ADDENDUM #1
NORTH TRUCKEE DRAIN REALIGNMENT-PHASE 3
BID #16/17-006 / PWP #WA-2017-022
BIDS DUE NO LATER THAN: 1:45 PM ON NOVEMBER 16, 2016
PUBLIC BID OPENING: 2:00 PM ON NOVEMBER 16, 2016

This addendum is to notify all potential proposers of clarifications made to the Bid documents as stated below.

1. **REVISED BID ITEM SCHEDULE:**
   A new Bid Item Schedule is provided as part of this addendum and should replace Pages 4-7 of the original bid document. Changes have been made to either quantities and/or descriptions in Items 41, 44 and 45.

2. **BID ITEM CLARIFICATIONS:**
   a) Change Bid Item 44 to Description “Install 36” Type IV RCP Storm Drain w/associated collars”
   b) Change Bid Item 45 to Description “Install 18” Type IV RCP Storm Drain w/associated collars”

3. **GEOTECHNICAL INVESTIGATION**
   Included as a reference and attached within this addendum is a “Geotechnical Investigation” prepared by Construction Materials Engineers, dated November 10, 2016

4. **PLAN SET CHANGES:**
   <u>North Truckee Drain Pipe Extension Improvements; Stantec</u>

   Replace PP-2 through PP-4 with attached Plan Sheets PP-2, PP-3, and PP-4

   Drawing RP-3, paragraph 8. MULCH AND TACKIFIER. Change second sentence as follows: Apply slurry consisting of plant based tackifier at 200 pounds/acre and recycled paper mulch at <u>*2000*</u> pounds/acre.

   <u>North Truckee Drain Realignment Phase 3; HDR</u>

   <u>Additional information related to water control gates</u>
   All gates shall have:
   - Have non-rising stems
   - be capable of 20 ft of non-seating water pressure
   - be non-self-contained frame style
   - manually operated
   - Waterman model S-45 or approved similar

18” pipe on sheet DT-6: 18”x18” with gate width of >28”
24” pipe on sheet DT-7: 24”x24” with gate width of >34”
48” pipe on sheet S-4: 48”x48” with gate width of >58”

Please note and adjust your bid according to the revisions, additions, deletions, clarifications or modifications as presented on this Addendum #1, which are made a part of this bid. NOTE: To avoid disqualification, this Addendum 1 (and any other addenda) must be signed by an authorized representative of the bidding firm in the space provided and must be submitted with your firm’s sealed proposal. Failure to return this addendum, duly signed, may be cause for rejection of the bid. ALL ADDENDA SHOULD BE SIGNED AND PLACED IN SEQUENTIAL ORDER AND ATTACHED TO THE FRONT OF THE BID PACKAGE, COMPLETE WITH ALL REQUIRED DOCUMENTS.

____________________________	____________________________
CONTRACTOR BUSINESS NAME	Dan Marran, C.P.M., CPPO Contracts and Risk Manager
X__________________________	
Authorized Signature	November 14, 2016
____________________________	
Printed Name of Person Signing	

CITY OF SPARKS
BID ITEM SCHEDULE – *REVISED BY ADDENDUM 1*

BID # 16/17-006
PWP # WA-2017-022
BID TITLE: North Truckee Drain Realignment – Phase 3

PRICES must be valid for 90 calendar days after the bid opening.

COMPLETION of this project is expected **PURSUANT TO CONTRACT DOCUMENTS**.

BIDDER acknowledges receipt of _______ Addenda.

_________________________________ _________________________________
Bidder Name (signature)

Item No.	Quantity	Unit	Description	Unit Price	Total Price
BASE BID ITEMS					
1	1	LS	Mobilization / Demobilization / Insurance / Bonds / Surveying & Staking	$ _______ /LS	$ _____________
2	1	LS	Clear and Grubbing	$ _______ /LS	$ _____________
3	1	LS	Traffic Control	$ _______ /LS	$ _____________
4	1	LS	Dewatering	$ _______ /LS	$ _____________
5	2,558	SY	Remove Plantmix Bituminous Surface	$ _______ /SY	$ _____________
6	753	LF	Remove Electric Fencing	$ _______ /LF	$ _____________
7	275	LF	Remove PCC Curb and Gutter – 36” Type 1, 24” Type 1	$ _______ /LF	$ _____________
8	2,212	LF	Remove and Replace Chain Link Fence	$ _______ /LF	$ _____________
9	11	EA	Remove Small Concrete Structures	$ _______ /EA	$ _____________
10	874	LF	Remove Storm Drain Culvert (all sizes and materials)	$ _______ /LF	$ _____________

Item No.	Quantity	Unit	Description	Unit Price	Total Price
11	1	LS	Remove Washoe County School District Box Beam Bridge	$ _______ /LS	$ ______________
12	444	LF	Remove and Relocate/Replace Private Underground Electric	$ _______ /LF	$ ______________
13	2724	SF	Remove Riprap	$ _______ /SF	$ ______________
14	1	LS	Remove and Reset NOAA NGS Benchmark Monument	$ _______ /LS	$ ______________
15	189	LF	Construct Reinforced Concrete Box Culvert and Transition Structure to NDOT RCBs (2-8’x8’) – “E” Sta. 325+18.61 to “E” Sta. 327+28.19	$ _______ /LF	$ ______________
16	137	LF	Construct Reinforced Concrete Box Culvert (6’x6’) – “W” Sta. 218+70.36 to “W” Sta. 219+35.21 & “W” Sta. 219+42.43	$ _______ /LF	$ ______________
17	16	LF	Construct Reinforced Concrete Box Culvert (1-10’x4’)	$ _______ /LF	$ ______________
18	739	LF	Construct Reinforced Concrete Box Culvert (2-8’x8’)	$ _______ /LF	$ ______________
19	2,540	LF	Construct Reinforced Concrete Box Culvert (2-14’x10’)	$ _______ /LF	$ ______________
20	2	EA	Construct Reinforced Concrete Confluence Structure	$ _______ /EA	$ ______________
21	5	EA	Construct Reinforced Concrete Access Vault	$ _______ /EA	$ ______________
22	2	EA	Construct Backflow Prevention Vault	$ _______ /EA	$ ______________
23	19	EA	Construct Small Concrete Structure (Manhole, Drop Inlet, Trash Rack)	$ _______ /EA	$ ______________
24	1	EA	Construct Sump Structure (“NTD” Sta. 41+50)	$ _______ /EA	$ ______________
25	286	LF	Construct Small Diameter (<31”) Gravity Main for Storm Drain and Laterals	$ _______ /LF	$ ______________

Item No.	Quantity	Unit	Description	Unit Price	Total Price
26	1,049	LF	Construct Large Diameter (>31”) Gravity Main for Storm Drain	$ _______ /LF	$ _____________
27	31	LF	Construct 3’ PCC U-Flume Channel	$ _______ /LF	$ _____________
28	275	LF	Construct PCC Curb and Gutter	$ _______ /LF	$ _____________
29	792	SF	Construct Riprap Aprons	$ _______ /SF	$ _____________
30	1	LS	Construct Rockery Retaining Wall (“NTD” Sta. 34+25)	$ _______ /LS	$ _____________
31	2,676	SY	Construct Plantmix Bituminous Pavement (3” AC on 6” Aggregate Base)	$ _______ /SY	$ _____________
32	521	CY	Construct Gravel Maintenance Road (15’ wide x 6” thick)	$ _______ /CY	$ _____________
33	1	LS	Hydroseeding, Landscape and Irrigation Repair/Restoration	$ _______ /LS	$ _____________
34	32	LF	Remove existing chain link fencing and gate	$ _______ /LF	$ _____________
35	1,400	CY	Pipe Stabilization	$ _______ /CY	$ _____________
36	33,000	CY	Place Backfill	$ _______ /CY	$ _____________
37	4,560	SY	Place 12-Inch Type II Class B aggregate base	$ _______ /SY	$ _____________
38	960	SF	Construct PCC commercial driveway with aggregate base	$ _______ /SF	$ _____________
39	4	EA	Construct 16-ft NDOT swing gate	$ _______ /EA	$ _____________
40	1	LS	Perform dewatering operations including bypass to overflow structure and removal/emergency removal plans	$ _______ /LS	$ _____________
41	**8**	EA	Install 48” Type IV Manhole w/36” Cir. SD Grate	$ _______ /EA	$ _____________

Item No.	Quantity	Unit	Description	Unit Price	Total Price
42	1	LS	Install PVC SDR35 Pipe Riser w/24” Cir. SD Grate	$ _______ /LS	$ _____________
43	1,760	LF	Install 60” Type IV RCP Storm Drain	$ _______ /LF	$ _____________
44	**25**	LF	**Install 36” Type IV RCP Storm Drain w/associated collars**	$ _______ /LF	$ _____________
45	**18**	LF	**Install 18” Type IV RCP Storm Drain w/associated collars**	$ _______ /LF	$ _____________
46	1,200	CY	Place slurry backfill	$ _______ /CY	$ _____________
47	1	LS	Construct 60” NDOT Type I Headwall with Wingwalls	$ _______ /LS	$ _____________
48	1	LS	Tideflex 60” Checkmate Valve w/downstream clamp & freight	$ _______ /LS	$ _____________
49	1,925	SF	Place Rip-Rap Grade 150	$ _______ /SF	$ _____________
50	5,550	SF	Place Rip-Rap Grade 400	$ _______ /SF	$ _____________
51	FA	FA	Force Account – General (CONTINGENT ITEM)	$ 500,000	$ 500,000
52	FA	FA	Force Account – Hazardous Materials and Unsuitable Soils (CONTINGENT ITEM)	$ 50,000	$ 50,000
Total Base Bid Price ____dollars **(written total base bid price including items 1 through 54)**					$ _____________

The quantity of the above contingent item(s) of work, as set forth on the Bid Item Schedule represent no actual estimate, are nominal only and may be greatly increased or decreased or reduced to zero. The increase or reduction of these quantities as compared with that set forth on the Bid Item Schedule shall not constitute a basis for claim by the Contractor for extra payment or damages.

The above items of work are represented on two independent plan sets: North Truckee Drain Realignment Phase 3 (HDR, October 12, 2016) and North Truckee Drain Pipe Extension Improvements (Stantec, October 14, 2016). The table below illustrates which items are associated with which plan set.

Item No.	North Truckee Drain Realignment Phase 3 (HDR)	North Truckee Drain Pipe Extension Improvements (Stantec)
1-3	X	X
4-33	X	
34-51		X
52-53	X	X

CONSTRUCTION MATERIALS ENGINEERS, INC.
6980 Sierra Center Parkway, Suite 90
Reno, NV 89511

November 10, 2016
Project No: 1870

Mr. Luke Hoffman P.E.
STANTEC
6980 Sierra Center Parkway, Suite 100
Reno, NV 89511

RE: Geotechnical Investigation
North Truckee Drain Pipe Extension, Sparks, Nevada

Dear Mr. Hoffman:

Construction Materials Engineers, Inc. (CME) is pleased to submit the following geotechnical investigation for the proposed North Truckee Drain Pipe Extension. The project boundaries begin immediately south of Interstate 80 and extends along the existing North Truckee Drain to the Truckee River in East Sparks.

The objectives of this study were to:

- Determine general soil and ground water conditions pertaining to design and construction of the proposed improvements.
- Provide recommendations for the design and construction of the project, as related to these geotechnical conditions.

The proposed pipeline alignment is contained in Section 11, Township 19N, Range 20E. The area covered by this report is shown on Plate A-1 (Site Plan) in Appendix A.

1.0 PROJECT DESCRIPTION

1.1 Background

The North Truckee Drain (NTD) begins in Northern Sparks and primarily flows in a southerly direction adjacent to Sparks Boulevard before changing course in an easterly direction immediately north of Interstate 80 (I-80). The NTD then flows beneath I-80, near the I-80 off ramp to Sparks Boulevard, and intersects with the People's Ditch. From this intersection point, the NTD flows in an easterly direction along the south side of I-80 for a distance of about 700 feet before heading in a southerly direction to intersect with the Truckee River.

The current project is part of an overall realignment and replacement of the existing NTD to carry the design flood flow of the 117-year storm. A box culvert, beginning in the People's Ditch and heading eastward along the south side of I-80, will be constructed to carry storm water to the Truckee River, near the Vista Narrows. This project will connect into this box culvert.

1.2 Pipeline Location

The proposed pipeline alignment begins approximately 300 feet north of Kleppe Lane in the existing NTD ditch. The pipeline extends south, crossing beneath Kleppe Lane and Greg Street, before terminating at the Truckee River.

1.3 Pipeline Description

The proposed project is in the preliminary design stages, but existing project information includes the following:

- Construction of a 60-inch diameter RCP within the existing North Truckee Drain extending from the box culvert design outlet, located near the south side of Interstate 80, to the Truckee River;
- Total length of the pipe alignment is about 1,920 feet;
- The pipeline will be covered to match existing grade;
- The pipeline will be routed beneath the existing bridge structures located at Kleppe Lane and Greg Street;
- Headwall and wing walls will be constructed at the Truckee River discharge location.

2.0 SITE CONDITIONS

The proposed pipeline alignment is located in a commercial and industrial area in East Sparks. Existing businesses are located on either side of the NTD within the project boundaries.

Greg Street is a four-lane arterial roadway with an east to west traveling direction overlying the pipeline alignment. Kleppe Lane is a two-lane roadway also with an east to west traveling direction overlying the pipeline alignment. Both roadway crossings have arch-type bridges (Con-Spans) with wing walls (refer to Photo #1).

Photo #1: Looking north at south side of the Kleppe Lane Bridge

The existing NTD is an open-cut ditch with side-slopes having gradients that vary from approximately 1H to 1V to 3H to 1V (horizontal to vertical). The depth of the ditch from adjacent top of bank is approximately ranges from 15 to 20 feet (refer to Photo #2 and #3).

Photo #2: Existing NTD looking south from the Kleppe Lane Bridge

Photo #3: Existing NTD looking north from the Kleppe Lane Bridge

3.0 LITERATURE RESEARCH

Several existing geotechnical investigations have been completed near the project site. Consequently, field exploration was not performed for this project. These geotechnical investigations include the following:

- Kleppe Lane Bridge Replacement Project, dated February 1995, SEA Engineers Inc. Field exploration for this project included drilling two borings on either side of bridge structure to depths of 22 feet.
- East Sparks Industrial Park, dated March 1979, SEA Engineers Inc. Field Exploration for this project included four borings drilled to depths of 30 feet along the east side of the existing NTD.
- Kleinfelder completed a Geotechnical Investigation for the North Truckee Drain Realignment Project. They completed one boring and a monitoring well to a depth of 30 feet near the north end of the pipeline alignment.

A review of existing published geologic maps, fault hazard reports, and soils maps to identify the presence of documented geologic hazards at the site was also completed.

Exploration locations from these existing geotechnical investigations are presented in Appendix A, while boring logs are presented in Appendix B.

4.0 GEOLOGIC AND GENERAL SOIL CONDITIONS

Sedimentation in the Truckee Meadows has been in progress at varying rates since the formation of the block faulted basin. Most of the sediments, including the coarse grained sand, gravel, and cobble deposits that underlie the majority of the Truckee Meadows, were deposited quite abruptly in the post-glacial period during torrential flooding. With the advent of a warm, drier climate, the volume and size distribution of sediment transported was greatly reduced and the sedimentation process became largely limited to the reworking of earlier deposits.

4.1 General Soil Conditions

A review of the published Geologic Map for the Vista Quadrangle (Bell, 1985) indicates that the project site is located in the Floodplain Deposits of the Truckee River (Qfl). These deposits primary consist of fine-grained sands and silts. Based on a review of the existing geologic information from the referenced geotechnical investigations near the project site, the soil profile consists of three prominent soil horizons to depths of about 30 feet below the existing ground surface:

1) The uppermost soil horizon is comprised of a fill soil classified as either a silty sandy gravel **(GM)** or well-graded sand with clay, gravel, and cobbles **(SW-SC).** The thickness of this soil horizon is variable and ranged from 1.5 to 14 feet. The thickest section of fill soils was encountered in the north end of the project site located near the juncture location with the proposed box culvert.

2) Underlying the uppermost fill soil horizon, a complex interbedding of three predominant soil types were encountered. These soil types consisted of fine-grained silty sand **(SM),** fat clay **(CH),** or lean clay **(CL).** The thickness of this soil horizon ranged from 10 to 22 feet. In general, the more granular deposits were encountered toward the upper portion of the soil profile. Based on SPT blow counts, this soil horizon has a very soft to very stiff relative density for cohesive soils and loose relative density for granular soils.

3) The lowermost soil horizon encountered was classified as either a poorly graded sand **(SP)** or poorly graded sand with gravel **(SP)**. Based on SPT blow counts, this soil horizon has a medium dense to very dense relative density. This soil horizon was encountered to the depth explored. The elevation of the boundary between the lowermost soil horizon and upper fine-grained soil horizon ranges from 4366 to 4375 feet. In general, the lowermost soil horizon becomes shallower toward the north end of the pipeline.

The lowermost soil horizon encountered representatives the uppermost deposits of the Tahoe Outwash Formation consisting of Pleistocene age glacial outwash deposit. This formation forms an extensive alluvial wedge across the Truckee Meadows, thickening eastward into Sparks with an estimated thickness of several hundred feet (Bingler,1975). This formation is characterized as a heterogeneous mixture of sands, gravels, cobbles and boulders. Boulder-sized particles up to 16 feet in diameter have been encountered in this deposit (Bingler, 1975).

4.2 SOIL MOISTURE AND GROUNDWATER CONDITIONS

Groundwater was encountered at an elevation ranging from 4375 to 4380 feet. Soils were generally encountered in a moist to very moist condition above the groundwater table.. Groundwater depth may fluctuate due to changes in precipitation, seasonal variations, or other conditions not noted at the time of our investigation.

5.0 SEISMIC HAZARDS

5.1 SEISMICITY

Much of the Western United States is a region of moderate to intense seismicity related to movement of the crustal masses (plate tectonics). By far, the most active regions outside of Alaska are along the San Andreas Fault zone of western California. Other seismically active areas include the Wasatch Front in Salt Lake City, Utah, which forms the eastern boundary of the Basin and Range physiographic province, and the eastern front of the Sierra Nevada Mountains, which is the western margin of the province. The project site lies near the eastern base of the Sierra Nevada, within the western extreme of the Basin and Range.

It is generally accepted that the maximum credible earthquake in this area would be in the range of magnitude 7 to 7.5 originating from the frontal fault system of the Eastern Sierra Nevada. The most active segment of this fault system that is closest to the Reno-Stead area is located at the base of the eastern flank of the Carson Range near Thomas Creek, Whites Creek and Mt. Rose Highway, some 11 miles southwest of the project site.

5.2 FAULTS

Based on a review of the referenced geologic map; updated geologic map of the Vista Quadrangle, Ramelli, 2011; and USGS Quaternary Faults on Google Earth Map, no mapped faults are shown trending through the proposed pipeline alignment.

The closest mapped fault is located less ¼ miles west of the pipeline. This fault is a concealed fault that is trending in a north to south direction and is dashed, meaning the fault is approximately location. This fault is part of a fault zone located at the base of the western flank of the Pah Rah Range and Virginia Range. Another fault is mapped about ½ miles east of the pipeline alignment, which is part of the same fault zone.

Quaternary earthquake fault evaluation criterion has been formulated by a professional committee for the State of Nevada Seismic Safety Council. These guidelines are consistent with the State of California Alquist-Priolo Act of 1972, which defines Holocene Active Faults as those with evidence of displacement within the past 10,000 years (Holocene time). Those faults with evidence of displacement during Pleistocene time (10,000 to 1,600,000 years before present) are classified as either late Quaternary Active Fault (10,000 to 130,000 years) or Quaternary Active Fault (> 130,000 years). Both of the latter fault designations are considered to have a decreased potential for activity compared to the Holocene Active Fault. An inactive fault is considered is a fault that does not comply with these age groups.

The faults closest to the pipeline have not been classified for age of the last rupture movement along the fault. However, since these faults are concealed, they are likely not Holocene Active Faults and likely classify as either Late Quaternary Active faults or Quaternary Active faults.

5.3 LIQUEFACTION

Liquefaction is a nearly a complete loss of soil shear strength that can occur during an earthquake, as cyclic shear stresses generate excessive pore water pressure between the soil grains. The higher the ground acceleration caused by a seismic event or the longer the duration of shaking, the more likely liquefaction will occur.

The soil types most susceptible to liquefaction are loose to medium dense cohesionless sands, soft to stiff non-plastic to low plastic silts, or any combination of silt-sand mixtures lying below the groundwater table. Liquefaction is generally limited to depths of 50 feet or less below the existing ground surface.

In general, soils encountered below the groundwater table are generally either fine-grained clay soils or medium dense to dense glacial outwash deposits. Because of the material types below the water table, it is our opinion that soil liquefaction potential within the majority of the pipeline alignment is low. However, as indicated in the referenced Geotechnical Investigation for the Kleppe Bridge, localized areas of soil liquefaction may occur.

6.0 SEISMIC DESIGN PARAMETERS

Seismic design parameters are based on site-specific estimates of spectral response ground acceleration as designated in the International Building Code (IBC, 2012). The benefit of this approach is that a response spectrum can be developed from this data and, based on the period of the structure, a spectral acceleration for that structure can be determined. These values are based on two criteria: site classification and site location (latitude and longitude). Site classification is based on the substrata soil profile type, as presented in Table 1 (Site Classification Definitions).

Table 1 – Site Classification Definitions

Site Classification	Soil Profile Type Description
A	Hard Rock
B	Rock
C	Very Dense Soil and Soft Rock
D	Stiff Soil Profile
E	Soft Soil Profile
F	Soil Type Requiring Site-Specific Evaluation

The soil/bedrock profile classification is based on two criteria: density (primarily for soils based on SPT blow count data or shear wave velocity) or hardness (based on shear wave velocity primarily for bedrock sites). These two criteria have to be determined to a depth of 100 feet bgs. Based on the soil profile encountered and known geologic conditions, a Site Classification of D (stiff soil profile) is recommended.

Spectral response acceleration values (S_s & S_1) are based on structures underlain by bedrock with a site classification of B. Acceleration values may amplify or attenuate depending on the subsurface geologic conditions. Therefore, the IBC provides correction factors (F_a & F_v) to modify the acceleration values if the site is located overlying subsurface geologic conditions with a site classification other than B.

Spectral response acceleration values were determined from the USGS website: *U.S. Seismic Design Maps* Table 2 provides a summary of seismic design parameters, based on 2010 ASCE 7, as referenced by IBC, including correction factors F_a & F_v. A printout of the design information including spectral response acceleration values is provided in Appendix C.

Table 2 – Seismic Design Parameters	
PARAMETER DESCRIPTION	**PIPELINE LOCATION**
Approximate Latitude of Site	39.5247
Approximate Longitude of Site	119.7059
Peak Ground Acceleration-MCE_R PGA (ASCE 7-10 Standard)	0.546 g
Design Peak Ground Acceleration-DPGA (ASCE 7-10 Standard)	0.424 g
Spectral Response Acceleration at Short period (0.2 sec.) S_s (for Site Class B)	1.593 g
Spectral Response Acceleration at 1-second Period, S_1 (for Site Class B)	0.547 g
Site Class Selected for this Site	D
Site Coefficient F_a, decimal	1.0
Site Coefficient Fv, decimal	1.500
Design Spectral Response Acceleration at Short period, S_{Ds} (Adjusted to Site Class B, SDs= 2/3 SMs)	1.062 g
Design Spectral Response Acceleration at 1-second Period, S_{D1} (Adjusted to Site Class B, SD1=2/3 SM1)	0.547 g
1) MCE_R PGA- Maximum credible earthquake geometric mean peak ground acceleration.	

7.0 DISCUSSION AND RECOMMENDATIONS

The proposed pipeline invert elevation is located below the groundwater table in soft floodplain deposits primarily consisting of either fat clays **(CH)** or lean clays **(CL).** These floodplain deposits have high in-place moisture contents. Because of the material types and moisture content of the native soils, the primary construction concern is stability of the pipeline support soils.

Clay soils exhibiting high plasticity characteristics and can shrink or swell in response to moisture changes. Moisture changes within these soils can occur as a result of seasonal variations in precipitation, poor site drainage, capillary action, or from other sources. Based on studies and experience, clay soil volume changes can cause differential movements within structural elements constructed within their sphere of influence. However, if groundwater table levels are maintained, and clay soils located below structural elements remain in near saturated conditions, shrink/swell potential will be low.

For purposes of this project, the following definitions shall apply:

- Fine-grained soil is defined as soil with more than 40 percent by weight passing the number 200 sieve and a plasticity index lower than 15.
- Clay soil is defined as a soil with more than 20 percent of the soil particles by weight passing the number 200 sieve and a plasticity index equal or greater than 15.

- Granular soil is defined as soil not meeting the above criteria with a particle sizing of less than 4-inches.

The recommendations provided herein, and particularly under **Site Preparation, Grading and Filling,** and **Construction Observation and Testing** are intended to reduce risks of structural distress related to consolidation or expansion of native soils and/or structural fills. These recommendations, along with proper design and construction of the planned structure and associated improvements, work together as a system to improve overall performance. If any aspect of this system is ignored or poorly implemented, the performance of the project will suffer. Sufficient construction observation and testing should be performed to document that the recommendations presented in this report are followed.

Structural areas referred to in this report include all areas of concrete slabs, asphalt pavements, as well as pads for any minor structures. All compaction requirements presented in this report are relative to ASTM D 1557*. Unless otherwise stated in this report, all related construction should be in accordance with the Standard Specifications for Public Works Construction, dated 2012.

Any evaluation of the site for the presence of surface or subsurface hazardous substances is beyond the scope of this study. When suspected hazardous substances are encountered during routine geotechnical investigations, they are noted in the exploration logs and reported to the client. No such substances were identified during our exploration.

7.1 Headwall and Wingwall Foundations

It is recommended that shallow, spread footings be used for foundation support and is the basis for our design recommendations. Provided that foundation grade soils preparation has been performed in accordance with the recommendations of Section 8.0, the bearing pressures presented in Table 3 can be utilized for the design of continuous wall footings.

Table 3 – Building Foundation Allowable Bearing Pressures

Loading Conditions	Maximum Soil Net Allowable Bearing Pressures[1] (pounds per square foot)
Dead loads plus full time live loads	2,500
Dead loads plus live loads, plus transient wind, or seismic loads.	3,325

NOTES:
1. The net allowable bearing pressure is that pressure at the base of the footing in excess of the adjacent overburden pressure.

Footings shall be set at least two feet below the adjacent exterior finish grades for frost protection. Continuous spread foundations should be at least 24 inches in width.

* Relative compaction refers to the ratio (percentage of the in-place density of a soil divided by the same soil's maximum dry density as determined by the ASTM D 1557 laboratory test procedure. Optimum moisture content is the corresponding moisture content of the same soil at its maximum dry density.

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. A friction factor of 0.40 may be utilized for sliding resistance at the base of the spread footing. A design value of 160 pounds per square foot per foot of depth is recommended for passive soil pressures. The passive pressure value is based on buoyant soil pressures. It should be understood that some lateral deformation on the order of 2 to 4 percent of the depth of embedment (Tomlinson, 1986) for a properly compacted backfill is required to mobilize the ultimate passive resistance. Therefore, due to the strain incompatibility in the simultaneous development of all components of lateral soil resistance as well as reducing the amount of displacement required to develop the design passive pressure, a factor of safety of 1.5 was applied to the passive pressure and sliding resistance from their ultimate values.

In designing for passive pressure, the upper one-foot of the soil profile should not be included unless confined by a concrete slab, or pavement. The passive pressure value is based on maintaining a near level surface gradient along the exterior of the retaining wall with a length that is at least 3.5 times the depth of the foundation below exterior finished grade. Design values are based on spread footings bearing on structural fill and backfilled with structural fill.

Seismic passive pressure was determined using charts developed by log spiral procedures (Shamsabadi et al, 2007). Under seismic loading, a reduction in passive pressure will occur and a design value of 110 pounds per square foot per foot of depth is recommended.

7.1.1 Settlements

Due to the presence of granular native soils and material characteristics of proposed fill soils, an elastic settlement response is expected and the majority of the settlement will occur rapidly, generally during the construction time frame.

To provide estimated settlements, foundation structural loads (dead and full-time live loading) of 5 klf were assumed for continuous foundations. Total settlements are anticipated to be on the order of ¾ inches or less. Differential settlement between foundations with similar loads and sizes is anticipated to be ½ of the total settlement.

Estimated settlements are based on the foundation grade soils prepared in accordance with the recommendations provided in this report. Structural fill moisture contents are critical. Failure to adequately moisture condition fills during placement will delay consolidation and may result in greater settlement of the structures and improvements.

7.2 Retaining Walls

7.2.1 Static Lateral Earth Pressures

Static lateral earth pressures are dependent on the relative rigidity and allowable movement of the retaining structure as well as the strength properties of the backfill soil and drainage conditions behind the retaining wall. A restrained retaining wall will have a higher lateral earth pressure than a retaining wall that is free to move (cantilever conditions). Restrained retaining wall lateral earth pressure is based on the at-rest soil condition (K_o). Lateral earth pressure values for the retaining wall that is free to rotate and deflect at the top of the wall (wall movement greater than 0.001H for cohesion less soils and greater than 0.01H for cohesive soils) are based on active soil conditions (K_a). It is assumed that the retaining wall will yield sufficiently to induce active soil pressures.

Table 4 (Static Lateral Earth Pressure Values) provides lateral earth pressures based on the assumption that the retaining wall is backfilled with granular, non-expansive soils in accordance with the recommendations presented in this report. The backfill should extend laterally behind the retaining wall at least the height of the retaining wall.

The following retaining wall assumptions were used to develop design parameters:

- The wall has a height of 8 feet;
- Backfill behind the retaining wall is sloped at either 2H:1V or 3H:1V (horizontal to vertical).

Lateral pressures consider flooding conditions producing saturated backfill conditions and hydrostatic pressures. The worst case lateral soil pressure scenario is a rapid drawdown, where hydrostatic pressures are imposed on the backfill side.

Table 4 – Static Lateral Earth Pressure Values

Wall Type	Static Lateral Earth Pressure (1,2,3,4)	
	3H:1V	2H:1V
Assumes movement of wall face to allow full development of active pressures (K_a).	86	93

NOTES:
1) Pounds per square foot per foot of depth;
2) Assumes no surcharge loads. Slope gradients are horizontal (H) to vertical (V);
3) Assumes backfill soils are granular complying with specifications provided in Grading and Filling (Section 8.5);
4) Assumes hydrostatic pressures.

Backfill behind the retaining wall should be densified to 90 percent relative compaction. Over-compaction should be avoided as it will increase the lateral forces exerted on the wall by the soil. Heavy equipment should not be used for placing and/or compacting backfill adjacent to the retaining wall and should be kept a minimum of three feet or at a distance determined by a1H:1V slope away from the base of the wall whichever is greater. Hand compaction equipment should be used adjacent to the wall.

7.2.2 Retaining Wall Drainage Recommendations

Unless higher seismic loads are acceptable, retaining wall drainage is required (refer to Section 7.2.3). The following retaining wall drainage design options are presented below:

- If drainage can be obtained through the front of the retaining wall, weep holes could be installed near the base of the retaining wall. Weep hole sizing and spacing is dependent on the amount of drainage anticipated behind the retaining wall. A filter cover should cover the weep holes to prevent piping and loss of backfill material. A pre-manufactured drain such as Mirafi® G100W or G100N, or approved equal is recommended. For this application, it is recommended that drain rock be used as backfill directly against the back face of the retaining wall, as presented in this report.

- Sub-drainage should be installed at the base of the foundation behind the retaining wall. The sub drain is comprised of a slotted non-corrosive piping system bedded in drain rock. Drain rock should be encapsulated with non-woven geotextile drainage fabric (refer to Table 5), have a thickness of at least 12 inches behind the back face of the retaining wall, and extend upward behind the retaining wall to 1 foot below finish grade. Drain rock shall meet the requirements of Section 200.03 (SSPWC, 2012) for a Class D backfill. The drain pipe should be sloped to allow the gravity flow of subsurface water to discharge locations away from the retaining wall. The discharge location should be protected from clogging by appropriate means.

- Alternately, a pre-manufactured drainage composite, such as Mirafi® G100W (G100N), or approved equal may be installed. If this drainage composite is selected, we should verify the filter characteristics of this drain system with the selected backfill soils. The drain system should extend to 1 foot below finish grade behind the retaining wall. Specific manufacturer's recommendations should be followed for application and installation of pre-manufactured drainage systems.

Table 5 – Drainage Geotextile Minimum Strength and Hydraulic Properties	
Trapezoid Tear Strength (ASTM D 4533)	50 lbs.
Puncture Strength (ASTM D 4833)	65 lbs.
Grab Strength (ASTM D 4632)	120 lbs.
Burst Strength (ASTM D 3786)	220 psi.
AOS (ASTM D4751)	**0.21 to 0.43 mm**

Based on the required use of this geotextile, strength properties are based on Class 3 survivability rating (AASTHO M288). Products such as a Mirafi 140N, or approved equal can be utilized for this project.

7.2.3 Seismically Induced Loading

The following definitions shall be used in the analysis of seismically induced loading:

- **PGA**: Design peak ground acceleration (PGA) is based on the design earthquake ground motions (2% probability in 50 years, IBC 2012).

- **k_h**: Horizontal ground acceleration component. This component is derived from the PGA, as described in this section.

- **K_{ae}**: Seismic active earth pressure coefficient.

- **P_{AE}**: Dynamic lateral earth pressure force: $P_{AE}=0.5\gamma H^2K_{AE}$, where γ=soil unit weight and H=height of the wall. This pressure is a combination of both static and dynamic loads such that $P_{AE}= P_a + \Delta P_{ae}$, where P_a is the static lateral pressure and ΔP_{ae} is the dynamic lateral component.

The dynamic response of most types of retaining walls is complex. Wall movements and pressures depend on the response of the soil underlying the wall; the response of the backfill; the inertial and flexural response of the wall itself; and the nature of the input motions. *Given the complex, interacting phenomena and the inherent variability and uncertainty of soil properties, it is not currently possible to accurately analyze all aspects of the seismic response of the retaining wall. As a result, models that make various simplifications about the soil, structure, and input motions are commonly used for seismic design of retaining walls* (Kramer, 1996). The standardized approach is the use of the Mononobe-Okabe method (M-O Method) that is a direct extension of the static Coulomb theory to pseudostatic conditions. In this analysis, pseudostatic accelerations are applied to a Coulomb active wedge. The pseudostatic soil thrust is then obtained from force equilibrium conditions. Using this method, K_{AE} can be determined.

The M-O method has limitations, especially when considering higher design peak acceleration and sloping backfill conditions. As seismic coefficients increase, the M-O equation degenerates into an infinite earth pressure. In this condition the failure wedge behind the wall becomes increasing larger until a nearly horizontal failure wedge results into unrealistic high pressures. This limitation of the equation is further exacerbated with a sloping backfill. These limitations with the M-O equation can be overcome by using the

Generalized Limit Equilibrium (GLE) Analysis procedure modeled by commercially available slope stability software program such as SLIDE (Reoscience, 2010). This analysis models the failure wedge as an independent block or combines several individual segments of the soil mass depending on the complexly of the backfill surface. The maximum equivalent external resistance force, which is equivalent to P_{AE}, is determined on the wall face to retain the soil wedge with a safety factor of 1.0. The wall face is modeled as a free boundary and the external resistance force is generally placed at a location of ½ the wall height.

Determination of k_h is based on the anticipated peak ground acceleration. The difference in determining the seismic induced loading for a yielding or restrained retaining wall is the value of the horizontal ground acceleration component.

- The horizontal ground acceleration for a yielding retaining wall is equal to 50 percent of the design PGA assuming some outward movement of the retaining wall is acceptable during an earthquake event (AASHTO, 2012).

- The horizontal ground acceleration for a restrained retaining wall is equal to the design PGA with no reduction (AASHTO, 2012).

The design peak ground acceleration is 0.42g (S_{DS}/2.5). For retaining walls that are yielding, a horizontal ground acceleration of 0.21g was used to determine the seismic active earth pressure coefficient.

Hydrostatic pressures on the retaining wall will occur during flooding conditions and subsequent rapid drawdown. However, it is our opinion that the potential of earthquake loading and flood conditions occurring simultaneously is remote. Consequently, dynamic lateral loads do not consider hydrostatic pressures and are based on wall drainage being incorporated into the wall design.

Table 6 (Seismically Induced Lateral Earth Pressure Values) provides seismically induced earth pressure values.

Table 6– Pseudo Static Lateral Earth Pressure Values

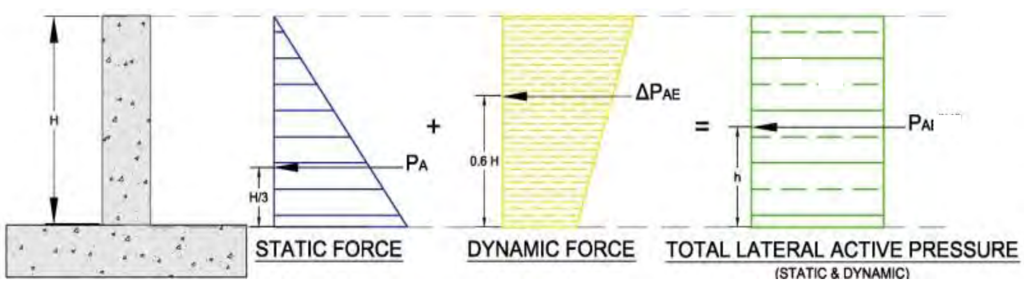
Earth Pressure Condition	**Pseudo Static Earth Pressure Coefficient**		**Seismically Induced Equivalent Fluid Pressure(1) (psf/ft)**	**Components of Lateral Pressures(1) (psf/ft)** (P_{ae}=ΔP_{ae}+P_a)	
	Slope	$K_{ae}^{(2)}$	$P_{ae} = (\gamma_{soil} * K_{ae})$	Seismic (ΔP_{ae})	Static (P_a)
Pseudo Static (assumes lateral wall displacement-active conditions)	3H:1V	0.575	72	31	41
	2H:1V	0.787	98	45	53

1) Pounds per square foot per lineal feet of wall. P_{ae} is the **total wall pressure** for pseudo static loading and includes static and seismic lateral earth pressure components. Static pressure is based on the unit weight of the soil not considering buoyant soil pressures. Assumes a Ø of 34^0 and γ of 125 pcf. Assumes no surcharge loading.

2) Based on a design peak ground acceleration (DPGA) of 0.42g. For walls that can with stand movement to mobilize active earth pressure conditions during the design earthquake event, ½ the design peak ground acceleration is the standard to use for design.

3) The static and seismic resultant forces are assumed to act at heights, ranging from 0.33 H to 0.6 H, respectively, where H is the wall height. The following equation (Kramer, 1996) may be used to calculate the total wall pressure resultant force location:

$$H = \frac{(P_a * 0.33H) + (\Delta P_{ae} * 0.6H)}{P_{ae}}$$



For example a wall with a height of 8 feet, a static force equal to P_a =53 psf/ft, and a dynamic force equal to ΔP_{ae}=45 psf/ft has a resultant force (P_{ae}=98 psf/ft) acting at a height (h) equal to about 3.6 feet.

7.2.3 Recommended Design Lateral Earth Pressures

Lateral earth pressures were evaluated for both static and dynamic conditions. The static condition assumed hydrostatic pressures and the dynamic lateral pressure assumed drained backfill conditions. The recommended design lateral earth is the higher value between these two conditions: static and dynamic. Table 7 provides recommended design earth pressure lateral loads.

Table 7 – Recommended Design Lateral Earth Pressure Values

Wall Type	Lateral Earth Pressure [1,2,3]	
	3H:1V	2H:1V
Assumes movement of wall face to allow full development of active pressures (K_a).	86	98

NOTES:
1) Pounds per square foot per foot of depth;
2) Assumes no surcharge loads. Slope gradients are horizontal (H) to vertical (V);
3) Assumes backfill soils are granular complying with specifications provided in Grading and Filling.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Site Preparation

All vegetation, topsoil, and existing rock rip-rap should be removed from the existing ditch slopes. The existing rock rip-rap could be saved for use as stabilizing fill or rock rip-rap located at the discharge location, if carefully removed to minimize soil contamination.

All areas to receive structural fill or structural loading that do not require to be stabilized (refer to Section 8.2) should be densified to at least 90 percent relative compaction in accordance with ASTM D 1557 for a minimum depth of 8 inches. It is recommended that soils have moisture contents of plus or minus 3 percent of optimum moisture (ASTM D1557) prior to densification. Moisture contents above 3 percent of optimum moisture will be acceptable if the soil horizon maintains its stability when subjected to construction equipment loads and density can be achieved in subsequent structural fill lifts. Scarification and moisture conditioning may be required to achieve the required soil moisture content recommendations. It is recommended that prior to densification, the moisture content of the soils be determined, to evaluate the need for moisture conditioning. After the densification process, a firm, stable surface should be produced.

Scarification, moisture conditioning, and densification of the channel slope will be difficult due to the steep slope gradient. It is recommended that channel slope soils are proof-rolled to densify loose soils and determine unstable soil areas.

Unstable soils due to excessive moisture content will be encountered at the bottom of the ditch and toward the lower portion of the ditch slope. These soils shall be removed and replaced with stabilizing fill as discussed in Section 8.2. The appropriate construction method to treat unstable soil areas will be determined during construction.

Dewatering could consist of sump pumps placed at the base of the excavation. Discharged ground water locations should comply with NDEP regulations and a ground water discharge permit will be required.

8.2 Stabilization Construction Methods

Two types of stabilization can be considered consisting of either rock fill stabilization (Section 8.2.1) or geogrid/stabilizing fill stabilization (Section 8.2.2). The stabilization fill layer shall be placed directly below the pipeline bedding (refer to section 8.3).

A test section is recommended to determine the required thickness of stabilizing fill. Stabilization is always a trial and error procedure with requirements and effectiveness varying within the same project. Additionally, the project manager should notify the contractor that excessive compactive efforts on the stabilizing fill can promote instability and thus it will be the contractor's responsibility to not damage otherwise firm site soils.

8.2.1 Rock Fill Stabilization

Rock fill shall be angular, well-graded, and consist of hard, durable particles without organics, clay lumps, or unstable substances. Rock fill general particle sizing is presented in the Table 8.

Table 8 – General Guideline Specification for Rock Fill

Sieve Size	Percent by Weight Passing
18-Inch	100
12-inch	20 - 40
2-inch	0 - 5

Rock fill shall have a minimum thickness of 30 inches and can be placed in a maximum single lift thickness of 18 inches. Rock fill shall be initially seated by pushing into the underlying substrate with a trackhoe bucket or dozer. The rock fill surface shall be densified by at least 5 complete passes with a minimum 10-ton roller. The final surface should be level and firm. Additional rock fill layers may be required if instability is still present.

A separation geotextile, meeting the specifications given in Table 9, should be placed over the rock fill. If the rock fill surface is coarse and angular, it is recommended to place a minimum 2-inch layer of densified drain rock, meeting the specifications of Table 11, over the rock fill prior to placement of the geotextile. The drain rock shall be densified in accordance with recommendations provided in Section 8.2.2.2.

Table 9 - Minimum Average Roll Values (MARV) for Separation Geotextile

Trapezoid Tear Strength (ASTM D 4533)	80 lbs.
Puncture Strength (ASTM D 4833)	80 lbs.
Grab Strength (ASTM D 4632)	200 lbs.
Burst Strength (ASTM D 3786)	250 psi.
Minimum permittivity (ASTM D 4491)	**≥ 0.5 sec^{-1}**
AOS (ASTM D4751)	**≤ 0.43 mm**

Based on the required use of this geotextile, strength properties are based on Class 1 survivability rating (AASTHO M288). Products such as a Mirafi 180N, or approved equal can be utilized for this project.

8.2.2 Geotextile/Stabilizing Fill Stabilization

Another option is the use of a Geotextile/Stabilizing Fill Stabilization system. This system has three separate components, as follows:

- Stabilizing fill geotextile shall be placed between the stabilizing fill and native soils to provide separation and reinforcement;
- 30-inch minimum stabilizing fill thickness placed in two lifts. The initial lift shall have a thickness of 18 inches;
- Separation geotextile between the stabilizing fill and pipe bedding/pipe backfill;

8.2.2.1 Materials

8.2.2.1.1 Stabilizing Fill Geotextile

The stabilizing fill geotextile should be woven and meet or exceed the following minimum properties presented in Table 10.

Table 10 - Stabilizing Fill Geotextile

Mechanical Properties	Minimum Average Roll Value (MARV)	
	MD (#/ft)	CD (#/ft)
Tensile Strength at ultimate (ASTM D 4595)	4600	4800
Tensile Strength at 5% strain (ASTM D 4595)	1400	1400
Apparent Opening Size (AOS)	**0.43 mm maximum**	

Products such as a Mirafi HP565, Terra Tex HPG-70 or approved equal can be utilized for this project.

8.2.2.1.2 Stabilizing Fill

Stabilizing fill shall consist of an angular, clean drain rock, meeting the requirements of Table 11. Class "D" backfill, meeting the requirements of Section 200.03.05 of the referenced SSPWC, can be used as stabilizing fill.

Table 11 – Stabilizing Fill Gradation Specifications

Sieve Size	Percent by Dry Weight Passing
2 inch	100
1½ inch	90 – 100
¾ inch	0 – 5

8.2.2.1.3 Separation Geotextile

Separation geotextile can be non-woven or woven and meet the material properties given in Table 9 or 10.

8.2.2.2 Placement Recommendations

The geotextile should be laid in accordance with the manufacturer's recommendation with a minimum joint overlap of 3 feet. Unless different recommendations are given by the manufacturer, the following minimum placement recommendations shall be followed:

- Prior to placement of the geotextile, the underlying soil surface should be smooth without sharp particles or abrupt edges;
- The geotextile shall be placed perpendicular to the pipeline direction. The overlap should be placed downstream;
- Construction equipment is prohibited from traveling directly over the geotextile;
- Preferably the stabilizing fill shall be placed from outside the excavation with a trackhoe or other similar equipment. As an alternative small track construction equipment, such as a Caterpillar D-4, or similar equipment can place the stabilizing fill over the geotextile inside the excavation;
- Stabilizing fill should be pushed ahead of the construction equipment during placement over the geotextile;
- It is recommended that the initial lift of stabilizing fill have a minimum loose lift thickness of 18 inches;
- Stabilizing fill and base layer should be densified with at least 5 passes with a vibratory plate whacker or equivalent equipment; and
- The stabilizing fill shall be fully encapsulated by the geotextile.

8.3 Trenching and Excavation

Excavations will require shoring or the trench sidewalls shall be sloped to maintain adequate stability. Regulations amended in Part 1926, Volume 54, Number 209 of the Federal Register (Table B-1, October 31, 1989) require that the temporary sidewall slopes be no greater than those presented in Table 12.

Table 12 - Maximum Allowable Temporary Slopes

Soil or Rock Type	Maximum Allowable Slopes[1] For Deep Excavations Less Than 20 Feet Deep[2]	
Stable Rock	Vertical	(90 degrees)
Type A[3]	3H:4V	(53 degrees)
Type B	1H:1V	(45 degrees)
Type C	3H:2V	(34 degrees)

NOTES:
1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
2. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.
3. A short-term (open 24 hours or less) maximum allowable slope of 1H:2V (63 degrees) is allowed in excavations in Type A soil that are 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3H:4V (53 degrees).

These regulations, including the classification system and the maximum slopes, have been adopted and are strictly enforced by the State of Nevada, Department of Industrial Relations, Division of Occupational Safety and Health. In general, Type A soils are cohesive, non-fissured soils, with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater. Type B are cohesive soils with an unconfined compressive strength between 0.5 and 1.5 tsf, while those designated as Type C have an unconfined compressive strength below 0.5 tsf. Numerous additional factors and exclusions are included in the formal definitions. Complete definitions and requirements on sloping and benching of trench sidewalls can be found in Appendix A and B of Subpart P of the previously referenced Federal Register. Appendices C through F of Subpart P apply to requirements and methodologies for shoring.

On the basis of our exploration, it is our opinion that the bulk of the site soils appear to be predominately Type B, although variations will exist. Any area in question should be considered Type C unless specifically examined by the geological engineer during construction. All trenching should be performed and stabilized in accordance with local, state, and OSHA standards. In any case bank stability will remain the responsibility of the contractor, who is present at the site, able to observe changes in ground conditions, and has control over personnel and equipment.

8.4 PIPELINE BEDDING AND BACKFILL

Bedding below the ground water table shall consist of Class "C" backfill material. Pipe bedding above the groundwater table shall comply with the specifications given for a Class A backfill material (SSPWC, 2012). A geotextile, complying with the material specifications provided in Table 9 shall be placed between the Class "A" and Class "C" bedding types.

Backfill will be imported to the site and shall comply with the requirements provided in Section 8.5 (Grading and Filling). All backfill soils shall be tested for conformance with project specifications prior to use as a trench backfill soil.

Due to the head room limitations beneath the bridge structures, densifying backfill will be difficult and alternative backfill methods will be required. It is assumed that densified backfill will at least be placed up to the springline of the pipe to provide adequate support for the pipeline. Several construction methods could be considered to fill the void between the springline to the ceiling of the arch culvert. These construction methods consist of either a CLSM (lean concrete slurry mix) or a combination of CLSM and Styrofoam.

Styrofoam may be placed in the void area above the springline to the inside wall of the arch culvert. The upper void between the top of the Styrofoam and inside of the arch culvert could be filled with pressurized CLSM.

Another construction alternative would be to leave the void, but seal the ends of the culvert with a concrete retaining wall. The intent is to prevent piping of backfill soils into the void area and create a sinkhole at the ground surface.

8.5 Grading and Filling

Structural fill is defined as supporting soil placed below foundations, concrete slabs-on-grade, or any structural element that derives support from the underlying sub-soils. Structural fill can be used in all areas not requiring stabilizing fill. Structural fill should be free of vegetation, organic matter, and other deleterious material and shall comply with the specifications presented in Table 13.

Table 13 - Guideline Specification for Structural Fill

Sieve Size	Percent by Weight Passing
4 Inch	100
¾ Inch	70 – 100
No. 40	15 – 60
No. 200	5 – 25
Maximum Liquid Limit	**Maximum Plastic Index**
40	10
Soluble sulfates:< 0.10 percent by weight of soil	

Structural fill material will likely have to be imported. Structural fill should be placed in maximum 8-inch thick (loose) level lifts or layers and densified to at least 90 percent relative compaction. The required moisture content of the soils, prior to densification, shall range between plus or minus 3 percent of optimum moisture, as determined by moisture-density relationship test results (ASTM D1557). Moisture contents greater than 3 percent of optimum moisture are acceptable if the soil lift is stable and required relative compaction can be attained in the soil lift and succeeding soil lifts.

Grading should not be performed with frozen soils or on frozen soils.

8.6 Concrete Slabs

All concrete slabs should be directly underlain by Type 2, Class B aggregate base. Unless specified in this report, the thickness of base material should be at least 6 inches. Aggregate base courses should be densified to at least 95 percent relative compaction. Prior to placement of the aggregate base course, subgrade soils shall be prepared in accordance with Sections 8.1. It is anticipated that aggregate base will placed on at least 2 feet of structural fill.

Aggregate base shall satisfy the requirements of Section 200.01 (Aggregates for Base Courses) of the *SSPWC* (2012) for Type 2, Class B, aggregate base.

Type II cement should be used for all concrete work. The contractor should submit a concrete mix design to the owner at least 10 working days prior to construction for approval.

The Reno area is a region with exceptionally low relative humidity. As a consequence, concrete flatwork is prone to excessive shrinking and curling. Concrete mix proportions and construction techniques, including the addition of excess water and improper curing, can adversely affect the finished quality of the concrete resulting in cracking, curling and spalling of slabs. We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute. Special considerations should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints and reinforcing should be provided to minimize any damage resulting from shrinkage.

9.0 CONSTRUCTION OBSERVATION AND TESTING SERVICES

The recommendations presented in this report are based on the assumption that the owner/project manager provides sufficient field testing and construction review during all phases of construction. Prior to construction, the owner/project manager should schedule a pre-job conference to include, but not be limited to: owner/project manager, project engineer, general contractor, earthwork and materials subcontractors, and geotechnical engineer. It is the owner's/project manager's responsibility to set-up this meeting and contact all responsible parties. The conference will allow parties to review the project plans, specifications, and recommendations presented in this report, and discuss applicable material quality and mix design requirements. All quality control reports should be submitted to the owner/project manager for review and distributed to the appropriate parties.

10.0 STANDARD LIMITATION CLAUSE

This report has been prepared in accordance with generally accepted local geotechnical practices. The analyses and recommendations submitted are based upon field exploration performed at the locations shown on Plate A-1 – Site Plan of this report. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. Sufficient construction observation should be completed in all phases of the project related to geotechnical factors to document compliance with our recommendations.

This report has been prepared to provide information allowing the engineer to design the project. The owner/project manager is responsible for distribution of this report to all designers and contractors whose work is affected by geotechnical recommendations. In the event of changes in the design, location, or ownership of the project after presentation of this report, our recommendations should be reviewed and possibly modified by the geotechnical engineer. If the geotechnical engineer is not accorded the privilege of making this recommended review, he can assume no responsibility for misinterpretation or misapplication of his recommendations or their validity in the event changes have been made in the original design concept without his prior review. The engineer makes no other warranties, either expressed or implied, as to the professional advice provided under the terms of this agreement and included in this report.

This report was prepared by CME for the account of the Stantec. The material in it reflects our best judgment in light of the information available to us at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based upon it, are the responsibility of such third parties. Construction Materials Engineers Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

The following appendices are included and complete this report:

Appendix A: Exploration locations from existing geotechnical investigations

Appendix B: Exploration Logs

Appendix C: USGS report

If you have any questions or require further information, please contact us.

Sincerely,

CONSTRUCTION MATERIALS ENGINEERS, INC.

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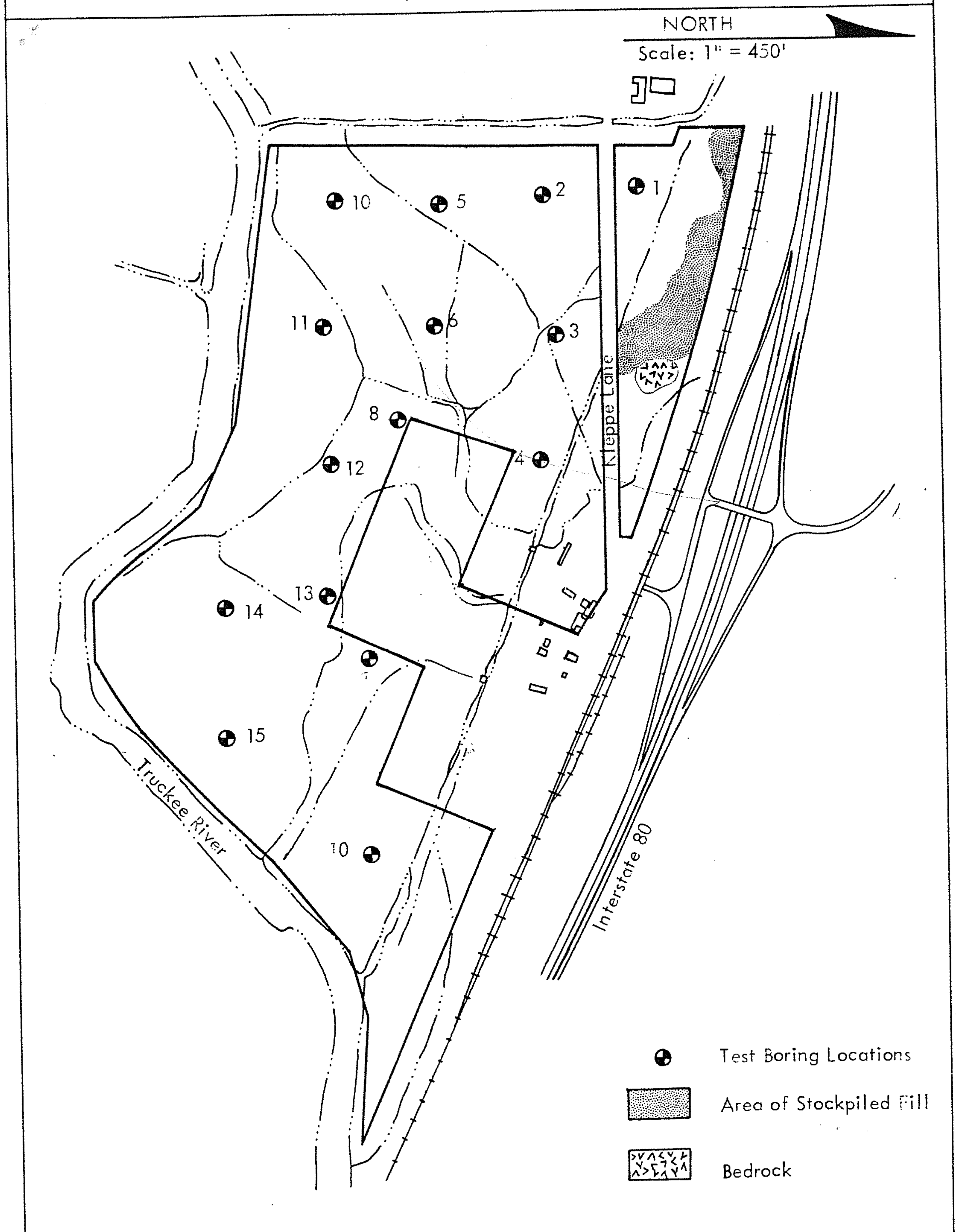
USGS Website: *Earthquake Hazards Program Quaternary Faults in Google Earth*
http://earthquake.usgs.gov/hazards/qfaults/google.php

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX A

PLOT PLAN



NEVIS INDUSTRIES/SPARKS INDUSTRIAL PARK

PROJECT NO. 545-1-783

PLAN VIEW AND CROSS SECTION

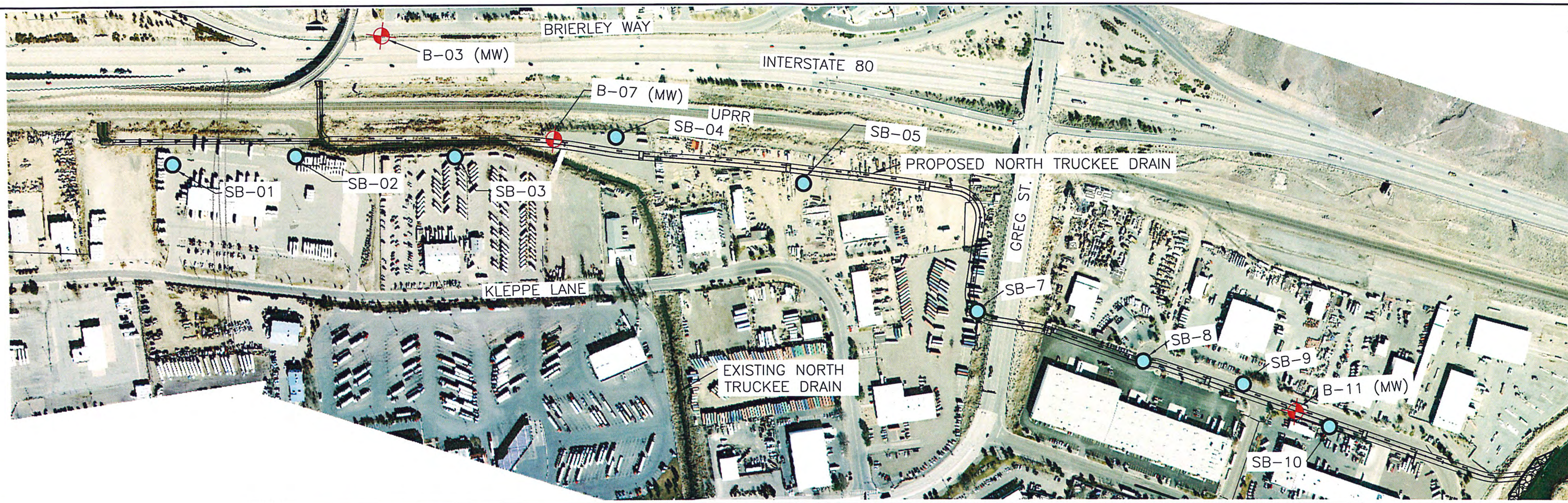
NORTH TRUCKEE DRAIN REALIGNMENT

SPARKS - NV

January 2013

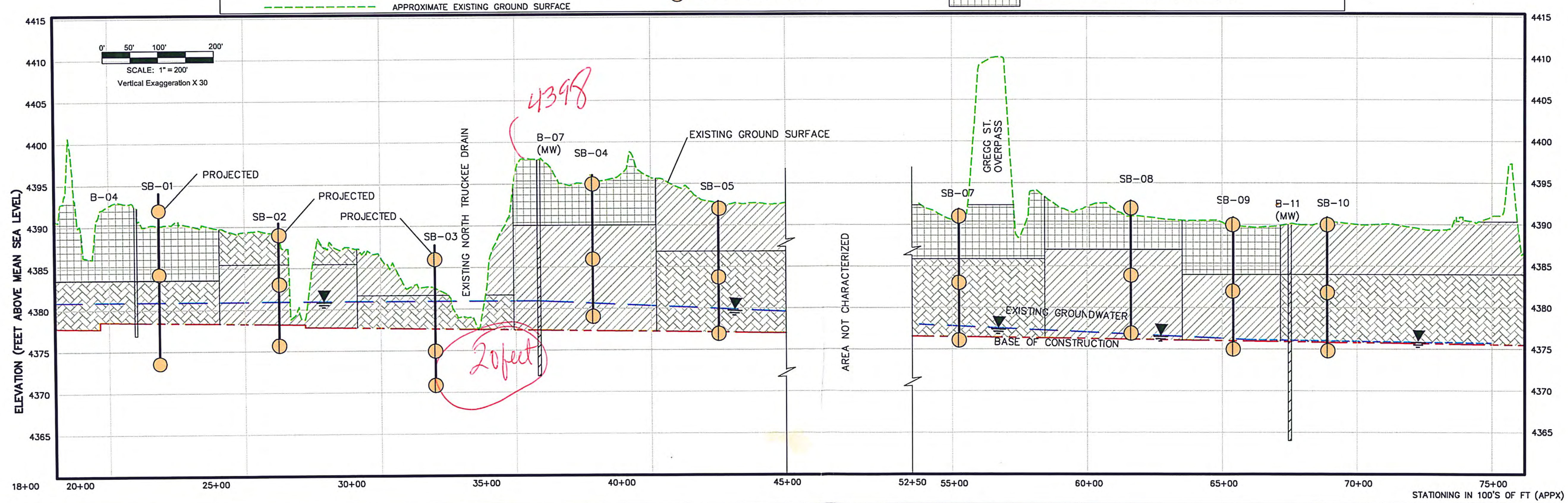
FIGURE: 2

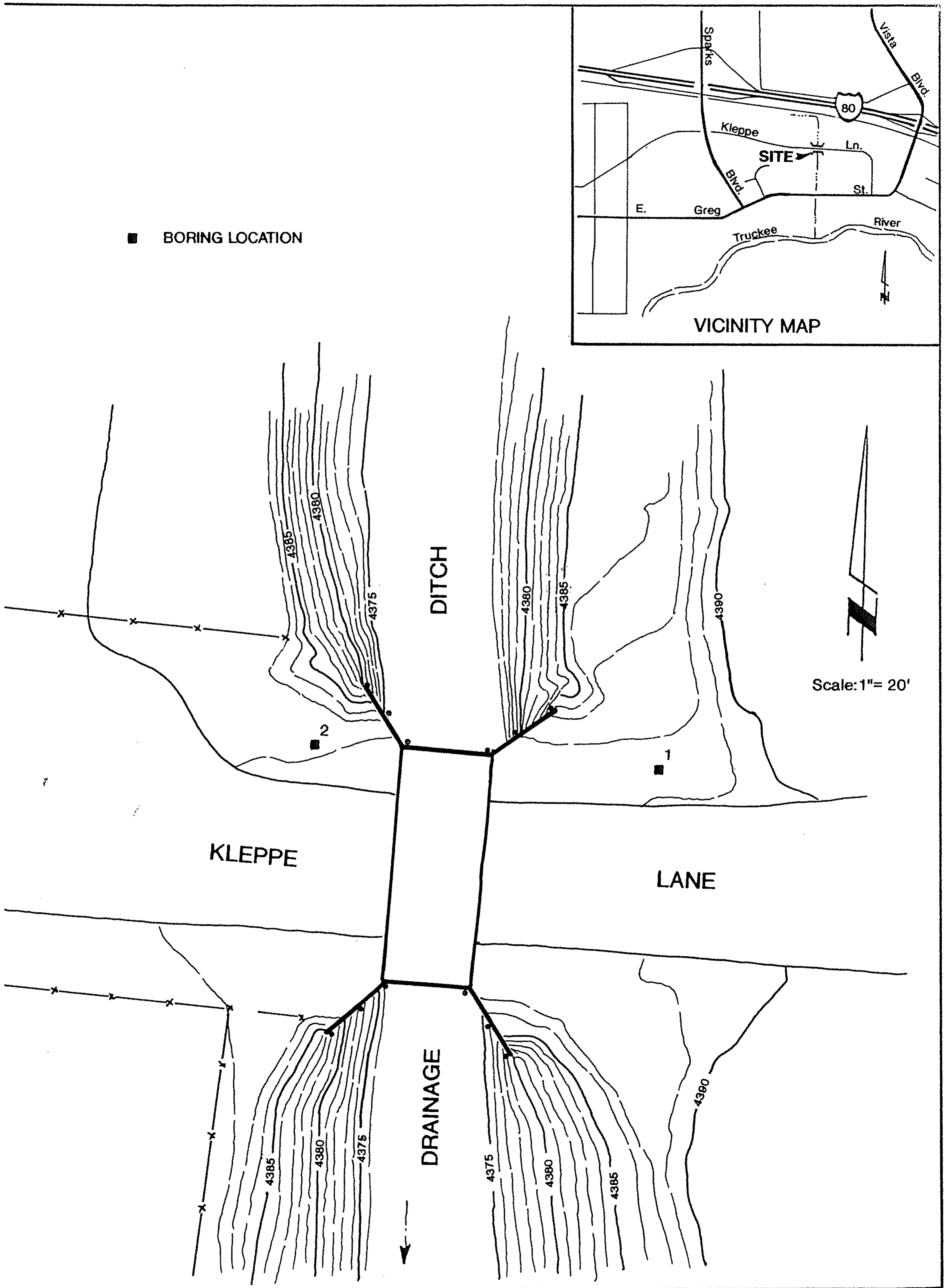
HDR



LEGEND

- GROUNDWATER MONITORING WELL
- EXISTING GROUNDWATER SURFACE ELEVATION (APRIL 2010)
- APPROXIMATE EXCAVATION EXTENT (BASE OF EXCAVATION)
- APPROXIMATE EXISTING GROUND SURFACE
- SOIL BORING (PLAN VIEW)
- SOIL BORING (PROFILE VIEW)
- SOIL SAMPLE LOCATIONS
- GROUNDWATER MONITORING WELL
- LOCKWOOD DIRECT BURRY SOIL BLOCK (>100 - <600 mg/kg)
- UNRESTRICTED REUSE (TPH-g,d,O<100mg/kg)
- LOCKWOOD BIOREMEDIATION (TPH-d,O>600mg/kg)





RENO/SPARKS, NEVADA

PLOT PLAN

Project No. 109-18-2

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX B

LOG OF BORING

BORING NO. 1
LOGGED BY DH
DATE 12-14-78
TYPE OF BORING H.S. Auger
GROUND ELEVATION ~~4385 (topo)~~
GROUND WATER DEPTH Not Encountered
DATE MEASURED

NOTES	Sample Number	Moisture Percent	Number of Blows	Depth	Log	DESCRIPTION
						0-5' Dry to moist, slightly stiff to stiff, light brown, Sandy Silt, 85% moderately plastic fines, 15% fine to very fine sand
				2		
	1A		20	4		
						5-11.5' Moist, slightly compact, brown, Silty Sand, 20-25% low plastic fines, 75-80% very fine to medium sand
				6		
				8		
	1B		12			
				10		4373
				12		11.5-18' Wet, slightly compact, bluish-grey, Poorly Graded Sand or Silty Sand, 5-10% low plastic fines, 90-95% very fine to medium sand 20½
	1C		11	14		
				16		
				18		18-20.5' Wet, soft, bluish-grey, Sandy Clayey Silt, 85% low to nonplastic fines, 15% very fine sand
	1D		7			
				20		20.5-24' Wet, very dense, bluish-grey, Gravelly Sand, 5% nonplastic fines, 80% fine to coarse sand, 15% gravel to 1"
				22		
	1E		100	24		

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

LOG OF BORING

BORING NO. 2
LOGGED BY DH
DATE 12-1-78
TYPE OF BORING H.S. Auger

GROUND ELEVATION 4386 (topo)
GROUND WATER DEPTH Not Encountered
DATE MEASURED

NOTES	Sample Number	Moisture Percent	Number of Blows	Depth	Log	DESCRIPTION
						0-5' Dry, soft, grey, Clayey Silt, nonplastic to low plastic fines, less than 5% very fine sand
				2		
	2A		5	4		
				6		5-7.5' Moist, soft, light brown, Silty Sand, 15% low plastic fines, 85% very fine to fine sand
	2B			8		7.5-16' Moist to wet, soft, dary grey, Silty Clay, low plastic fines, less than 5% very fine sand
			5			
				10		
				12		20'
	2C			14		
				16		16-21' Wet, soft, greenish-grey, Sandy Silt, 70% low plasticity fines, 30% very fine to fine sand
			4			
				18		
				20		4365
	2D		5			
				22		21-24' wet, loose, grey-brown, Gravelly Sand, 5% non-plastic fines, 85% fine to coarse sand, 10% gravel to 1"
	2F			24		24-30' Wet, loose, grey-brown, Gravelly Sand, 90% well graded sand, 10% gravel to 1" across
				26		

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

LOG OF BORING

BORING NO. 5
LOGGED BY DH
DATE 12-2-78
TYPE OF BORING H.S. Auger

GROUND ELEVATION 4387 (topo)
GROUND WATER DEPTH 12'
DATE MEASURED 12-2-78

NOTES	Sample Number	Moisture Percent	Number of Blows	Depth	Log	DESCRIPTION
				2		0-6' Dry, loose to slightly compact, light brown, Clayey Silt, nonplastic to low plastic fines, less than 5% very fine sand
	5A		11	4		
				6		6-11' Moist to wet, slightly compact, light brown, Silty Sand, 30% nonplastic to low plastic fines, 70% very fine to fine sand
	5B		13	8		
				10		
	5C		12	12		11-16' Wet, slightly compact, brown-grey, Clayey Silt, low plastic fines, less than 5% very fine sand
				14		4367 20
				16		16-20' Wet, slightly compact to compact, greenish-grey, Silty Clay, low to medium plasticity, less than 5% very fine sand
	5D		15	18		
				20		20-29' Wet, compact to dense, brownish-grey, medium grain, Poorly Graded Sand - Gravelly Sand, less than 5% nonplastic fines, 85% medium to coarse sand, 10% gravel; cobbles to boulders suspected
				22		
	5E		35	24		
				26		

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

LOG OF BORING

BORING NO. ~~10~~
LOGGED BY DH
DATE 12-12-78
TYPE OF BORING H.S. Auger

GROUND ELEVATION 4388 (topo)
GROUND WATER DEPTH 17'
DATE MEASURED 12-12-78

NOTES	Sample Number	Moisture Percent	Number of Blows
	10A		9
	10B		5
	10C		2
	10D		7
	10E		23

Depth: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26

DESCRIPTION

0-10'
Dry to moist, loose, light brown, Silty Sand, 20-30% low plastic fines, 70-80% fine to very fine sand

10-22'
Wet, very soft, grey-brown, Sandy Clayey Silt, 85-90% low to medium plasticity fines, 10-15% very fine sand

4366 22'

22-26'
Wet, very compact, bluish-grey, Silty Sand grading to Poorly Graded Sand, 10% low plastic fines, 90% fine to medium sand, minor fine gravel

26-29'
Wet, very dense, bluish-grey, Silty Gravelly Sand, 10% nonplastic fines, 90% fine to coarse sand, minor gravel to 3/8"

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.

Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

sea

PROJECT NO. 545-1-783
PLATE 2k

LOG NO.: 1
LOGGED BY: A. Jones
DATE: 11-29-94
TYPE OF BORING: Hollow Stem Auger

GROUND ELEVATION: 4389 feet (topo)
GROUND WATER DEPTH: 14.5 feet (topo)
DATE MEASURED: 11-29-94

NOTES	Sample Number	Location	Moisture Percent	Number of Blows	Depth in Feet
	1A	■	15.3	7	2–4
	1B	■		19	5–6
	1C	■	47.9	6	10–11
pp=0.75	1D	■		4	15–16
	1E	■		6	20–21

Depth in Feet: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22

Soil Class	DESCRIPTION
GM	0.0-1.5: Moist, compact, brown Silty Sandy Gravel with estimated 10% non plastic fines, 40% very fine to coarse sand, 50% gravel to 2" diameter. Cobbles encountred.
SM	1.5-5.5: Moist, loose to compact, brown Silty Sand with estimated 35% slightly plastic fines, 65% very fine to medium sand. With minor gravel and cobbles.
CL	5.5-8.0: Moist, firm, black brown Sandy Clay with estimated 80% medium to high plastic fines, 20% very fine sand.
CH	8.0-18.0: Moist, soft, black brown Sandy Clay with estimated 90% high plastic fines, 10% very fine sand.
SP	18.0-21.5: Wet, loose, black brown Poorly Graded Sand with estimated 5% non plastic fines, 95% very fine to medium sand.

16½

4371

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.
Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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LAS VEGAS, NEVADA
PHOENIX, ARIZONA

City of Sparks
Kleepe Lane Bridge
Sparks, Nevada

Sheet 1 of 1
Project No. 109-18-2
Plate 2a

LOG OF BORING

LOG NO.: 2
LOGGED BY: A. Jones
DATE: 11-29-94
TYPE OF BORING: Hollow Stem Auger

GROUND ELEVATION.: 4390 feet (topo)
GROUND WATER DEPTH: 15.5 feet (topo)
DATE MEASURED: 11-29-94

NOTES	Sample Number	Location	Moisture Percent	Number of Blows	Depth in Feet
	2A	■	15.3	4	2–4
	2B	■		13	4–6
	2C	■	29.6	8	10–12
	2D	■		5	14–16
	2E	■		12	20–22

Soil Class	DESCRIPTION
GM	0.0–1.5: Moist, compact, brown Silty Sandy Gravel Fill
SM	1.5–4.5: Moist, loose to compact, brown Silty Sand with estimated 35% slightly plastic fines, 65% very fine to medium sand. With minor gravel and cobbles.
CL	4.5–8.0: Moist, firm, brown Sandy Clay with estimated 85% medium plastic fines, 15% very fine sand.
SM	8.0–12.5: Moist, loose to compact, brown Silty Sand with estimated 35% slightly plastic fines, 65% very fine to medium sand.
SM	12.5–19.0: Moist to wet, loose to compact, black brown Silty Sand with estimated 25% non plastic fines, 75% very fine to medium sand.
SP	19.0–21.5: Wet, compact, black brown Poorly Graded Sand with estimated 5% non plastic fines, 95% very fine to medium sand.

17.5

4371

EXPLANATION

Number of Blows: Record number of blows for one foot penetration of sampler using 140 pound hammer falling 30 inches.
Description: Describe soil type by Unified Soil Classification System with emphasis on in-place or natural condition.

■ Splitspoon Sample Location

▨ Thin Wall Shelby Location

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PHOENIX, ARIZONA

City of Sparks
Kleepe Lane Bridge
Sparks, Nevada

Sheet 1 of 1
Project No. 109-18-2
Plate 2b

RAMELLI

4398

LOG OF B-07 (MW)

Depth (ft)	Well Design Info.	Dry Density lbs/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	USCS / SOIL DESCRIPTION
0	CEMENT			23		FILL: BROWN WELL-GRADED SAND WITH CLAY, GRAVEL AND COBBLES (GW-GC) dry, low plastic fines, fine to coarse sand, angular gravel up to ¾ inches in diameter.
3	BENTONITE					
4						Coarse gravel and cobbles
5	SAND			50/5"		
8						FILL: BROWN WELL-GRADED SAND WITH SILT (SW-SM) moist, non-plastic fines, fine to coarse sand, fine gravel.
10	FILTER			60		Sampler encountered gravel/cobbles > 2.5 inches in diameter
12				36		4384
14				29		Sampler encountered gravel/cobbles > 1.5 inches in diameter
14.5						DARK BROWN FAT CLAY (CH) moist, very stiff, medium plastic fines, fine sand. pp = 2.0 tsf (pocket penetrometer)
17			38	14	97 LL = 78 PI = 48	Stiff pp = 1.75-2.25 tsf (pocket penetrometer) 10'
19.5				11		Medium stiff, low to medium plastic fines pp = 1.0-1.5 tsf (pocket penetrometer)
23						4375 GRAY TO BROWN POORLY GRADED SAND WITH SILT (SP-SM) wet, very dense, non-plastic fines, fine to coarse sand, rounded gravel up to 1½ inches in diameter.
24.5				91/11"	94 9	Interbedded with approximately 6 inch layers of lean clay
28						BROWN CLAYEY SAND (SC) wet, medium dense, low plastic fines, fine to coarse sand, fine to coarse gravel.
29.5				25	43	

DEPTH IN FEET

DATE: 7-2-09
TOTAL DEPTH: 31.0 feet
LOGGED BY: J. PEASE
EQUIPMENT: MOBILE B-57, AUTOHAMMER

KLEINFELDER Bright People. Right Solutions.

LOG OF B-07 (MW)

PROPOSED NORTH TRUCKEE DRAIN REALIGNMENT GEOTECHNICAL INVESTIGATION SPARKS, NEVADA

PLATE 8

DRAFTED BY: K. Carter PROJECT NO. 102314.104

LOG OF B-08

Depth (ft)	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	SOIL DESCRIPTION
0			34		FILL: REDDISH BROWN SILTY SAND (SM) dry, non to low plastic fines, fine to coarse sand, fine to coarse gravel, organics (roots), debris.
5			15		
6					DARK BROWN LEAN CLAY WITH SAND (CL) slightly moist, stiff, low to medium plastic fines, fine to coarse sand.
10			60		
11.5					Hard pp > 4.5 tsf (pocket penetrometer)
12.5			2		BROWN SILTY SAND WITH GRAVEL (SM) slightly moist, very loose, non to low plastic fines, fine to coarse sand, fine to coarse gravel.
15	96	29	13	63 LL = 35 PI = 16	REDDISH GRAY BROWN LEAN CLAY (CL) moist to wet, medium stiff, non-plastic fines, fine to coarse sand. Increase in sand, fine to coarse gravel
17.5			50/2"		
18.5					YELLOWISH BROWN SILTY SAND WITH GRAVEL (SM) wet, very dense, non-plastic fines, fine to coarse sand, fine gravel Lenses of approximately 3-6 inches of reddish brown silt
20			73	19	
25			25		Medium dense
30			50/2"		Very dense

DEPTH IN FEET

DATE: 7-1-09
TOTAL DEPTH: 31.0 feet
LOGGED BY: S. RAHE
EQUIPMENT: CME 55, CATHEAD, TRACK MOUNTED

KLEINFELDER Bright People. Right Solutions.

LOG OF B-08

PROPOSED NORTH TRUCKEE DRAIN REALIGNMENT GEOTECHNICAL INVESTIGATION SPARKS, NEVADA

PLATE 9

DRAFTED BY: K. Carter PROJECT NO. 102314.104

CONSTRUCTION MATERIALS ENGINEERS, INC.

APPENDIX C

USGS Design Maps Summary Report

User–Specified Input

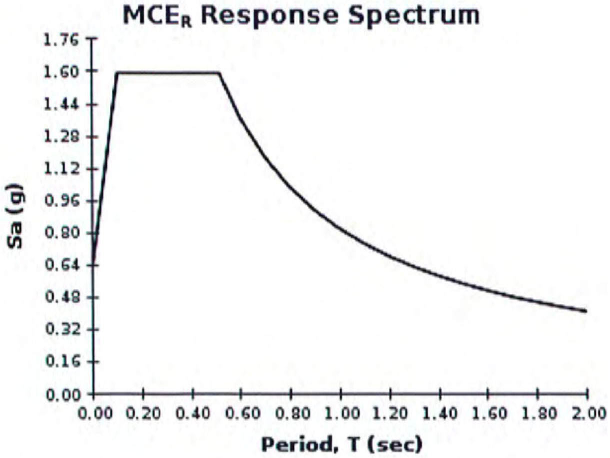
Report Title	NORTH TRUCKEE DRAIN PIPELINE Wed April 27, 2016 22:40:48 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	39.5247°N, 119.7059°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

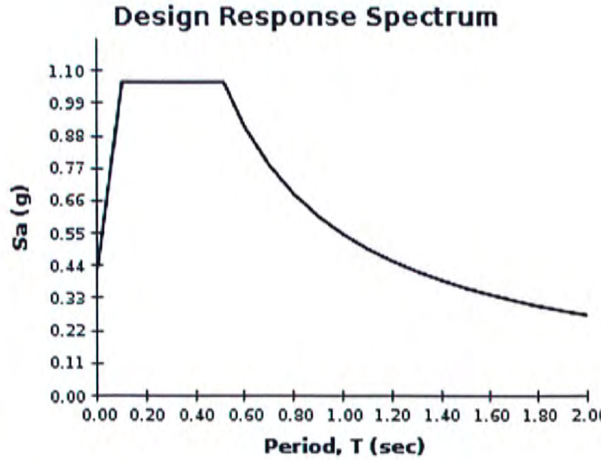
$S_S =$	1.593 g	$S_{MS} =$	1.593 g	$S_{DS} =$	1.062 g
$S_1 =$	0.547 g	$S_{M1} =$	0.820 g	$S_{D1} =$	0.547 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (39.5247°N, 119.7059°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.593$ g

From Figure 22-2 [2] $S_1 = 0.547$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.593 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.547 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.593 = 1.593$ g

---

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.547 = 0.820$ g

---

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.593 = 1.062$ g

---

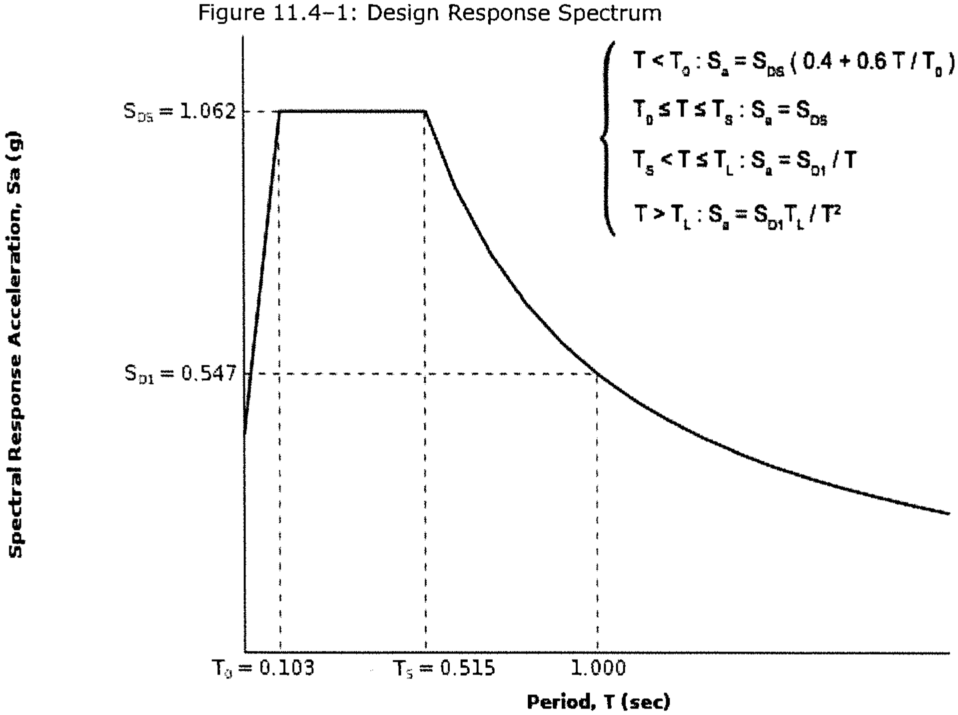
Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.820 = 0.547$ g

---

Section 11.4.5 — Design Response Spectrum

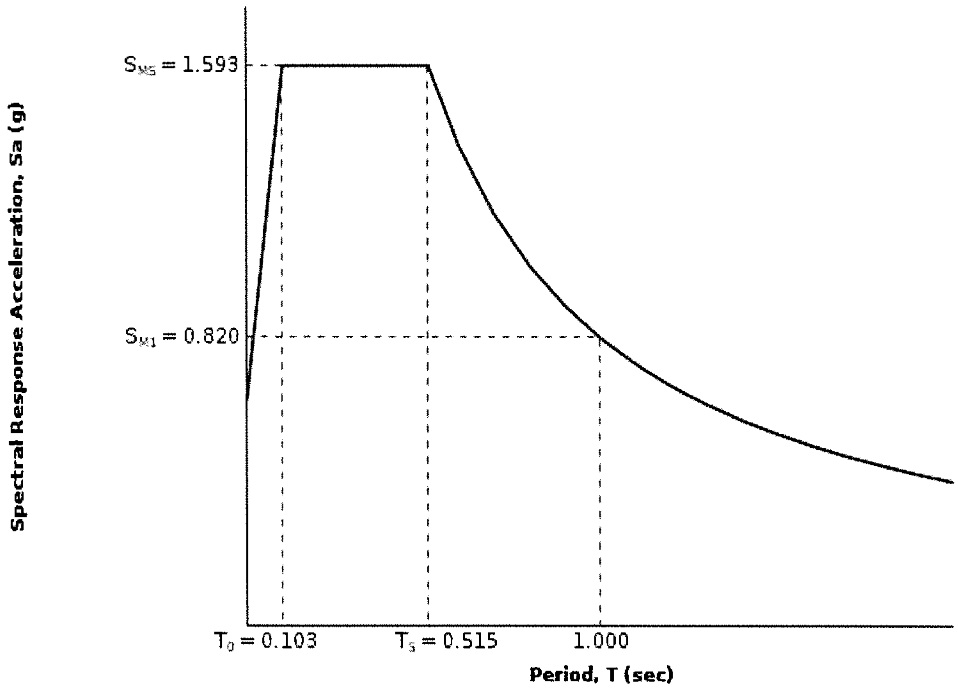
From Figure 22-12 [3] $T_L = 6$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.546

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.546 = 0.546$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.546 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.931

From Figure 22-18 [6] C_{R1} = 0.924

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.062 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.547 g, Seismic Design Category = D

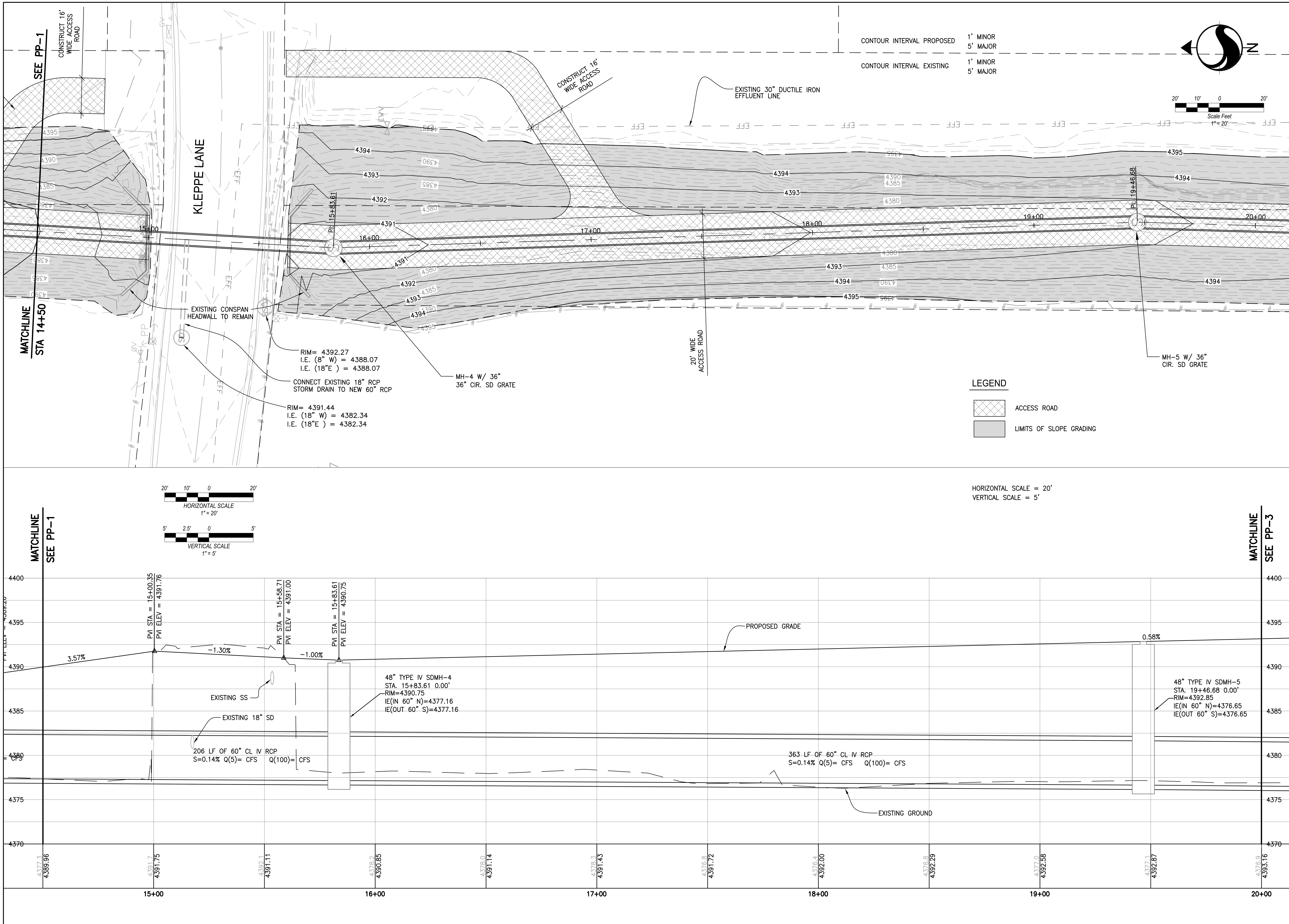
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf



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Revision	By	Appd.	YY.MM.DD
1.0 ADD 30" EFFLUENT LINE.	LH	LH	16.11.10

Issued	By	Appd.	YY.MM.DD
1.0 ADDENDUM I	LH	LH	16.11.10

ISSUED FOR BID

Client/Project
CITY OF SPARKS

NORTH TRUCKEE DRAIN
PIPE EXTENSION IMPROVEMENTS
Sparks, NV

Title
PLAN AND PROFILE
STA. 14+50 TO STA. 20+00

Permit-Seal

PROFESSIONAL ENGINEER STATE OF NEVADA LUKE D. HOFFMAN Exp: 6/30/2017 CIVIL No. 18529 11/10/2016

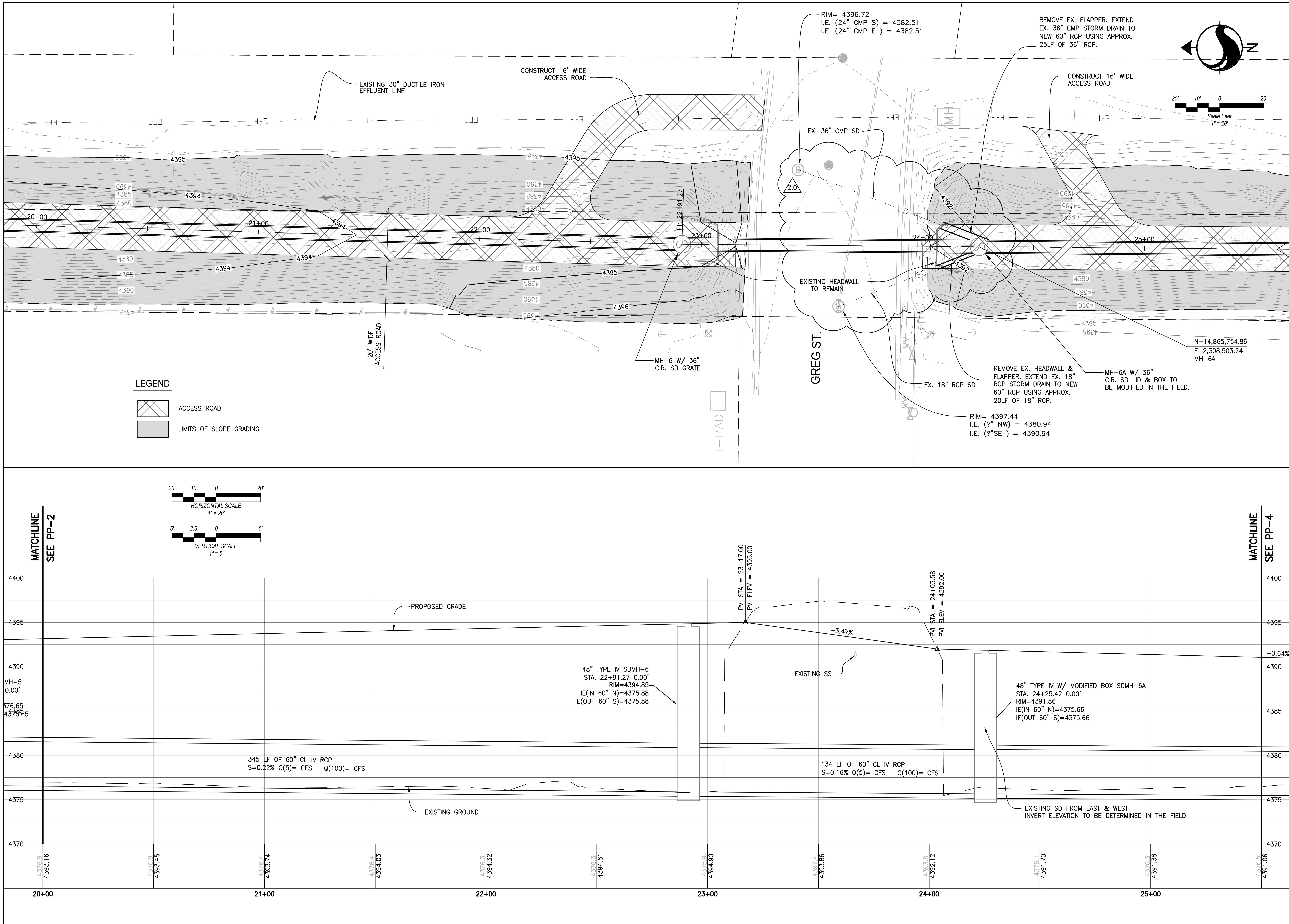
Project Number: 180101405
File Name: 01405_NTD_PP.DWG

Dwn.	Chkd.	Dsgn.	YY.MM.DD
DL	LH	DL	14.10.14

Drawing No. PP-2

Revision 0 Sheet 6 of 17

ORIGINAL SHEET - ANSI D



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Revision	By	Appd.	YY.MM.DD
2.0 ADD MH6A & UPDATE STORM DRAIN CONNECTIONS BOTH EXISTING & NEW.	LH	LH	16.11.10
1.0 ADD 30" EFFLUENT LINE.			

Issued	By	Appd.	YY.MM.DD
1.0 ADDENDUM 1	LH	LH	16.11.10

ISSUED FOR BID

Client/Project
CITY OF SPARKS

NORTH TRUCKEE DRAIN
PIPE EXTENSION IMPROVEMENTS
Sparks, NV

Title
PLAN AND PROFILE
STA. 20+00 TO STA. 25+00

Permit-Seal

PROFESSIONAL ENGINEER STATE OF NEVADA
LUKE C. HOFFMAN
Exp: 6/30/2017
CIVIL
No. 18529
11/10/2016

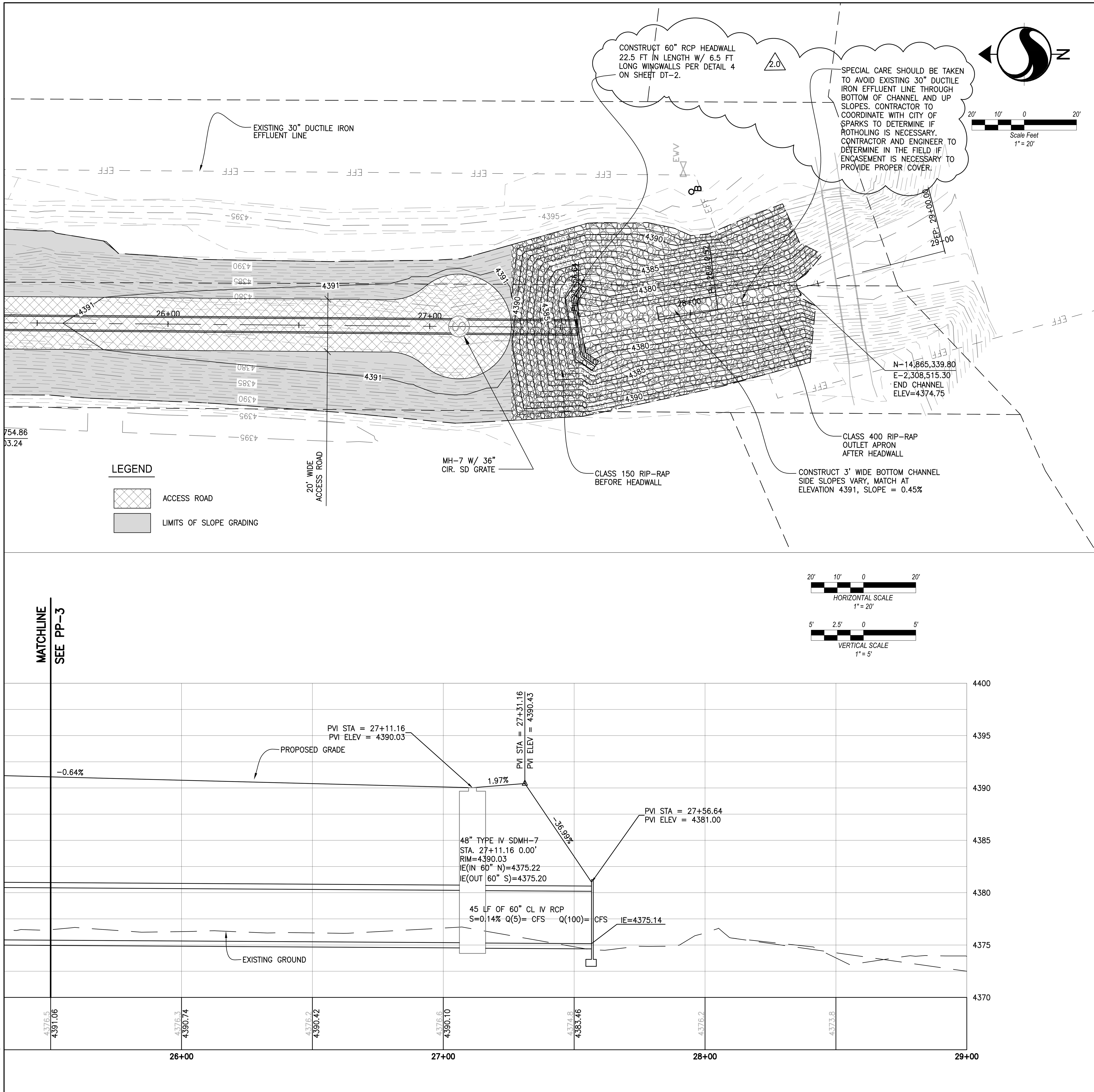
Project Number: 180101405
File Name: 01405_NTD_PP.DWG

DL	LH	DL	16.10.14
Dwn.	Chkd.	Dsgn.	YY.MM.DD

Drawing No. PP-3

Revision 0
Sheet 7 of 17

ORIGINAL SHEET - ANSI D



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Revision	By	Appd.	YY.MM.DD
2.0 ADD HEADWALL AND EFFLUENT CROSSING NOTES.	LH	LH	16.11.10
1.0 ADD 30" EFF LINE.			

Issued	By	Appd.	YY.MM.DD
1.0 ADDENDUM 1	LH	LH	16.11.10

ISSUED FOR BID

Client/Project
CITY OF SPARKS

NORTH TRUCKEE DRAIN
PIPE EXTENSION IMPROVEMENTS
Sparks, NV

Title
PLAN AND PROFILE
STA. 25+50 TO STA. 29+00

Permit-Seal

PROFESSIONAL ENGINEER STATE OF NEVADA
LUKE C. HOFFMAN
Exp: 6/30/2017
CIVIL
No. 18529
11/10/2016

Project Number: 180101405
File Name: 01405_NTD_PP.DWG

Dwn.	Chkd.	Dsgn.	YY.MM.DD
DL	LH	DL	16.10.14

Drawing No. PP-4

Revision 0
Sheet 8 of 17

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2016/11/10 3:06 PM By: Hoffman, Luke

ORIGINAL SHEET - ANSI D