELTA T =   0.7000 minutes
    Kh/Kv anisotropy =  10.0000
                                              K       TRANSMISSIVITY
                                          (FT/DAY)     (FT^2/DAY)
Bouwer & Rice, 1976:                      --------     ----------
  Partially penetrating well:              31.199        1170.61
  Fully penetrating well:                  25.707         964.53
U.S. NAVY, 1974:                           30.348        1138.66
HVORSLEV, 1951:                            40.423        1516.67




VARIABLES for well:  2

                   Rc =   0.375  ft,  4.50 inches
                   Rw =   0.725  ft,  8.70 inches
        Screen Length =  15.42   ft
   Static water depth =   5.62   ft
    Screen base depth =  30.42   ft
   Aquifer base depth =  30.42   ft
   Aquifer sat. thk H =  24.801  ft
    Bouwer and Rice A =   2.219 (dimensionless)
    Bouwer and Rice B =   0.323 (dimensionless)
            ln(Re/Rw) =   3.166 (partially penetrating)
    Bouwer and Rice C =   1.570 (dimensionless)
            ln(Re/Rw) =   2.596 (fully penetrating)
                   Lw =  24.800  ft
                   Le =  15.420  ft
                   T1 =   0.0667 min
                   Y1 =   1.3830 ft
                   T2 =   0.5000 min
                   Y2 =   0.2300 ft
              DELTA T =   0.4333 minutes
    Kh/Kv anisotropy =  10.0000
                                              K       TRANSMISSIVITY
```

```
                                   (FT/DAY)    (FT^2/DAY)
Bouwer & Rice, 1976:               --------    ----------
  Partially penetrating well:        86.069       2134.60
  Fully penetrating well:            70.573       1750.21
U.S. NAVY, 1974:                     83.111       2061.15
HVORSLEV, 1951:                     114.409       2837.34



▯
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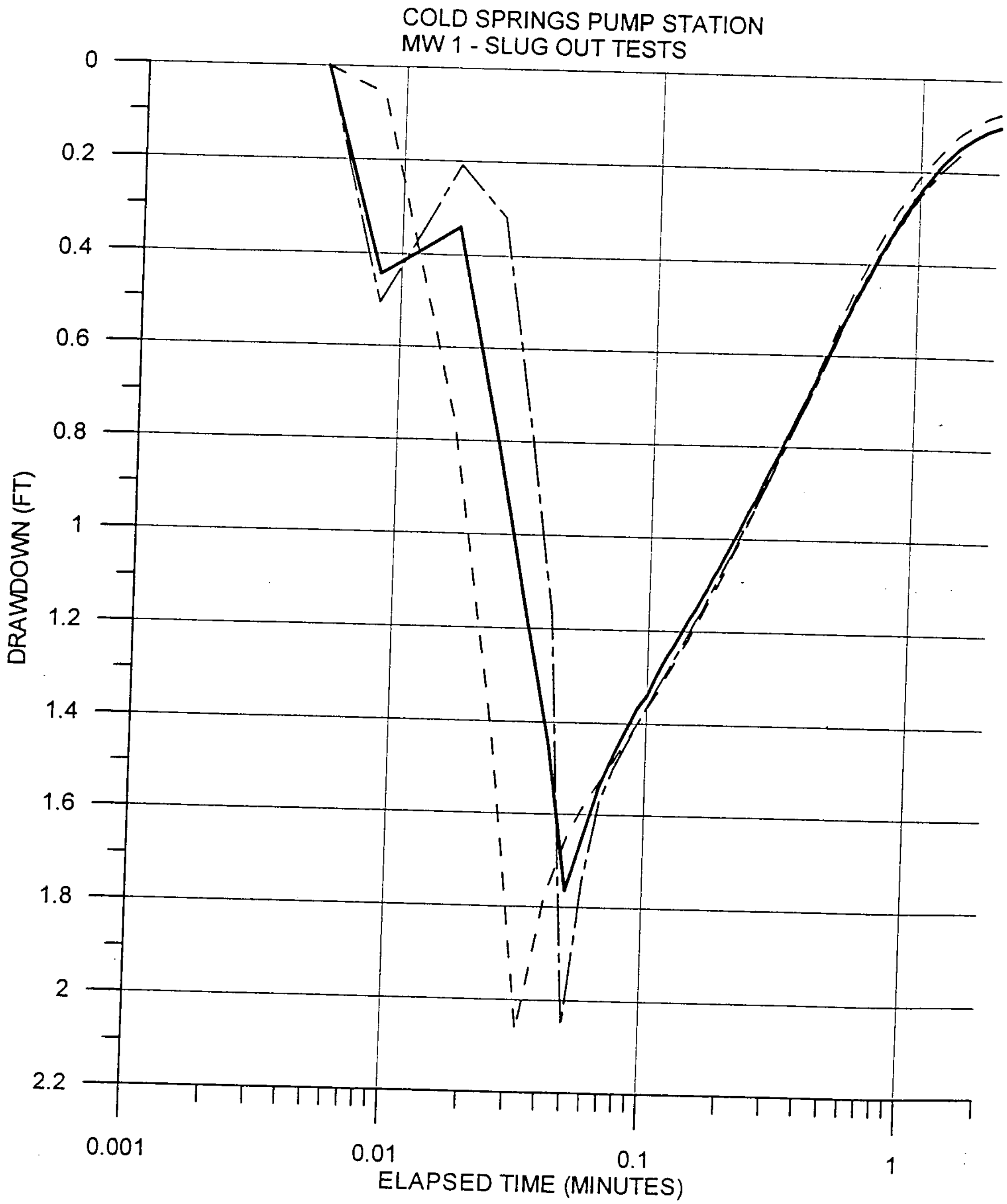
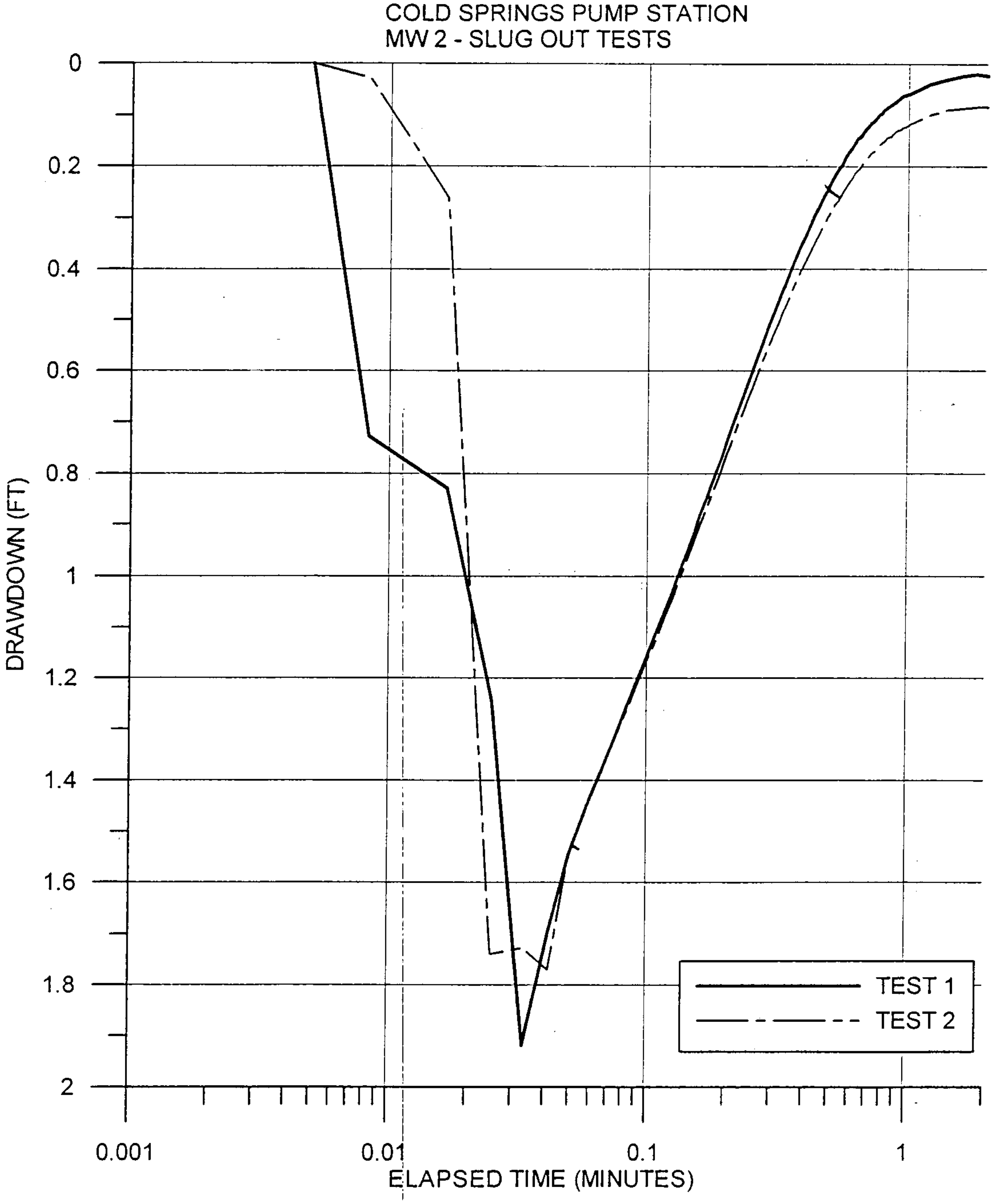

COLD SPRINGS PUMP STATION
MW 1 - SLUG OUT TESTS
DRAWDOWN (FT)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
0.001
0.01
0.1
1
ELAPSED TIME (MINUTES)


COLD SPRINGS PUMP STATION
MW 2 - SLUG OUT TESTS
DRAWDOWN (FT)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
0.001
0.01
0.1
1
ELAPSED TIME (MINUTES)
TEST 1
TEST 2

APPENDIX C

Suggested Specifications
For
Earthwork and Pavement Construction

APPENDIX C

SUGGESTED SPECIFICATIONS FOR EARTHWORK AND PAVEMENT CONSTRUCTION

COLD SPRINGS
WASTEWATER TREATMENT PLANT UPGRADES
WASHOE COUNTY, NEVADA

1.0 GENERAL

1.1 **Scope** - The work done under these specifications shall include clearing, stripping, removal of unsuitable material, excavation, installation of subsurface drainage, preparation of natural soils, placement and compaction of onsite and imported structural fill material, and placement and compaction of pavement materials.

1.2 **Contractor's Responsibility** - A geotechnical investigation was performed for the project by Kleinfelder dated August 22, 2003. The Contractor shall attentively examine the site in such a manner that he can confirm existing surface conditions with those presented in the geotechnical report. He shall satisfy himself that the quality and quantity of exposed materials and subsurface soil or rock deposits have been satisfactory represented by the Geotechnical Engineer's report and Civil Engineer's drawings. Any discrepancy that may be of prior knowledge to the Contractor or that is revealed through his investigations shall be made available to the Owner. It is the Contractor's responsibility to review the attached report prior to construction. The selection of equipment for use on the project and the order of work will similarly be his responsibility such that the requirements included in following sections have been met.

1.3 **Geotechnical Engineer** - The work covered by these specifications shall be observed and tested by the Geotechnical Engineer, Kleinfelder, who shall be hired by the Owner. The Geotechnical Engineer will be present during the site preparation and grading to observe the work and to perform the tests necessary to evaluate material quality and compaction. The Geotechnical Engineer shall submit a report to the Owner, including a tabulation of all tests performed. The costs of retesting of unsuitable work performed by the Contractor shall be deducted from the payments to the Contractor.

1.4 **Standard Specifications** - Where referred to in these specifications, "Standard Specifications" shall mean the current Standard Specifications for Public Works Construction for Washoe County, City of Sparks, City of Reno, Carson City, and City of Yerington (1996, rev. 2001).

1.5 **Compaction Test Method** - Where referred to herein, relative compaction shall mean the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by ASTM D1557 Compaction Test Procedure. Optimum moisture content shall mean the moisture content at maximum dry density as determined above.

2.0 SITE PREPARATION

2.1 **Clearing** - Areas to be graded shall be cleared and grubbed of all vegetation and debris. These materials shall be removed from the site by the Contractor.

2.2 **Stripping** - Surface soils containing roots and organic matter shall be stripped from areas to be graded and stockpiled or discarded as directed by the Owner. In general, the depth of stripping of the topsoil will be approximately four inches. Deeper stripping, where required to remove weak soils or accumulations or organic matter, shall be performed when determined by the Geotechnical Engineer. Strippings shall be removed from the site or stockpiled at a location designated by the Owner.

2.3 **Removal of Existing Fill** - Existing fill soils, trash, and debris in the areas to be graded shall be removed prior to the placing of any compacted fill. Portions of any existing fills that are suitable for use in compacted fill may be stockpiled for future use. All organic material, topsoil, expansive soils, oversize material or other unsuitable material shall be removed from the site by the Contractor or disposed of at a location on site, if so designated by the Owner.

2.4 **Ground Surface** - The ground surface exposed by stripping shall be scarified to a depth of six inches, moisture conditioned to the proper moisture content for compaction, and compacted as required for compacted fill. Recompaction shall be approved by the Geotechnical Engineer prior to placing fill.

3.0 EXCAVATION

3.1 **General** - Excavations shall be performed to the lines and grades indicated on the plans.

The data presented in the geotechnical investigation report is for information and only the Contractor shall make his own interpretation with regard to the methods and equipment necessary to perform the excavation and to obtain material suitable for fill.

3.2 **Materials** - Soils which are removed and are unsuitable for fill should be placed in non-structural areas of the project. When necessary, these soils may be placed in deeper fills if approved by the Geotechnical Engineer.

3.3 **Treatment of Exposed Surface** - The ground surface exposed by excavation shall be scarified to a depth of six inches, moisture conditioned to the proper moisture content for compaction, and compacted as required for compacted fill. Recompaction shall be approved by the Geotechnical Engineer prior to placing fill.

4.0 STRUCTURAL FILL

4.1 **Materials** - Fill material shall consist of suitable onsite or imported fill. All materials used for structural fill shall be reasonably free of organic material, have a liquid limit less than 30, a plasticity index less than 15, 100% passing the six-inch sieve, at least 70% passing the 3/4 inch sieve, and less than 35% passing the No. 200 sieve.

4.2 **Placement** - All fill materials shall be placed in layers of eight inches or less in loose thickness and uniformly moisture conditioned. The lift should then be compacted with a sheepsfoot roller or other approved compaction equipment to achieve at least 90% relative compaction in areas under structures, utilities, roadways, parking areas, and to at least 85% in undeveloped areas. No fill material shall be placed, spread, or rolled while it is frozen or thawing, or during unfavorable weather conditions.

4.3 **Benching** - Fill placed on slopes steeper than 5 horizontal to 1 vertical shall be keyed into firm, native soils or rock by a series of benches. Benching can be conducted simultaneously with placement of fill. However, the method and extent of benching shall be checked by the Geotechnical Engineer.

4.4 **Compaction Equipment** - The Contractor shall provide and use sufficient equipment of a type and weight suitable for the conditions encountered in the field. The equipment shall be capable of obtaining the required compaction in all areas, including those that are inaccessible to ordinary rolling equipment.

4.5 **Recompaction** - When, in the judgment of the Geotechnical Engineer, sufficient compaction effort has not been used, or where the field density tests indicate that the required compaction or moisture content has not been obtained, or if "pumping" or other indications of instability are noted, the fill shall be reworked and recompacted as needed to obtain a stable fill at the required density and moisture content prior to placing additional fill materials.

4.6 **Responsibility** - The Contractor shall be responsible for the maintenance and protection of all embankments and fills made during the contract period and shall bear the expense of replacing any portion, which has become displaced due to carelessness, negligent work, or failure to take proper precautions.

5.0 UTILITY TRENCH BEDDING AND BACKFILL

5.1 **Material** - Pipe bedding shall be defined as all material within six inches of the perimeter of the pipe. Backfill shall be classified as all material within the remainder of the trench. Material for use as bedding shall consist of clean, granular materials, having a sand equivalent of not less than 30, and shall conform to requirements for Class A backfill listed in Section 200.03 of the Standard Specifications.

5.2 **Placement and Compaction** - Pipe bedding shall be placed in thin layers not exceeding eight inches in loose thickness, conditioned to the proper moisture content for compaction, and compacted to at least 90% relative compaction. All other trench backfill shall be placed in thin layers not exceeding eight inches in loose thickness, conditioned to the proper moisture content, and compacted as required for adjacent fill. If not specified, backfill should be compacted to at least 90% relative compaction in areas under structures, utilities, roadways, parking areas, concrete flatwork, and to 85% relative compaction in undeveloped areas.

6.0 SUBSURFACE DRAINAGE

6.1 **General** - Subsurface drainage should be constructed as shown on the plans. Drainage pipe should meet the requirements set forth in the Standard Specifications.

6.2 **Materials** - Permeable "drain rock" material used for subdrainage shall be graded gravel which meets requirements for Class C or Class D backfill listed in Section 200.03 of the Standard Specifications or other material approved by the Geotechnical Engineer.

6.3 **Geotextile Fabric** - Non-woven filter fabric should be placed between the permeable drain rock and native soils. Filter cloth with an equivalent opening size greater than the No. 100 sieve size, and a grab strength not less than 100 pounds should be used. The Geotechnical Engineer should be consulted on a specific basis when a particular fabric is chosen so that compliance to the above recommendations can be verified.

6.4 **Placement and Compaction** - Drain rock shall be placed in thin layers not exceeding eight inches in loose thickness and compacted as required for adjacent fill, but in no case will be less than 85% relative compaction. Placement of geotextile fabric will be in accordance with the manufacturer's specifications and should be checked by the Geotechnical Engineer.

7.0 AGGREGATE BASE FOR CONCRETE SLABS

7.1 **Material** - Aggregate base for concrete slabs shall consist of Type 2, Class B aggregate base conforming to requirements in Section 200.01.03 of the Standard Specifications.

7.2 **Placement** - Aggregate base shall be compacted and kept moist until placement of concrete. Compaction shall be by suitable vibrating compactors. Aggregate base shall be placed in layers not exceeding eight inches in thickness. Each layer shall be compacted by at least four passes of the vibratory compaction equipment or until 95% relative compaction has been obtained.

8.0 SUBGRADE AND AGGREGATE BASE FOR PAVED AREAS

8.1 **Subgrade Preparation** - After completion of the utility trench backfill and prior to placement of aggregate base, the upper six inches of subgrade soil shall be uniformly compacted to at least 90% relative compaction. This may require scarifying, moisture conditioning, and compacting in both cut and fill areas.

8.2 **Aggregate Base** - Aggregate materials shall meet the requirements of the appropriate sections of the "Standard Specifications" for Type 1, Class A or Type 2, Class B aggregate base. The aggregate base materials must be approved by the Geotechnical Engineer prior to use.

After the subgrade is properly prepared, the aggregate base shall be placed in layers, moisture conditioned as necessary, and compacted by rolling to at least 95% relative compaction. The compaction thickness of aggregate base shall be as shown on the approved plans.

9.0 ASPHALT CONCRETE PAVEMENT

9.1 **Thickness** - The compacted thickness of asphalt concrete shall be shown on the approved plans.

9.2 **Materials** - Aggregate materials for asphalt concrete in parking lots and light traffic areas shall conform to the requirements listed for Type 3 bituminous aggregates in Section 200.02 of the "Standard Specifications." Aggregate materials for asphalt concrete in dedicated roadways and heavy traffic areas shall conform to the requirements listed for Type 2 bituminous aggregates. Asphalt concrete mixes shall utilize AR-4000, AC-20, or AC-20P grade of asphalt cement. The AC-20P grade is required within the City of Reno for at least the top two inches of pavement. The Contractor shall submit a proposed asphalt concrete mix design to the Owner for review and approval prior to paving. The mix design shall be based on the Marshall Method.

Where prime coat is specified, the type and grade of asphalt for use as prime coat shall be SS-1 or SS-1h with an application rate of 0.15 to 0.25 gallons per square yard. The type and grade of asphalt for use as tack coat shall be SS-1, SS-1h, CSS-1, or CSS-1h with an application rate of 0.08 to 0.13 gallons per yard.

The type and grade of asphalt for use as fog seal coat shall be SS-1 or SS-1h, with an application rate of 0.07 to 0.12 gallons per square yard (diluted emulsion). Sand blotter, if needed to prevent "pick-up," shall be spread at a rate of 10 to 15 pounds per square yard.

9.3 **Placement and Compaction** - The asphalt concrete material and placement procedures shall conform to appropriate sections of the "Standard Specifications." The asphalt concrete material shall be compacted to between 92% and 97% of the Theoretical Maximum Rice Specific Gravity, and to a minimum of 96% of the maximum Marshall Density.

APPENDIX D

Application for Authorization to Use

APPENDIX D
APPLICATION FOR AUTHORIZATION TO USE

COLD SPRINGS
WASTE WATER TREATMENT PLANT UPGRADES
WASHOE COUNTY, NEVADA

Kleinfelder, Inc.
4875 Longley Lane, Suite 100
Reno, Nevada 89502

To whom it may concern:

Applicant understands and agrees that the "Geotechnical Investigation Report, Cold Springs Waste Water Treatment Plant Upgrades," dated August 22, 2003, Job No. 33247.01, for the subject site is a copyrighted document, that Kleinfelder, Inc. is the copyright owner and that unauthorized use or copying of said document for the subject site is strictly prohibited without the express written permission of Kleinfelder, Inc. Applicant understands that Kleinfelder, Inc. may withhold such permission at its sole discretion, or grant permission upon such terms and conditions as it deems acceptable.

Applicant agrees to accept the contractual terms and conditions between Kleinfelder, Inc. and Kennedy Jenks Consultants originally negotiated for preparation of this document. Use of this document without permission releases Kleinfelder, Inc. from any liability that may arise from use of this report.

To be Completed by Applicant

____
(company name)

____
(address)

____
(city, state, zip)

_____________ _____________
(telephone) *(FAX)*

By: _______________________________

Title: _______________________________

Date: _______________________________

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By: _______________________________
(Kleinfelder, Inc. project manager)

Date: _______________________

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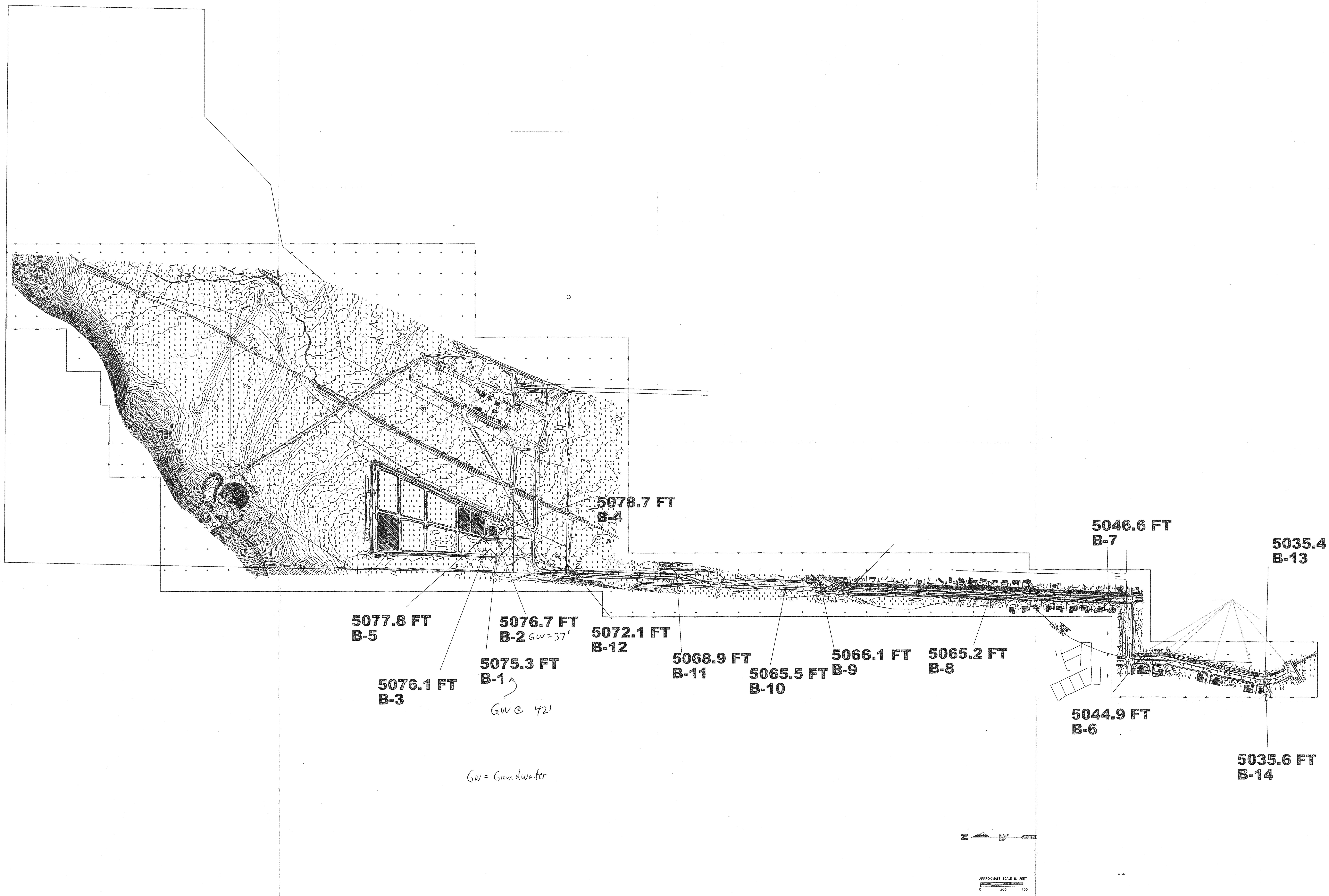

KLEINFELDER
4875 LONGLEY LANE, SUITE 100
RENO, NEVADA 89502
Tel. (775) 689-7800

PROJECT NO. 33247.01

SITE PLAN

COLD SPRINGS WASTEWATER TREATMENT PLANT

COLD SPRINGS, NEVADA

PLATE
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