GEO-TEST

GEOTECHNICAL ENGINEERING SERVICES JOB NO. 1-20905, REV. 1 DE FOURI STREET BRIDGE SANTA FE, NEW MEXICO

PREPARED FOR

THE LOUIS BERGER GROUP, INC.

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May 7, 2013
Job No. 1-20905, Revision 1

The Louis Berger Group, Inc.
2019 Galisteo St., Suite M-1
Santa Fe, New Mexico 87505

Attn: Mr. Richard K. Rotto, P.E.

RE: Geotechnical Engineering Services
De Fouri Street Bridge
Santa Fe, New Mexico

Dear Mr. Rotto:

Submitted herein is Revision 1 to our Geotechnical Engineering Services Report for the above referenced project. The report contains the results of our field investigation and laboratory testing, and recommendations for bridge foundation design.

It has been a pleasure to serve you on this project. If you should have any questions, please contact this office.

Respectfully submitted:
GEO-TEST, INC.

Robert D Booth, P.E.

ROBERT D. BOOTH
NEW MEXICO
5711
LICENSED PROFESSIONAL ENGINEER

Reviewed by:

Charles M. Miller, P.E.

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TABLE OF CONTENTS

INTRODUCTION 1

PROPOSED CONSTRUCTION 1

FIELD EXPLORATION 1

LABORATORY TESTING 2

SUBSURFACE SOIL CONDITIONS 2

CONCLUSIONS AND RECOMMENDATIONS 3

FOUNDATIONS 3

- Shallow Spread-Type Footings 3
- Lateral Loads 4
- Drilled Pier Foundations 4
- Lateral Pier Loads 4

CONSTRUCTION CONSIDERATIONS 5

ABUTMENT WALLS 6

SITE SEISMICITY 6

SITE GRADING 6

MOISTURE PROTECTION 7

FOUNDATION REVIEW AND INSPECTION 7

CLOSURE 8

BORING LOCATION MAP 9

BORING LOGS 10

SUMMARY OF LABORATORY RESULTS 13

GRAIN SIZE DISTRIBUTION 15

PIER DESIGN CHARTS ..... APPENDIX A

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INTRODUCTION

This report presents the results of geotechnical engineering services performed by this firm at the site of the proposed widening of the existing De Fouri Street Bridge crossing the Santa Fe River in downtown Santa Fe, New Mexico.

The objectives of this investigation were to:

1) Evaluate the nature and engineering properties of the subsurface soils underlying the bridge site.

2) Provide recommendations for the design and construction of the new bridge foundations to support the widened portion of the bridge.

The investigation includes subsurface exploration, selected soil sampling, laboratory testing of the samples, performing an engineering analysis and preparation of this report.

PROPOSED CONSTRUCTION

It is understood that the project consists of widening of the existing vehicular De Fouri Street Bridge crossing the Santa Fe River in downtown Santa Fe, New Mexico. The existing bridge is a two-span bridge structure with span lengths of about 26.5 feet, bearing on unreinforced masonry abutments and an unreinforced masonry pier. It is our understanding that both masonry abutments and the center pier bear at an elevation of about 6929 feet, or about 11 feet below the elevation of the existing bridge deck. It has been determined to replace the entire bridge with a new, 50 foot long single span bridge. The new bridge will be widened to the west side of the existing bridge for a total width of approximately 41 feet.

Foundation loads (factored) on the new abutments will be relatively heavy, on the order of 17 kips per foot for shallow foundations and 160 kips per shaft for drilled pier foundations (or 112 kips per pile for pile foundations) using LRFD methodology.

Should structural loads or project details vary significantly from those outlined above, this firm should be notified for review and possible revision of recommendations contained herein.

FIELD EXPLORATION

Two exploratory borings (1 near each abutment) were drilled to depths ranging

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from approximately 30 to 72.5 feet below existing site grades. The locations of the borings are shown on the Boring Location Map, Figure 1. The soils encountered in the borings were continuously examined, visually classified, and logged during the drilling operation. Drilling was accomplished using a truck mounted drill rig equipped with a 5-inch diameter continuous flight hollow stem auger. Subsurface materials were sampled at five-foot intervals or less utilizing a 2.0 inch O.D., I.4 inch I.D. open tube split barrel sampler driven by a standard penetration test hammer. The texture, moisture content, relative density, color, and other physical properties of soils were observed and noted by the field engineer. Samples along with drill cuttings were visually classified to maintain a continuous geologic/lithologic log of the soils encountered in the borings. The boring logs are presented in a following section of this report.

LABORATORY TESTING

Selected soil samples were tested in the laboratory to evaluate the engineering properties of the soils. Moisture contents were determined to evaluate the various soil deposits with depth. The results of these tests are shown on the boring logs.

Sieve analysis and Atterberg limits tests were performed to aid in soil classification. The results of these tests are presented on the Summary of Laboratory Results and on the individual test reports included in the following sections of this report.

SUBSURFACE SOIL CONDITIONS

As indicated by the exploratory borings, the soils underlying the proposed abutment locations are relatively uniform generally consisting of a 2 layer geotechnical profile which can be described as follows:

- Stratum 1 A surface layer consisting predominantly of sand and gravel with varying amounts of cobbles which extends from the surface to depths of about 8 and 9½ feet below existing site grades at the boring locations. These soils are generally non-plastic or low in plasticity and range from medium dense to very dense. Due to the proximity of the borings to the existing bridge abutments, it is believed that many of the soils encountered within this stratum consist of man-made fill soils associated with backfill placed behind the existing masonry abutments.

- Stratum 2 Directly underlying the surface stratum, clayey and/or silty sands with varying amounts of gravel were encountered and extended to depths between 30 and 72½ feet below grade, the full depth of the

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borings. These soils rangd from non-plastic to medium in plasticity and are very dense. Although these subsurface materials were logged as very dense soils, most of these materials are believed to consist of weathered bedrock of the Santa Fe Formation. The Santa Fe Formation in this area is a sedimentary rock deposited approximately 1 to 5 million years ago and has the engineering properties of a very dense soil or weak rock. Based on experience by this firm in the general vicinity of the project, these materials extend to depths well in excess of 100 feet.

Detailed descriptions of the strata encountered are presented on the boring logs.

No free groundwater was encountered in the borings, although a very moist zone encountered a depth of 40 feet at boring location no. 1. With this exception, soil moisture contents were relatively low to moderate throughout the depth of the borings.

CONCLUSIONS AND RECOMMENDATIONS

Based on the subsurface soil conditions underlying the proposed bridge, along with the magnitude of loads involved, it is recommended that the proposed bridge abutments be supported on shallow spread-type footings bearing directly on the native Santa Fe Formation soils encountered at depths between 8 and 10½ feet at the boring locations. The footings should bear at minimum depth of 4 feet below lowest adjacent finished grade, or below the maximum depth of scour, whichever is the greater depth. As an alternate to shallow foundations, deep foundations would also provide reliable support of the bridge structure. Although various types of deep foundations could be used, considering the subsurface soil conditions along with the magnitude of the foundations loads involved, the use of drilled, straight, cast-in-place concrete piers is considered the most economical deep foundation system. Detailed recommendations for the design of shallow spread-type footings, as well as drilled pier foundations, along with recommendations for the required earthwork, are presented in the following sections of this report.

FOUNDATIONS

Shallow Spread-Type Footings

Conventional spread-type footings, bearing directly on the very dense silty or clayey sand with gravel native soils (Santa Fe Formation soils) are recommended for the support of the bridge abutments. An ultimate soil bearing capacity of 15,000 pounds per square foot (psf) is recommended for the design

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of shallow spread-type footings bearing at a minimum depth of 4 feet below finished site grades, or below the maximum depth of scour, whichever is the lower elevation. The minimum recommended width of footing is 2.0 feet.

Maximum settlements of foundations designed and constructed as recommended herein are estimated not to exceed 1.0 inch. Differential movements should be less than 75 percent of total movements.

Lateral Loads

Resistance to lateral forces will be provided by soil friction between the base of the footings and passive earth resistance against their sides. A coefficient of friction of 0.45 should be used for computing the lateral resistance between bases of footings and the soil. With backfill as recommended in the site grading section of this report, a passive soil resistance equivalent to a fluid weighing 325 pounds per cubic foot should be used for analysis.

Drilled Pier Foundations

Ultimate downward capacities for various diameters of drilled, straight cast-in-place concrete piers are presented on the design chart presented in Appendix A of this report. The design chart presents the ultimate downward capacities of 24, 30, and 36 inch diameter drilled piers versus depth below existing site grades. The ultimate capacities were developed using skin friction only and neglect any end bearing capacity.

The capacities apply to the ultimate soil supporting capability of isolated piers. Piers can be considered isolated provided they are at least 6 diameters, center to center. Piers spaced at 3 diameters, center to center, would require a group capacity reduction factor of 0.67. A linear interpolation between 0.67 for 3 diameter spacing to 1.0 for 6 diameter spacing is recommended.

Maximum settlements of drilled pier foundations designed and constructed as recommended herein are estimated not to exceed 1.0 inch for the soil moisture contents encountered during this investigation or moisture contents introduced during construction. Differential movements should be less than 75 percent of total movements. Significant moisture increases in the supporting soils after construction could cause additional movements.

Lateral Pier Loads

Resistance to lateral forces on the drilled pier foundations can be analyzed by this firm using the computer program LPILE Plus for windows, Version 5.0 by

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Ensoft, Inc., once the configuration of the piers is established and lateral forces are known.

CONSTRUCTION CONSIDERATIONS

The following criteria should be utilized in the installation of the straight, drilled, cast-in-place concrete piers.

1) Straight drilled pier excavations should be advanced with a single flight auger, or bucket auger bitts, to their design depth. It should be verified by observation and measurement the excavations are open to that depth. The auger should be placed back in the holes and two additional passes made to clean loose material present in the bottom of the holes.

2) It is anticipated that only minor caving and sloughing will occur as a result of the drilling operations. Accordingly, concrete quantities are anticipated to be near the neat volumes indicated by the plans.

3) The drilled pier excavations should be observed by a representative of the geotechnical engineer to verify proper diameter, depth, bearing conditions, reinforcement, and concrete placement.

4) Concrete should be placed through a hopper or other device approved by the geotechnical engineer so that it is channeled in such a manner to free fall and clear the walls of the excavation and reinforcing steel until it strikes the bottom. Adequate compaction will be achieved by free fall of the concrete up to the top 5 feet. The top 5 feet of concrete should be vibrated in order to achieve proper compaction.

5) The concrete should have an ultimate compressive strength of not less than 3,000 psi and be workable and plastic. A design slump of 4 to 6 inches should be achieved.

In any case, all drilled piers should be installed in accordance with Section 502 – Drilled Shaft Concrete Bearing Piles of the New Mexico Department of Transportation Standard Specifications for Highway and Bridge Construction.

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ABUTMENT WALLS

Lateral pressure against vertical abutment walls will depend upon their degree of restraint. Walls which are restrained so as to limit movement at the top to less than 0.001 times the height of the wall should be designed for an "at rest" earth pressure of 55 pounds per square foot per foot of depth. Walls free to move at the top should be designed of an "active" earth pressure equal to 35 pounds per square foot per foot of depth. The recommended equivalent fluid pressures do not include hydrostatic forces and are applicable to a condition of horizontal backfill without surcharge loads. Analysis of earth pressures produced by sloping backfill or surcharge loads can be provided by this firm upon request. If hydrostatic forces are anticipated, an additional pressure of 60 pounds per square foot per foot of depth should be added to the design pressures recommended above.

During backfilling, the contractor should be limited to the use of hand operated compaction equipment within a zone of about 5 feet horizontally from the back of the walls. The use of heavier equipment could apply lateral pressures well in excess of the earth pressure, particularly over the upper portions of the walls. Backfill should be compacted in accordance with the criteria outlined in the Site Grading section of this report.

SITE SEISMICITY

Based on the Standard Penetration Test blow counts, or N-values, along with our experience in the general area of the site, Site Class C should be used for structural design in accordance with IBC 2009.

SITE GRADING

The following general guidelines should be included in the project construction specifications to provide a basis for quality control during site preparation. It is recommended that all structural fill and backfill be placed and compacted under engineering observation and in accordance with the following:

1) After the ground surface has been cleared and grubbed, and any necessary excavations have been made, the exposed soils should be densified.

2) Densification of the exposed soils should consist of scarifying to a depth of 6 inches, moisture conditioning to as deep as practicable to the optimum moisture content or above, and compacting the area to a minimum of 95 percent of maximum dry density as determined in

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accordance with ASTM D-1557.

3) Most of the existing soils will be suitable for use as structural fill or backfill, although the removal of oversized particles may be required. All fill should be non-expansive, free of vegetation and debris, and contain no rocks larger than 3 inches. The gradation of the backfill material, as determined in accordance with ASTM D-422, should be as follows:

Sieve Size	Percent Passing
3 inch	100
No. 4	60-100
No. 200	10- 45

4) The plasticity index of the material should be no greater than 15 when tested in accordance with ASTM D-4318.

5) Fill or backfill, meeting the specifications outlined above, should be placed in 8-inch loose lifts and compacted with approved compaction equipment. Each lift should be firm and non-yielding.

6) Tests for degree of compaction should be determined by the ASTM D-1556 method or ASTM D-6938. Observation and field tests should be carried on during filling and backfill placement by the geotechnical engineer assist the contractor in obtaining the required degree of compaction. If less than 95 percent relative compaction is indicated, additional compaction effort should be made with adjustment of the moisture content as necessary until 95 percent compaction is obtained.

MOISTURE PROTECTION

Precautions should be taken during and after construction to minimize moisture increases of foundation soils or behind retaining walls. It is recommended that positive drainage be provided away from the abutments to prevent ponding adjacent to the foundations. Backfill should be well compacted and should meet the specifications outlined in the site grading section of this report.

FOUNDATION REVIEW AND INSPECTION

This report has been prepared to aid in the evaluation of this site and to assist in the design of this project. It is recommended that the geotechnical engineer be provided the opportunity to review the final design drawings and specifications in order to determine whether the recommendations in this report are applicable to the final design. Review of the final design drawings and

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specifications should be noted in writing by the geotechnical engineer.

Variations from soil conditions presented herein may be encountered during construction of this project. In order to permit correlation between the conditions encountered during construction and to confirm recommendations presented herein, it is recommended that the geotechnical engineer be retained to perform sufficient review during construction of this project. Observation and testing should be performed during construction to confirm that suitable fill soils are placed upon competent materials and properly compacted and foundation elements penetrate the recommended soils.

CLOSURE

Our conclusions, recommendations and opinions presented herein are:

(1) Based upon our evaluation and interpretation of the findings of the field and laboratory program.

(2) Based upon an interpolation of soil conditions between and beyond the explorations.

(3) Subject to confirmation of the conditions encountered during construction.

(4) Based upon the assumption that sufficient observation will be provided during construction.

(5) Prepared in accordance with generally accepted professional geotechnical engineering principles and practice.

This report has been prepared for the sole use of The Louis Berger Group, Inc., specifically for the design of the proposed De Fouri Street Bridge crossing the Santa Fe River in Santa Fe, New Mexico, and not for use by any third parties.

We make no other warranty, either express or implied. Any person using this report for bidding or construction purposes should perform such independent investigation as he deems necessary to satisfy himself as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on this project. If conditions are encountered during construction that appears to be different than indicated by this report, this office should be notified.

All soil samples will be discarded 60 days after the date of this report unless we receive a specific request to retain the samples for a longer period of time.

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BORING LOCATION MAP

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Defouri Street Bridge
Santa Fe, New Mexico
Job No. 1-20905

Figure 1

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GEOTECHNICAL ENGINEERING, ENVIRONMENTAL
MATERIAL TESTING
SANTA FE – ALBUQUERQUE – LAS CRUCES

GEO-TEST

Project: DeFouri Street Bridge
Date: 01/30/2013 Project No: 1-20905
Elevation: Type: 5" OD HSA

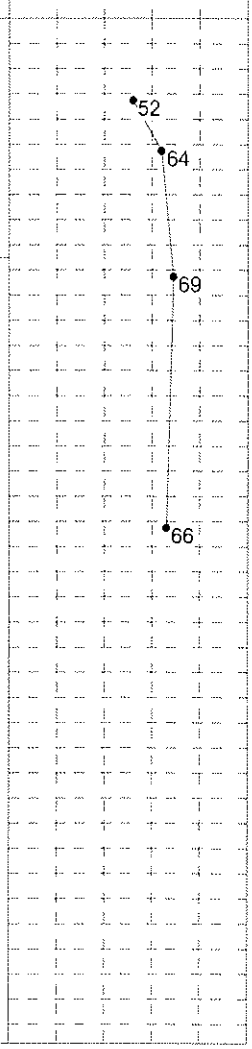
LOG OF TEST BORINGS

GROUNDWATER DEPTH

NO: 1

During Drilling: None After 24 Hours:

DEPTH (Ft)	LOG	SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
			SS	20-24-28 52	1		GP-GM	SAND AND GRAVEL, trace silt, very dense, slightly moist, brown to light brown (Probable abutment wall backfill) Intermittent Cobbles from 4.5' to 9.5'
5			SS	14-32-32 64	3			
10			SS	14-26-43 69	10		SP	CLAYEY SAND, some gravel, fine to medium grained, low to medium plasticity, very dense, slightly moist to moist, brown/reddish/light brown (Santa Fe Formation)
15			SS	24-50/3"	9			
20			SS	20-26-40 66	13			
25			SS	50/6"	10			
30			SS	50/3"	7			
35			SS	50/3"				
40			SS	50/3"	28			



LEGEND

SS - Split Spoon
AC - Auger Cuttings
UD/SL - Undisturbed Sleeve
AMSL - Above Mean Sea Level
CS - Continuous Sampler
UD - Undisturbed
ST - Shelby Tube

Stratification lines represent approximate boundaries between soil types. Transitions may be gradual. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to factors other than those present at the time measurments were made.

LOG OF TEST BORING 1-20905 GUADALUPE - DEFOURI ST. BRIDGES.GPJ GEO TEST.GDT 4/19/13

GEO-TEST

Project: DeFouri Street Bridge
Date: 01/30/2013 Project No: 1-20905
Elevation: Type: 5" OD HSA

LOG OF TEST BORINGS

GROUNDWATER DEPTH

NO: 1

During Drilling: None After 24 Hours:

DEPTH (Ft)	LOG	SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION	N blows/ft (20 40 60 80)
45			SS	30-50/4"	7				
50			SS	50/1"	6				
55			SS	50/6"	9		SP	CLAYEY SAND, some gravel, fine to medium grained, low to medium plasticity, very dense, slightly moist to moist, brown/reddish/light brown (Santa Fe Formation)	
60			SS	37-50/6"	6				
65			SS	36-50/4"	6				
70			SS	50/6"	7				
75								AUGER REFUSAL AT 72.5' SAMPLER REFUSAL AT 70'	
80									

LOG OF TEST BORING 1-20905 GUADALUPE - DEFOURI ST. BRIDGES.GPJ GEO TEST.GDT 4/19/13

LEGEND

SS - Split Spoon
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Project: DeFouri Street Bridge
Date: 01/21/2013 Project No: 1-20905
Elevation: Type: 5" OD HSA

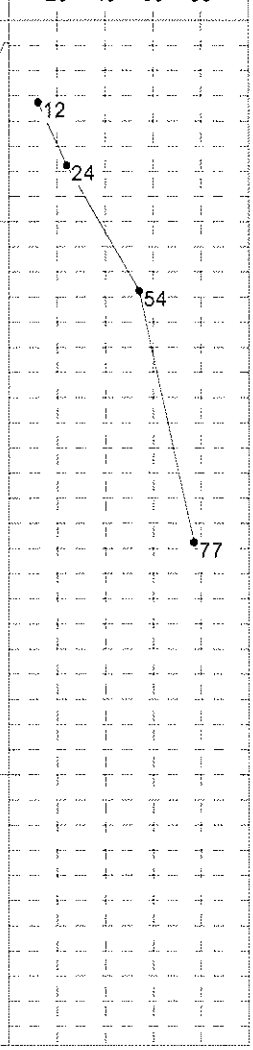
LOG OF TEST BORINGS

GROUNDWATER DEPTH

NO: 2

During Drilling: None After 24 Hours:

DEPTH (Ft)	SAMPLE INTERVAL	TYPE	N. BLOWS/FT	MOISTURE %	DRY DENSITY (pcf)	USC	DESCRIPTION
						AC	9" AC
5		SS	20-9-3 12	6		GP-GM	SAND WITH GRAVEL AND COBBLES, fine to coarse grained, some silt or clay, non-plastic to low plasticity, medium dense, slightly moist to moist, brown
		SS	12-11-13 24	6			
10		SS	45-29-25 54	2		SC-SM	CLAYEY AND/OR SILTY SAND WITH GRAVEL, fine to coarse grained, non-plastic to medium plasticity, very dense, slightly moist to moist, brown to red/brown (Santa Fe Formation)
15		SS	50/6"	9			
20		SS	25-39-38 77	10			
25		SS	32-50/3"	11			
30							STOPPED AUGER AT 30' STOPPED SAMPLER AT 25.5'
35							
40							



LOG OF TEST BORING 1-20905 GUADALUPE - DEFOURI ST. BRIDGES.GPJ GEO TEST.GDT 4/19/13

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AC - Auger Cuttings
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Stratification lines represent approximate boundaries between soil types. Transitions may be gradual. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to factors other than those present at the time measurments were made.

SUMMARY OF LABORATORY RESULTS

						SIEVE ANALYSIS PERCENT PASSING											
TEST HOLE	DEPTH (FEET)	UNIFIED CLASS	(%) MOIST	LL	PI	NO 200	NO 100	NO 40	NO 10	NO 4	3/8"	1/2"	3/4"	1"	1 1/2"	2"	4"
1	2.5	GP-GM	1.3	NP	NP	9	13	22	35	43	51	53	55	68	100		
1	4.5		3.2														
1	9.5	SC-SM	10.1	27	7	29	50	77	93	94	95	95	100				
1	14.5		9.0														
1	19.5		12.8														
1	24.5	SC	9.5	33	15	19	24	41	80	93	97	97	100				
1	29.5		7.1														
1	39.5		28.3			15	20	39	70	84	95	95	100				
1	44.5		7.4														
1	49.5		5.8														
1	54.5	SC	9.4	26	10	23	32	59	93	100							
1	59.5		5.5														
1	64.5		6.3														
1	69.5		7.1														
2	2.5		5.6														
2	5.0		5.9														
2	10.0	SW-SM	2.0	NP	NP	5	10	19	38	64	74	76	100				
2	15.0	SM	8.5	NP	NP	14	21	40	71	84	95	100					
2	20.0	SC	10.4	37	16	13	19	38	65	71	77	80	81	100			

GEO-TEST

LL = LIQUID LIMIT
PI = PLASTICITY INDEX
NP = NON PLASTIC or NO VALUE

Project: DeFouri Street Bridge
Location: Santa Fe, New Mexico
Number: 1-20905

SUMMARY OF LABORATORY RESULTS

						SIEVE ANALYSIS PERCENT PASSING											
TEST HOLE	DEPTH (FEET)	UNIFIED CLASS	(%) MOIST	LL	PI	NO 200	NO 100	NO 40	NO 10	NO 4	3/8"	1/2"	3/4"	1"	1 1/2"	2"	4"
2	25.0	SM	11.2	NP	NP	13	23	48	87	94	98	100					

GEO-TEST

LL = LIQUID LIMIT
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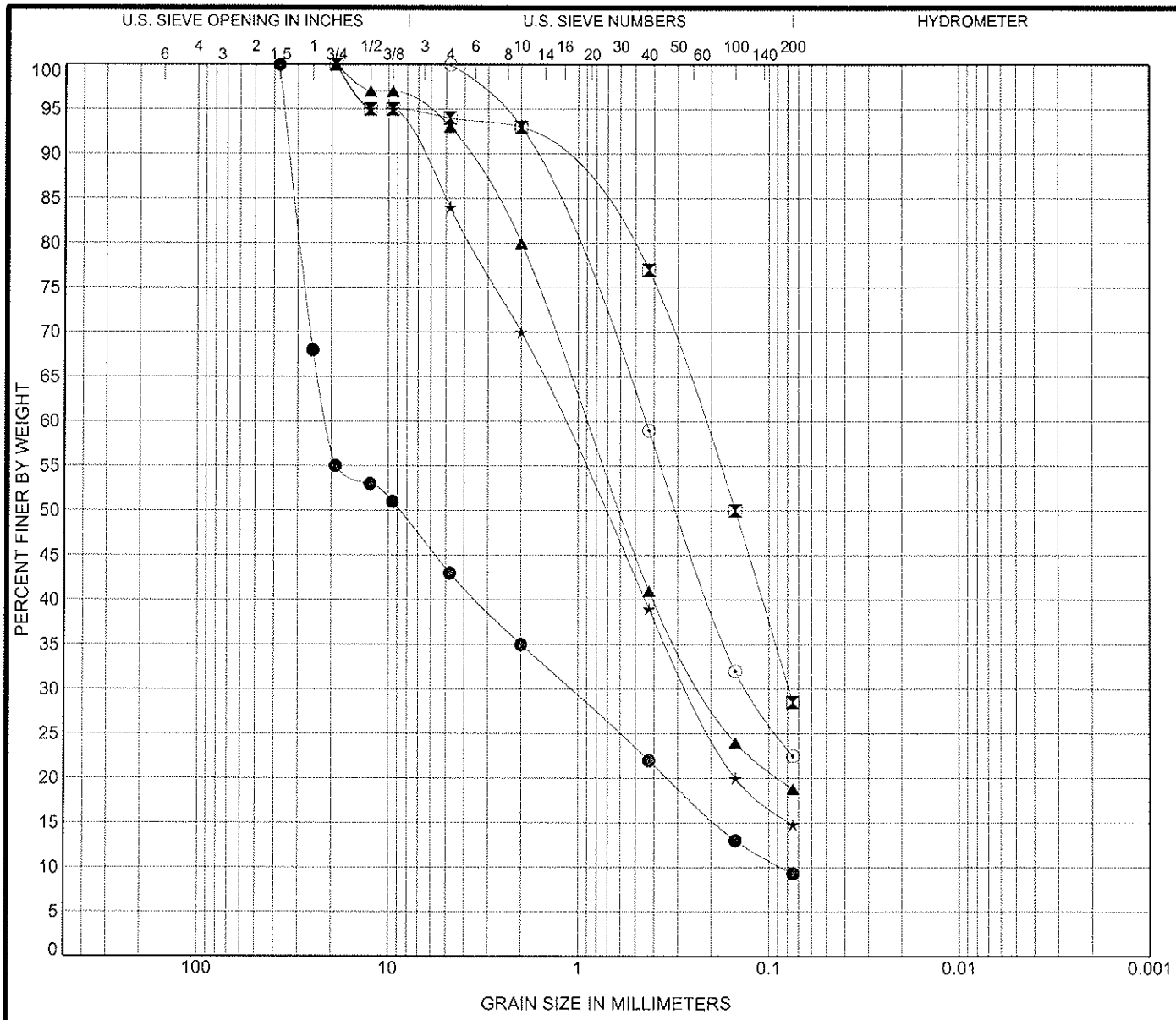
Project: DeFouri Street Bridge
Location: Santa Fe, New Mexico
Number: 1-20905



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	1	2.5	POORLY GRADED GRAVEL with SILT and SAND(GP-GM)	NP	NP	NP	0.67	246.93
⧗	1	9.5	SILTY, CLAYEY SAND(SC-SM)	27	20	7		
▲	1	24.5	CLAYEY SAND(SC)	33	18	15		
★	1	39.5						
⊙	1	54.5	CLAYEY SAND(SC)	26	16	10		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	1	2.5	37.5	21.115	1.102	0.086	57.0	33.7	9.3	
⧗	1	9.5	19	0.221	0.079		6.0	65.5	28.5	
▲	1	24.5	19	0.904	0.217		7.0	74.2	18.8	
★	1	39.5	19	1.214	0.26		16.0	69.2	14.8	
⊙	1	54.5	4.75	0.445	0.13		0.0	77.5	22.5	

GEO-TEST

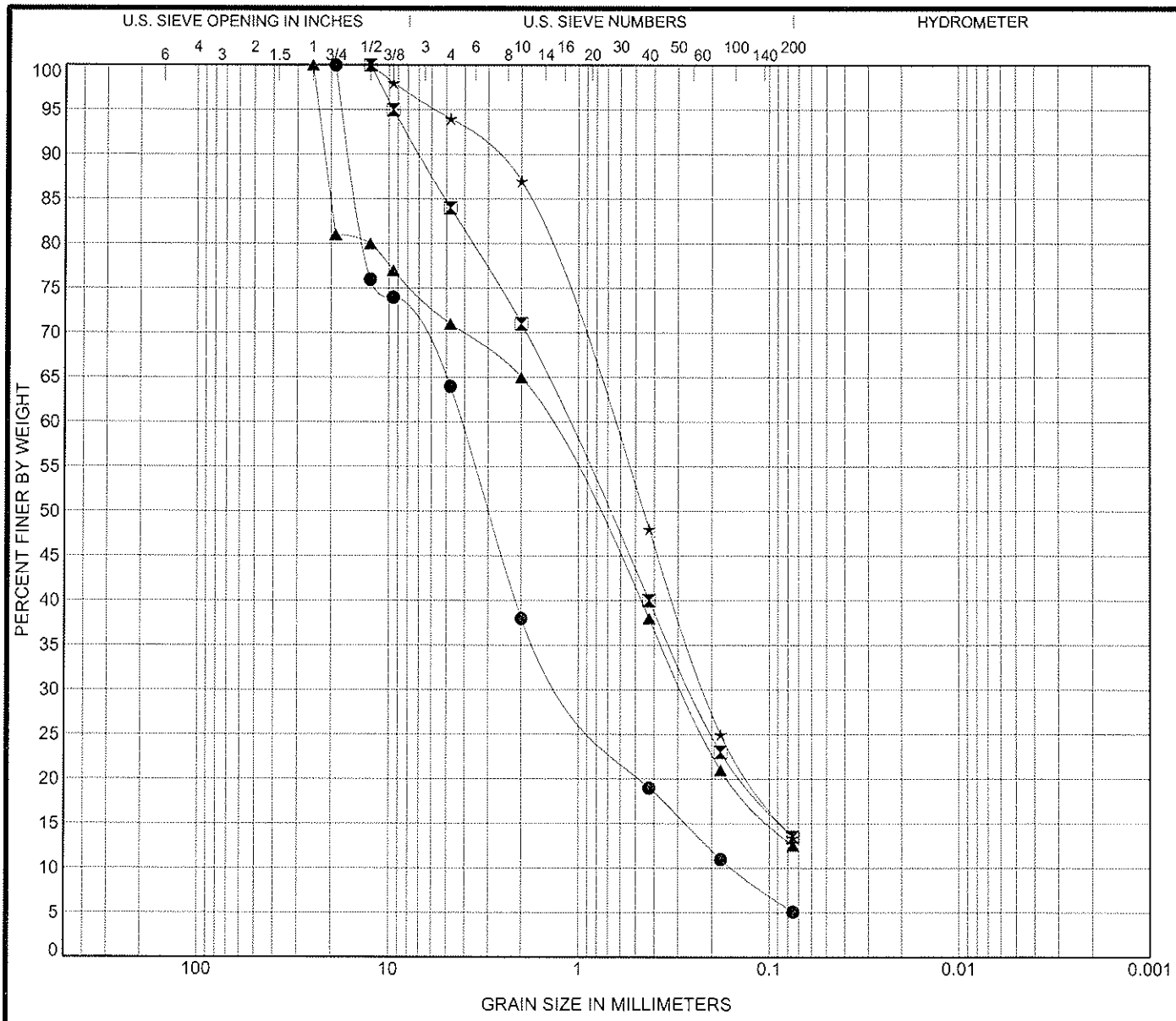
GRAIN SIZE DISTRIBUTION

Project: DeFouri Street Bridge

Location: Santa Fe, New Mexico

Number: 1-20905

US GRAIN SIZE 1-20905 GUADALUPE - DEFOURI ST. BRIDGES.GPJ GEO TEST.GDT 4/19/13



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	2	10.0	**WELL-GRADED SAND with SILT and GRAVEL(SW-SM)**	**NP**	**NP**	**NP**	**1.68**	**26.80**
⧗	2	15.0	**SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**		
▲	2	20.0	**CLAYEY SAND with GRAVEL(SC)**	**37**	**21**	**16**		
★	2	25.0	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	2	10.0	**19**	**4.158**	**1.042**	**0.155**	**36.0**	**58.9**	**5.1**	
⧗	2	15.0	**12.5**	**1.154**	**0.256**		**16.0**	**70.5**	**13.5**	
▲	2	20.0	**25**	**1.501**	**0.284**		**29.0**	**58.5**	**12.5**	
★	2	25.0	**12.5**	**0.684**	**0.217**		**6.0**	**80.6**	**13.4**	

GEO-TEST

GRAIN SIZE DISTRIBUTION

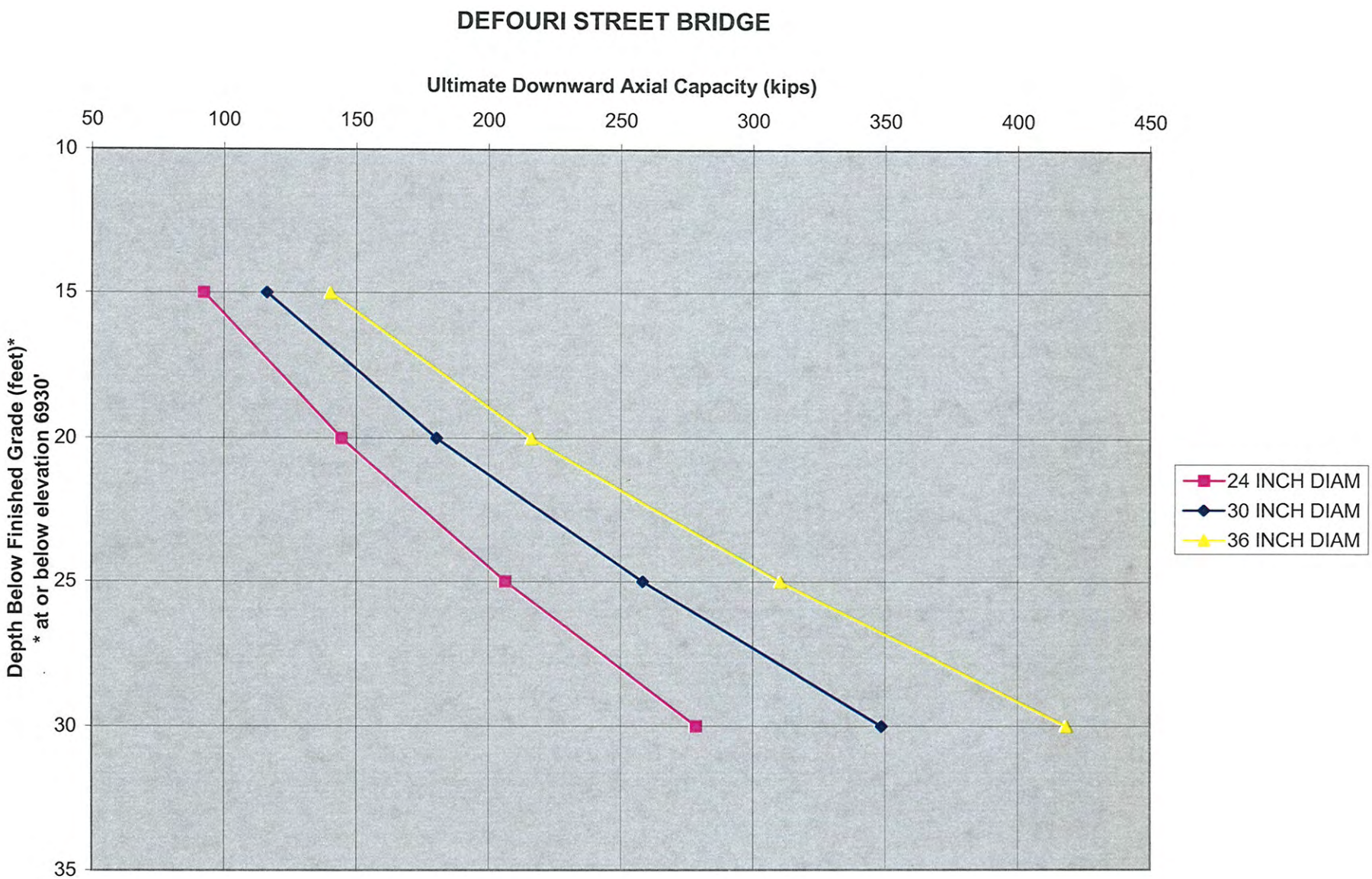
Project: DeFouri Street Bridge

Location: Santa Fe, New Mexico

Number: 1-20905

US GRAIN SIZE 1-20905 GUADALUPE - DEFOURI ST. BRIDGES.GPJ GEO TEST.GDT 4/19/13

APPENDIX A


DEFOURI STREET BRIDGE
Ultimate Downward Axial Capacity (kips)
50
100
150
200
250
300
350
400
450
10
15
20
25
30
35
Depth Below Finished Grade (feet)*
* at or below elevation 6930'
24 INCH DIAM
30 INCH DIAM
36 INCH DIAM

GEO-TEST

May 2, 2013
Job No. 1-20905

The Louis Berger Group, Inc.
2019 Galisteo St., Suite M-1
Santa Fe, New Mexico 87505

Attn: Mr. Richard K. Rotto, P.E.

RE: Geotechnical Engineering Services
De Fouri Street Bridge
Santa Fe, New Mexico

Dear Mr. Rotto:

Submitted below are the LRFD soil parameters as requested.

LRFD SOIL PARAMETERS	
SOIL PARAMETER	**VALUE**
Backfill friction angle	33 deg
Backfill dry unit weight	125 pcf
Backfill saturated unit weight	135 pcf
Minimum equivalent fluid pressure (active)	35 pcf
Allowable settlement	1 inch
Bearing resistance factor	0.45
Sliding resistance factor	0.8
Non-saturated unit weight	120 pcf
Saturated unit weight	130pcf
Elastic modulus of existing soil	1,700 ksf
Effective friction angle of existing soil	36 deg below 10 feet
Cohesion	500 psf below 10 feet

GEO-TEST, INC.
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FAX (505) 471-2245

8528 CALLE ALAMEDA NE
ALBUQUERQUE,
NEW MEXICO
87113
(505) 857-0933
FAX (505) 857-0803

2805-A LAS VEGAS CT.
LAS CRUCES,
NEW MEXICO
88007
(575) 526-6260
FAX (575) 523-1660

It has been a pleasure to serve you on this project. If you should have any questions, please contact this office.

Respectfully submitted:
GEO-TEST, INC.

Charles M. Miller, P.E.

CHARLES M. MILLER
NEW MEXICO
6018
LICENSED PROFESSIONAL ENGINEER

Reviewed by:

Robert D. Booth, P.E.

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