WOOD RODGERS

May 20, 2016
Project No. 2664.001

Mr. Tim Scheideman
Director of Land Development – Northern Nevada
10345 Professional Circle, Suite 100
Reno, NV 89521

RE: Geotechnical Design Update
D'Andrea – Pump Station #3
Sparks, Washoe County, Nevada

REF: Geotechnical Investigation
Pump Station #3
Sparks, Washoe County, Nevada
James Edward Engineering, Inc.
February 2007
Project No. 1341.001

Dear Mr. Scheideman:

Wood Rodgers is pleased to present this geotechnical design update for the referenced project. A geotechnical design report was prepared in February 2007 for the proposed pump station. The purpose of this report is to examine the original document and to the extent necessary, bring the original geotechnical report into conformance with the requirements of the 2012 International Building Code.

One test pit was advanced in 2007; the soil profile consisted of 2 feet of sandy fat clay capping sandy gravel. Some uncompacted fill has been randomly placed across the site since performance of the original investigation, but the parcel essentially appears to be relatively unchanged since 2007. Proposed pad grade for the building pad has been raised since the original investigation was completed; current pad grade is approximately 3 ½ feet higher at elevation 4771.5 (NAVD88) feet.

Site Grading

The 2007 grading recommendations indicated that due to the presence of clay soils at least two-feet of structural fill should be maintained between foundations and slabs-on-grade and the underlying clay layer. It is the intent of this update to perpetuate the separation requirement requirement. In addition, prior to placing any fill, the existing pad grade should be scarified for a minimum depth of 12-inches, moisture conditioned to at least optimum, and compacted to not less than 90 percent of the soil's maximum dry density per ASTM D1557.

The surface two feet of structural fill beneath pad grade and foundations should consist of structural fill as indicated in Table 1. Fills beneath this grade, herein after referred to as non-select fill, may consist of any onsite generated soils. All fills shall be moisture conditioned to at least optimum and compacted to at least 90 percent of the soil's maximum dry density. Subgrade, and any fills, placed within the building pad shall be density (ASTM D6938) tested at least once for every 12-inch maximum loose lift. Fills meeting the definition of rockfill shall still be tested per ASTM D6938. If the pin cannot be advanced due to the rockiness of the fill, the test location shall be reported and refusal noted in the daily field report.

Table 1 - Guideline Specification for Structural Fill

Sieve Size (ASTM D6913)	Percent by Weight Passing
6 Inch	100
4 Inch	90 - 100
¾ Inch*	70 - 100
No. 40	15 - 70
No. 200	5 - 30
Maximum Liquid Limit (ASTM D4318)	40
Maximum Plasticity Index	15
Soluble Sulfate Level (ACI 318, Table 4.3.1)	Negligible

*Soils presenting less than 70 percent passing are acceptable and shall be referred to as rock fill providing the material is uniformly graded such that no large voids are created between individual rock particles.

Seismic Design Considerations

The site has been classified as Site Class C (very dense soil and soft rock). Because planned fill depth is less than 10-feet, it is our opinion Site Class C would still be appropriate. Based on a representative latitude and longitude of the site (39.5615 ^{o}N, -119.6805 ^{o}W) the following seismic design values were determined using the USGS seismic design web application. The USGS detailed report is presented in the appendix to this letter.

Table 2 - Summary of IBC Seismic Design Values

Lat.	Lon.	S_S	S_1	SDC	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
39.5615	-119.6805	1.498	0.500	C	1.000	1.300	1.498	0.650	0.999	0.433

Seismic Hazards

Slope Instability – the site is not proximate to any slopes aggressive enough to present a significant potential for slope instability.

Seismic Settlement – given that fills beneath the structure will be structural and the underlying clay and bedrock are not susceptible to seismic settlement, the amount of seismic settlement to be experienced should be considered negligible.

Fault Rupture – The USGS interactive fault map was consulted to identify the potential for surface rupture. No Holocene Active faults have been mapped trending through or proximate to the project site.

Liquefaction – Given proximity to bedrock and depth to groundwater, a potential for liquefaction does not exist.

Foundations

Provided the foundation soils have been prepared in accordance with the recommendations of this update, the bearing pressures presented in Table 3 can be utilized for design.

Table 3 - Allowable Foundation Bearing Pressures

Loading Condition	Maximum Net Allowable Bearing Pressure (PSF)[1]
Dead Load Plus Full Time Live Load	3,000
Dead Load Plus Live Loads, Plus Transient Wind or Seismic Loads	4,000

[1] Net allowable bearing pressure is that pressure at the base of the footing in excess of the adjacent overburden pressure.

Foundations should be set at least 24-inches below adjacent finished exterior grade. Based on our recommendations presented herein, structural settlement is anticipated to be on the order of ½-inch or less. Differential settlement across the structure would be considered less than ¼-inch.

Lateral Earth Pressures

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. The recommended coefficient of base friction is 0.45 and has been reduced by a factor of 1.5 on the ultimate soil strength. Lateral earth pressures imposed on any retaining walls are dependent on the relative rigidity and movement of the structure, soil type, and moisture conditions behind the wall. Recommended lateral earth pressures are presented in Table 4. These values do not include hydrostatic forces.

Table 4 - Lateral Earth Pressures

Condition	Active (psf/f)		Passive (psf/f)		At Rest
	Static	Pseudo-Static	Static	Pseudo-Static	
Level	35	52	400	350	55

Seismic earth pressures have been based on ½ the site Peak Ground Acceleration (PGA – 0.560g); this approach assumes that some lateral displacement of the wall due to the design event is acceptable. Traffic loading may be modeled by increasing the analyzed wall height by two feet.

Corrosivity Testing

Corrosivity testing as discussed in AWWA C105 was performed during the original investigation. Table 5 summarizes the test results and point assessment.

Table 5 - AWWA C105 Test Summary

Test	Results
Resistivity	1100 Ω-cm
pH	8.29
Oxidation-Redox Potential	+340 mV
Sulfides	Not Present

Table 5 - AWWA C105 Test Summary

Test	Results
Moisture Content	10.3
Soil Description	0 - 2' Sandy Fat Clay, 2 - 8' Sandy Gravel
Potential Stray Direct Current	Not Applicable

Soluble sulfate testing, an indicator to corrosivity potential for concrete, was in the negligible range.

Yard Improvements

The yard surrounding the pump station may be capped by an aggregate surface or plantmix bituminous pavement. If an aggregate surface is selected, we recommend the section consist of at least 8-inches of Type 2, Class B aggregate base compacted to at least 95-percent of the soil's maximum dry density (ASTM D1557). If a plantmix bituminous surface is the preferred alternative, we recommend the pavement section consist of 3-inches of Type 3 asphaltic concrete capping 6-inches of compacted Type 2, Class B aggregate base. The Type 3 pavement should still possess at least 1 ½-percent lime to aid in stripping reduction. Compaction and void requirements shall be consistent with the City of Sparks' standards for dedicated roadways.

Summary

It is our opinion once the site is graded in accordance with our recommendations, the site will be well suited for the intended use. We appreciate the opportunity to provide our services for you. Please contact our office should you have any questions or comments.

Sincerely,

Wood Rodgers, Incorporated

PROFESSIONAL ENGINEER - STATE OF NEVADA
MISCHELLE J. SMITH
EXP: 6-30-16
CIVIL
No. 6972

5-20-16

Mischelle J. Smith, PE
RE Number 6972
Expires 6/30/16

for BC

Blake D. Carter, PE
RE Number 22331
Expires 12/31/16

Design Maps Detailed Report

ASCE 7-10 Standard (39.5615°N, 119.6805°W)

Site Class C – "Very Dense Soil and Soft Rock", 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 = 1.498 g
From Figure 22-2 [2]	S_1 = 0.500 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 C, 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* ≥ 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 = C and S_S = 1.498 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 = C and S_1 = 0.500 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.498 = 1.498$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.500 = 0.650$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

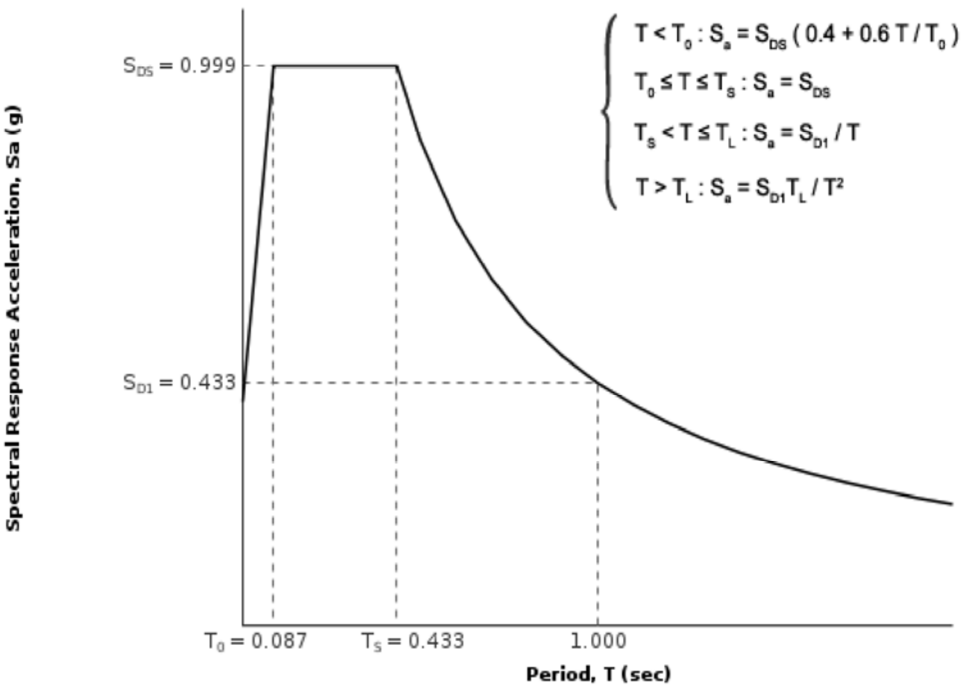
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.498 = 0.999$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.650 = 0.433$ g

Section 11.4.5 — Design Response Spectrum

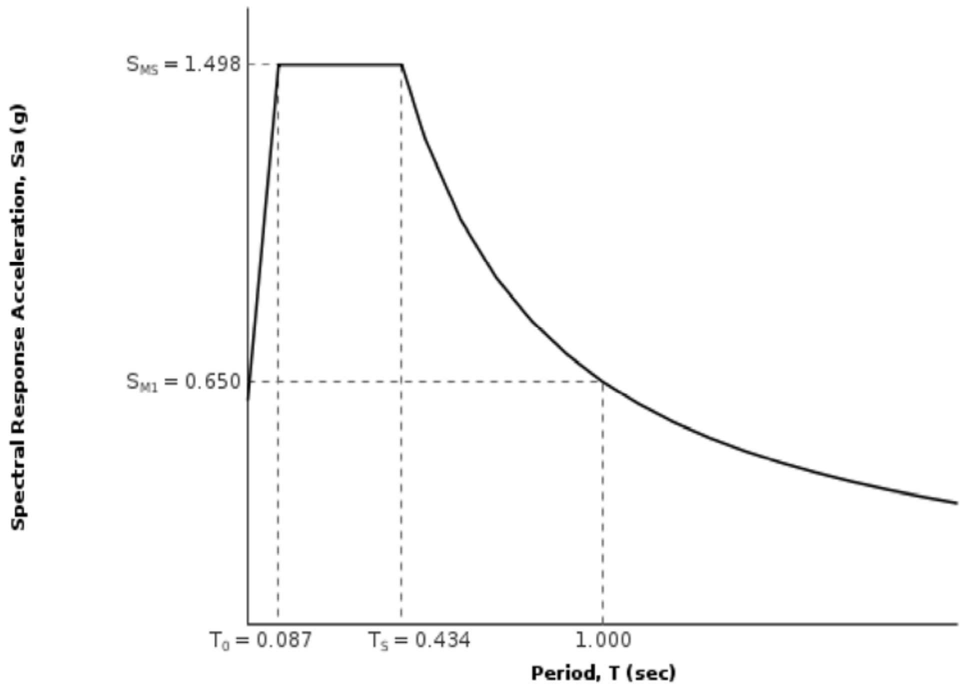
From Figure 22-12 [3] $T_L = 6$ seconds

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 = 0.560

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.560 = 0.56$ 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 = C and PGA = 0.560 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} = 0.948

From Figure 22-18 [6] C_{R1} = 0.947

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 ≤ S_{DS} < 0.33g	B	B	C
0.33g ≤ S_{DS} < 0.50g	C	C	D
0.50g ≤ S_{DS}	D	D	D

For Risk Category = I and S_{DS} = 0.999 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 ≤ S_{D1} < 0.133g	B	B	C
0.133g ≤ S_{D1} < 0.20g	C	C	D
0.20g ≤ S_{D1}	D	D	D

For Risk Category = I and S_{D1} = 0.433 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

GEOTECHNICAL INVESTIGATION
PUMP STATION #3
SPARKS, WASHOE COUNTY, NEVADA

PREPARED FOR:

LENNAR RENO
Mr. Clay Miller
10345 Professional Circle Suite 100
Reno, Nevada 89521

February 2007

JAMES EDWARD ENGINEERING
I N C O R P O R A T E D

February 7, 2007
Project No. 1341.01

Mr. Clay Miller
LENNAR RENO
10315 Professional Circle
Reno, Nevada 895211

RE: D'ANDREA – CORTINA PUMP STATION #3

Dear Mr. Miller:

This letter presents our addendum addressing Pump Station #3 for the referenced project. A geotechnical investigation report has been previously prepared by Summit Engineering Corporation for Ryder Homes, entitled *Geotechnical Investigation D'Andrea – Phase 3, Sparks, Nevada*. As stated in our proposal, JEE assumes that Lennar has secured the right for JEE to rely on this information to be accurate and true. Except where specifically modified or amended by JEE in this addendum, the recommendations presented in the Summit Engineering report are considered valid and applicable.

As can be seen in Figure 1, pad grade for the pump station has been established at elevation 4771.5, approximately 1 ½-feet above original grade. This grade will result in foundation excavations along the east side of the structure to be founded in cut, while the remainder of the structure will be founded in fills grading to approximately 8-feet in thickness. One test pit was advanced in the immediate vicinity of Pump Station #3. The soil profile encountered typically consisted of 2 feet of sandy fat clay capping sandy gravel with few low plastic fines.

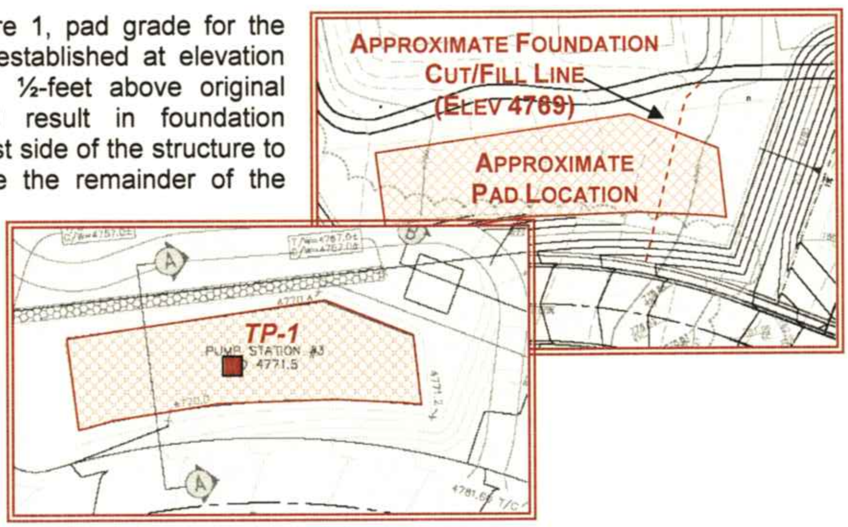


FIGURE 1 – Original Topography & Current Design Pad Location

Due to the presence of the surface clays and the potential for a cut/fill contact to trend beneath the structure, we recommend that all structural improvements be founded on not less than 2-feet of compacted structural fill. Where clay soils are penetrated in parking and drive areas before the 2-feet zone is met, the overexcavation may be terminated. Prior to placing fill, the exposed subgrade shall be scarified, moisture conditioned to at least optimum, and compacted to not less than 90 percent of the soil's maximum dry density (ASTM D 1557). All fill placed for Pump Station #3 shall meet the structural fill requirements set forth in our mass grading addendum dated July 5, 2006 (attached). In addition, the fill must exhibit a negligible soluble sulfate level at the in-place borrow source. To facilitate foundation and utility construction the surface two feet of the structural fill for the pad shall consist of the 4-inch minus structural cap.

The site can be classified as a Site Class C (very dense soil and soft rock) listed in Table 1615.1 of the 2003 International Building Code. Based on the average latitude and longitude of the site (39.5615°N, -119.6805°W), the mapped spectral response accelerations for the 0.2 seconds (S_s) and 1 second (S_1) periods are 1.36 and 0.49, respectively (1997 Maximum Considered Earthquake Ground Motion for the Conterminous 48 States, http://eqdesign.cr.usgs.gov/html/design-lookup.html). Based on these mapped spectral response accelerations, the Site Coefficients F_a and F_v, as a function of site class, are 1.0 and 1.31, respectively.

Provided the foundation soils have been prepared in accordance with the recommendations of this update, the bearing pressures presented below can be utilized for design. For frost protection, footings should all be set at least two feet below adjacent outside or unheated interior finish grades. Based on the recommendations contained within this report structural settlement is anticipated to be on the order of ½-inch, or less. Differential settlement across the structure should be less than ¼-inch.

ALLOWABLE FOUNDATION BEARING PRESSURES	
Loading Conditions	**Maximum Soil Net Allowable Bearing Pressures[1,2] (pounds per square foot)**
Dead Loads plus full time live loads	3,000
Dead Loads plus live loads, plus transient wind, or seismic loads.	4,000

NOTES:
1. The net allowable bearing pressure is that pressure at the base of the footing in excess of the adjacent overburden pressure.
2. Foundation minimum width and depth shall be as established by code.

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. The recommended coefficient of base friction is 0.45 and has been reduced by a factor of 1.5 on the ultimate soil strength. Lateral earth pressures imposed on retaining walls are dependent on the relative rigidity and movement of the structure, soil type, and moisture conditions behind the wall. Recommended lateral earth pressures are presented in the following table. These values do not include hydrostatic or seismic forces.

LATERAL EARTH PRESSURES	
Wall Type	Static Lateral Earth Pressure (psf/f)
Rotation of wall face to allow full development of Active Pressure	35
Passive Pressure	350

Since the line into the pump station will be approximately 4' below final grade, grading into the site, corrosivity test data was performed on the native granular soils. The following items address corrosivity data as covered in AWWA Standard C-105.

1 – Earth resistivity - 1100 Ω-cm (saturated paste – Method 2510B, 10 points)
Earth resistivity was determined on a remolded sample obtained from the sandy gravels extending below the surface clays. Top of pipe for the water line has been established at 4 feet below roadbed grade. The test sample was obtained at an interval of 2 feet to 4 feet which is representative of the pipe embedment units. Groundwater was not present at the time of our investigation and lies at a depth below the zone of influence with the piping associated with Pump Station #3. Therefore, the percentage of time the soil is likely to be water saturated is anticipated to be less than 5 percent. The soil sample was obtained on February 1, 2007. Due to the weather pattern and depth of sample, freezing soil conditions were not present at the time of sampling.

2 – pH – 8.29 (saturated paste – Method 9045B, 0 points)

3 – Oxidation-reduction potential – (0 points)
Results of redox potential indicated a potential greater than +100 mV indicating that the soil is sufficiently aerated (+340 mV).

4 – Sulfides – (0 points)
No effervescence was noted from the soil sample upon being subjected to the 3-percent sodium azide in a 0.1N iodine solution indicating that the sulfide concentration is negative.

5 – Moisture Content – Good drainage, generally dry (0 points)
10.3 percent by dry weight of soil.

6 – Soil description
0 – 2' **Sandy Fat Clay (CH) with Gravel and Cobbles** – stiff to hard, moist, dark brown
2 – 8' **Sandy Gravel (GP) with Few Low Plastic Fines** – medium dense to dense, slightly moist, light brown

7 – Potential Stray Direct Current – Not applicable

Based on the soil characteristics and the AWWA guidelines the native granular soil is considered corrosive to ductile-iron pipe

Sulfate testing was also performed on the sample. Sulfate testing on the native soils yielded results in the negligible range. IBC requirements for concrete exposed to sulfates and de-icing salts are presented in the following table. Mix designs, with associated qualification tests and certificates of compliance, shall be in accordance with the ACI 211.1 trial batch method and shall be submitted to the owner for review at least two weeks prior to use. In addition, a negligible sulfate limit has been specified for structural fill. All concrete 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.

IBC REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATES AND DEICING SALTS									
Use	Exposure		Cement[3]	Coarse Aggregate Size (in)[1,3]	Minimum Sacks of Cement/ Yard[3]	Min 28 Day Compressive Strength (psi)[3]	Maximum Water/ Cement Ratio[3]	Maximum Slump (in)[3]	Entrained Air (%)[3]
Structural –Foundations, Stemwalls, And Interior Slabs-on-Grade	Sulfates[4]	Neg. Mod. Severe V. Sev.[5]	Type II-V or Type II with Flyash	- - - -	5.5 6.0 6.5 6.5	3000 4000 4500 4500	0.50 0.50 0.45 0.45	4 4 4 4	- - - -
	Recommended		**Type II-V or Type II +Flyash**	**#67 (SOG)**	**5.5**	**3000**	**0.5**	**4**	**-**
Exterior[2] – Curbs, Gutters, Walks and Driveways	Sulfates[4]	Neg. Mod. Severe V. Sev.[5]	Type II-V or Type II with Flyash	- - - -	5.5 6.0 6.5 6.5	3000 4000 4500 4500	0.55 0.50 0.45 0.45	4	4 ½ – 7 ½
	De-Icing Salts	Severe Weathering Region		-	-	4500	0.45	4	6 min
	Recommended		**Type II-V or Type II +Flyash**	**# 67**	**6.5**	**4500**	**0.45**	**4**	**6 min**

[1] Aggregate size may be adjusted providing the contractor can acceptably demonstrate his ability to work and finish the product, and all other requirements are met.
[2] Fibers may be added to increase durability.
[3] Requires the project structural engineer's approval
[4] Testing may be warranted once design grades are approached.
[5] In very severe sulfate exposure areas, Type V plus Pozzolan cement is required.

We appreciate the opportunity to provide these services for you. Please do not hesitate to contact our office should you have any related questions or comments.

Sincerely,

JAMES EDWARD ENGINEERING
INCORPORATED

James G. Smith, PE
President

Mischelle J. Smith, PE
Engineering Manager
RE Number 6972
Expires 6-30-08

PROFESSIONAL ENGINEER - STATE OF NEVADA
MISCHELLE J. SMITH
2-7-7
Exp. 6-30-08
CIVIL
No. 6972

cc: Mr. Seth Padovan, PE, TBG Engineering
Mr. Tim Grover, PE, TMWA

LOG OF TEST PIT NO. 1

PROJECT NAME:	D'ANDREA PUMP STATION #3
LOCATION:	SEE PLAN
DATE:	2/1/2007

PROJECT NUMBER:	1341.01
SURFACE ELEVATION:	SEE PLAN
EXPLORATION EQUIPMENT:	BACKHOE

Depth in Feet	Unified Soil Classification	Graphical Log	Sample	Sample Type	Sample No.	Moisture	Visual Description	Pocket Penetrometer (tsf)	Moisture Content (% of Dry Weight)	Laboratory Tests
0 - 2	CH			B	1A	S	0 - 2' Sandy Fat Clay (CH) - stiff, slightly moist, red brown, with Cobble float			
2 - 4	GP			B	1B	S	2 - 7' Sandy Gravel (GP) with Few Low Plastic Fines - dense to very dense, slightly moist, tan			
4 - 7										

Bottom of Test Pit @ 7 Feet
No Free Water Encountered

GROUNDWATER & SOIL MOISTURE					SAMPLE TYPE		LABORATORY TESTS
	Depth	Hour	Date	D - DRY	A - Drill Cuttings	B - Bulk Sample	A- Atterberg Limits
∇	NE		2/1/2007	S - SLIGHTLY MOIST	C - CME Sample	R - Rotary Cuttings	B- Grain Size Distribution
▼				M - MOIST	S- 2" O.D. 1.38" I.D. Tube Sample		C- Consolidation
NE- No Free Water Encountered				V - VERY MOIST	U- 3" O.D. 2.42 " I.D. Tube Sample		MD- Moisture/Density
				W - WET	T- 3" O.D. Thin-Walled Shelby Tube		DS - Direct Shear

JAMES EDWARD ENGINEERING
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Plate
A-2