GEOTECHNICAL ENGINEERING REPORT
CHICO WAY BRIDGE REPLACEMENT
3000 CHICO WAY NW
CHICO, WASHINGTON

PROJECT NO. 81075880
MARCH 12, 2008

Prepared for:

SARGENT ENGINEERS, INC.
320 RONLEE LANE NW
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Prepared by:

ZZA-Terracon

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Tacoma, Washington 98409

March 12, 2008

Sargent Engineers, Inc.
320 Ronlee Lane NW
Olympia, Washington 98502

Attention: Mr. Erik C. Martin

Subject: Geotechnical Engineering Report
Chico Way Bridge Replacement
3000 Chico Way NW
Chico, Washington
ZZA-Terracon Project No.: 81075880

Dear Mr. Martin,

The enclosed report contains our findings from field exploration, laboratory testing and geotechnical engineering analyses for the referenced project site. These services were completed in general accordance with the scope of work in the Agreement dated December 6, 2007. The information in this report is based on our understanding of the proposed construction, and the soil and groundwater conditions encountered in borings completed at the site on December 11, 2007.

We recommend that the project plans and specifications be submitted to ZZA-Terracon for a general review to confirm that the recommendations in this report are interpreted and implemented properly in the construction documents. We recommend that a representative from our firm be present during portions of project construction to confirm that the soil and groundwater conditions are consistent with those that form the basis for the engineering recommendations in this report.

We appreciate the opportunity to provide these services. Please call if you have any questions.

Respectfully Submitted,

ZZA-Terracon

James B. Thompson, P.E.
Principal

JAMES B. THOMPSON
STATE OF WASHINGTON
19409
REGISTERED
PROFESSIONAL ENGINEER

3/12/08

EXPIRES 04 / 03 / 10

TABLE OF CONTENTS

INTRODUCTION 1
PROJECT DESCRIPTION 1
SURFACE CONDITIONS 1
SUBSURFACE CONDITIONS 2
Field Exploration & Laboratory Testing 2
Soil Conditions 2
Water Level Observations 2
CONCLUSIONS AND RECOMMENDATIONS 2
General 2
Seismic Considerations 3
Approach Embankments 3
Bridge Foundations 4
Retaining Walls for North Approach 6
GENERAL COMMENTS 7

Enclosures

Appendix A - Field Exploration Procedures and Logs

Appendix B – Laboratory Testing Procedures and Results

Appendix C – General Notes
Unified Soil Classification System

GEOTECHNICAL ENGINEERING REPORT

PROPOSED CHICO WAY BRIDGE REPLACEMENT
CHICO, WASHINGTON

ZZA-Terracon Project No. 81075880
March 12, 2008

INTRODUCTION

This report summarizes the results of our geotechnical evaluation for the proposed Chico Way Bridge Replacement at 3000 Chico Way NW in Chico, Washington. These services were completed in general accordance with the scope of work described in the Agreement dated December 6, 2007. Our scope of services included field explorations, laboratory testing, geotechnical engineering analyses, and preparation of this report.

PROJECT DESCRIPTION

We understand that the project plans include demolition of the existing Chico Way NW bridge crossing of Chico Creek that was damaged in the December 2007 floods, and construction of a new two lane bridge. We anticipate that the new bridge will be constructed at approximately the same location and elevation as the existing bridge, expect that the new bridge will be longer.

The new bridge will be a single-span structure supported on pile foundations. The span length and foundation loads for the new bridge are not currently available. The structure will be designed in accordance with the AASHTO LRFD Bridge Design Specifications, 4th Edition, with Interims, and the current WSDOT Bridge Design Manual.

SURFACE CONDITIONS

The project site is located in a residential area at 3000 Chico Way NW in Chico, Washington. The existing bridge was damaged during the December 2007 storms when Chico Creek undercut the slope at the north abutment, resulting in failure of the north abutment and collapse into the creek.

South of the bridge, the creek flows in a northerly direction parallel to Chico Way NW. The creek turns to the east where it passes beneath the bridge. North of the bridge, the creek flows in a northeasterly direction. Existing armoring was observed on both banks of the creek during our site visit. However, most of the armoring in the vicinity of the north abutment had been removed during the flooding.

A water line had been supported on the east side of the existing bridge. This water line was damaged when the bridge collapsed.

SUBSURFACE CONDITIONS

Field Exploration & Laboratory Testing

Subsurface conditions at the project site were evaluated by completing two borings on December 11, 2007 using a trailer-mounted hollow stem auger drill rig. The approximate locations of the borings are shown on Figure 1- Site and Exploration Plan. The field exploration procedures and logs of the borings are presented in Appendix A.

Soil samples were collected from the borings and placed in sealed containers for further examination and laboratory testing. Selected samples tested for moisture content and grain size analysis. The laboratory testing procedures and results are presented in Appendix B.

Soil Conditions

The borings encountered 4 to 6 inches of asphalt underlain by 6 to 12 inches of gravel base course. The pavement sections were underlain by deposits of probable fill to depths of 12.5 and 10 feet in Borings B-1 and B-2, respectively. In Boring B-1, the probable fill consisted of stiff silt and sandy silt with gravel. In Boring B-2, the probable fill consisted of very stiff silt with gravel and very dense silty gravel.

Native deposits of very dense sandy gravel and gravelly sand with varying amounts of silt were encountered below the probable fill and to the maximum depth explored (Elevation 49 feet). Observations of the drill cuttings and action of the drill rig indicated that cobbles and also possibly boulders are present in the native sandy gravel and gravelly sand deposits.

Water Level Observations

Groundwater was observed in Borings B-1 and B-2 at depths of 26 and 16 feet, respectively, at the time of drilling. Site groundwater levels are generally expected to be at or close to creek levels.

CONCLUSIONS AND RECOMMENDATIONS

General

The project site is suitable for construction of the proposed bridge replacement. The following sections present conclusions and recommendations for seismic consideration, approach embankments, bridge foundations, and retaining walls for north approach.

Seismic Considerations

We understand that the Chico Way Bridge will be designed based on a risk level of 10 percent probability of exceedance in 50 years (an approximately 475 year recurrence interval).

Figure 6-5 of the Geotechnical Design Manual (M46-03) indicates that the peak ground acceleration on bedrock at the project site is expected to be 0.35 g for this risk level.

Based on the descriptions presented in Section 3.10.5 of AASHTO, the site is best classified as Soil Profile Type II with a Site Coefficient, S, of 1.2 per Table 3.10.5.1-1. Soil Profile Type II is a profile with stiff cohesive or deep cohensionless soils where the soil depth exceeds 200 feet and the soil types overlying the rock are stable deposits of sands, gravel, or stiff clays.

The liquefaction potential of the site is expected to be low based on the results of the test borings However, it should be recognized that the existing fill has a variable composition and that zones of potentially liquefiable soils could be present within the fill that were not detected by the borings. The primary impact of liquefaction of the existing fill would be bank failure during a seismic event. If a bank failure were to occur, impacts on the pile foundations for the new bridge are expected to be negligible.

Approach Embankments

It will be necessary to reconstruct the stream banks, particularly the eroded north bank and areas where the banks will be disturbed by new construction. We understand that stream bank slopes of 1¾:1(horizontal: vertical) are currently planned. In our opinion, this inclination can be used for the stream banks providing that the slopes are reconstructed using quarry spalls (Section 9-13.6) or riprap (Section 9-13.1). Rubble from demolition of the existing bridge could be considered providing that the rubble meets the criteria for quarry spalls or riprap.

All loose or disturbed material should be removed from the stream banks as part of the reconstruction. We recommend that a representative from our firm observe the exposed subgrade conditions before placing the quarry spalls or riprap.

Scour protection should be provided for the stream banks in the vicinity of the new bridge.

Bridge Foundations

General

We understand that the new bridge will be supported on deep foundations to mitigate the potential concern for scour. The deep foundations should extend through the existing fill and penetrate into the underlying native, very dense gravelly sand and sandy gravel deposits.

Drilled shafts have been considered as one possible deep foundation option, but were not selected for two primary reasons. First, the cobbles and boulders present in the gravelly sand and sandy gravel might result in difficult drilling conditions. Second, the gravelly sand and sandy gravel are not expected to be stable in an open hole, particularly below the water table, and casing or drilling mud would likely be required for drilled shaft construction. These two factors would have significant impacts on the production rates and costs for drilled shafts.

Three different types of driven piles have been considered: precast concrete, steel pipe piles, and steel H-piles. It is likely that driven piles will encounter hard driving due to the presence of cobbles and boulders in the gravelly sand and sandy gravel. Precast concrete piles are not recommended for this project because of the potential for pile damage during driving. Steel pipe piles are also not recommended because of the potential for pile damage. In addition, steel pipe piles might have difficulty achieving adequate penetration into the gravelly sand and sandy gravel.

Driven H-piles have been selected as the preferred foundation type for the new bridge. We understand that 12 inch H-piles are currently being considered. We recommend that 12-inch H-piles have a minimum center to center spacing of 3 feet. The following report sections provide design and construction recommendations for 12 inch H-piles.

Downdrag

We understand that site grades will not be raised at the abutments. As mentioned previously, the liquefaction potential of the site soils located below the water table is considered to be low and the potential seismic induced ground subsidence is expected to be small. As a result, it is our opinion that pile downdrag forces need not be considered in design.

Axial Capacity

The strength limit state axial capacity of 12-inch H-piles in compression was analyzed in accordance with the Nordlund/Thurman Method in Cohesionless Soils as described in

Section 10.7.3.8.6f of the AASHTO LRFD Bridge Design Specifications, 4th Edition, with Interims. A resistance factor of 0.45 was applied to the ultimate axial capacity as described in Table 10.5.5.2.3-1 of AASHTO. Plots of the total factored axial capacity of 12 inch H-piles as a function of pile tip elevation for both the south and north abutments are presented in Figure 2.

The H-piles will penetrate into very dense gravelly sand and sandy gravel. The service limit state capacity of the H-piles is not expected to be a controlling factor in pile design. Consequently, the service limit state capacity of the H-piles has not been evaluated consistent with Section C10.7.2.3.1 of AASHTO.

Lateral Capacity

We understand that the computer program Lpile will be used to analyze the response of the piles under lateral loads. This computer program has the capability of internally generating the p-y curves for the soils adjacent to the piles. Six different soil models are available in the Lpile program to compute the p-y cures. We recommend that the Sand of Reese, Cox, and Koop model be used to generate p-y curves for the site soils.

The soil input parameters needed to generate p-y curves for the Sand of Reese, Cox and Koop model include the total unit weight, friction angle, and modulus of subgrade reaction of the soils. Recommended values for these soil input parameters are presented in Table 1(Sheets 1 and 2) for the south and north abutments, as a function of elevation and analysis type (static or dynamic).

Pile group effects should be considered when analyzing the response of piles under lateral loads.

Minimum Penetration

The H-piles should have adequate length to extend through the existing fill and penetrate into the underlying native, very dense sandy gravel and gravelly sand deposits. In addition, the length of the H-piles should be adequate to achieve fixity at the pile tip. We recommend that the H-piles have a minimum penetration of 10 feet into the native, very dense sandy gravel and gravelly sand. The minimum pile penetration required for fixity at the pile tip should be analyzed with the Lpile computer program

Construction Considerations

Obstructions to pile driving might be encountered in the existing fill. The contractor should be prepared to excavate and remove obstructions from the existing fill, or to use predrilling, as necessary to accommodate pile installation...

Cobbles and boulders within the native, gravelly sand and sandy gravel deposits might also represent obstructions to pile driving. We recommend that the project plans and specifications include provisions for the installation of replacement piles, as required, if some of the piles encounter refusal on obstructions in the gravelly sand and sandy gravel at elevations above the required minimum penetration.

The installation of H-piles should be observed by a representative our firm to determine if the subsurface conditions are as expected and to confirm pile capacities.

Retaining Walls for North Approach

We understand that conventional, cast-in-place concrete retaining walls are planned for the north approach. These retaining walls will extend back from the abutments along both sides of Chico Way NW.

Boring B-2, drilled in the vicinity of the north abutment, encountered 10 feet of fill. The fill consisted of 5 feet of very stiff silt with gravel underlain by 5 feet of very dense silty gravel. The underlying native soils consisted of very dense sandy gravel and gravelly sand with varying amounts of silt.

It appears that the new retaining walls can be supported on conventional spread footings. Based on the results of Boring B-2, we recommend that the spread footings for the retaining walls be founded on the very dense silty gravel fill encountered in Boring B-2 at a depth of 5 feet below the existing ground surface, or on a zone of structural fill which replaces the overlying very stiff silt fill and extends down to the very dense silty gravel fill. If required, the zone of structural fill should extend a minimum horizontal distance beyond the edges of the footing equal to the thickness of the zone of structural fill.

It is important to note that the composition and consistency of fill can vary significantly over a short distance depending on the history of fill placement. Consequently, we strongly recommend that the condition of the zones of existing fill located beneath the retaining wall footings be evaluated during construction to confirm that the existing fill is suitable for footing support.

We recommend that the spread footings for the cast-in-place concrete retaining walls have a minimum depth of embedment of 18 inches below lowest adjacent finished grade. The footings should have a minimum width of 2 feet. The service limit state bearing resistance q_{serv} was evaluated in accordance with AASHTO Section 10.6.2.4.2 for an assumed allowable total settlement of 1.0 inch. The unfactored strength limit state bearing resistance q_n of the spread footings was evaluated in accordance with AASHTO Section 10.6.3.1.2. The results are presented in Figure 3 as plots of bearing resistance versus effective footing width, B'.

We recommend that the following soil parameters and resistance factors be used for retaining wall design. The walls were assumed to be free to move during seismic loading and 0.5PGA was used to determine Kae.

Parameter	Value
Soil Unit Weight, γ ((soil above wall footing base level)	125
Soil Friction Angle, Φ (soil above wall footing base level)	32
Active Earth Pressure Coefficient, K_a	0.31
Passive Earth Pressure Coefficient, K_p	3.0
Seismic Earth Pressure Coefficient, K_{ae}	0.42
Coefficient of Sliding, tan δ	0.58

	Resistance Factor, φ		
Limit State	**Bearing**	**Shear Resistance to Sliding**	**Passive Pressure Resistance to Sliding**
Strength	0.5	0.8	0.5
Service	1.0	1.0	1.0
Extreme Event	1.0	1.0	1.0

GENERAL COMMENTS

ZZA-Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. ZZA also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the explorations performed at the indicated locations. This report does not reflect variations that may occur between explorations, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become

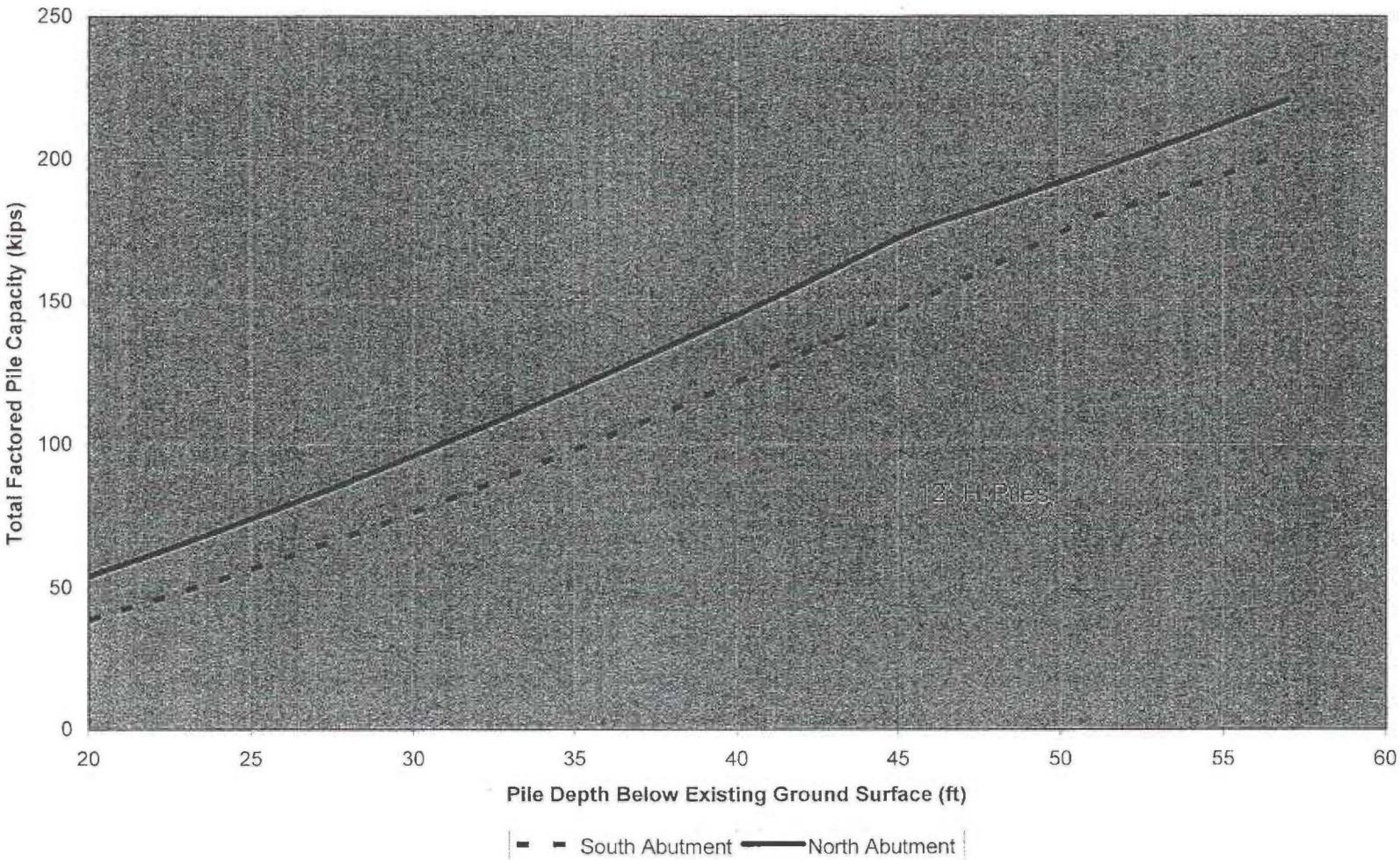
evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for the project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of Sargent Engineers, Inc. for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices in this area at the time the report was prepared. No warranties, either expressed or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless ZZA-Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

Figure 2

Strength Limit State Axial Pile Capacity in Compression



Notes: 1) Strength limit state axial pile capacity in compression computed using the Nordlund/Thurman Method
2) Resistance factor = 0.45

Figure 3
Bearing Resistance q_n or q_{serv}

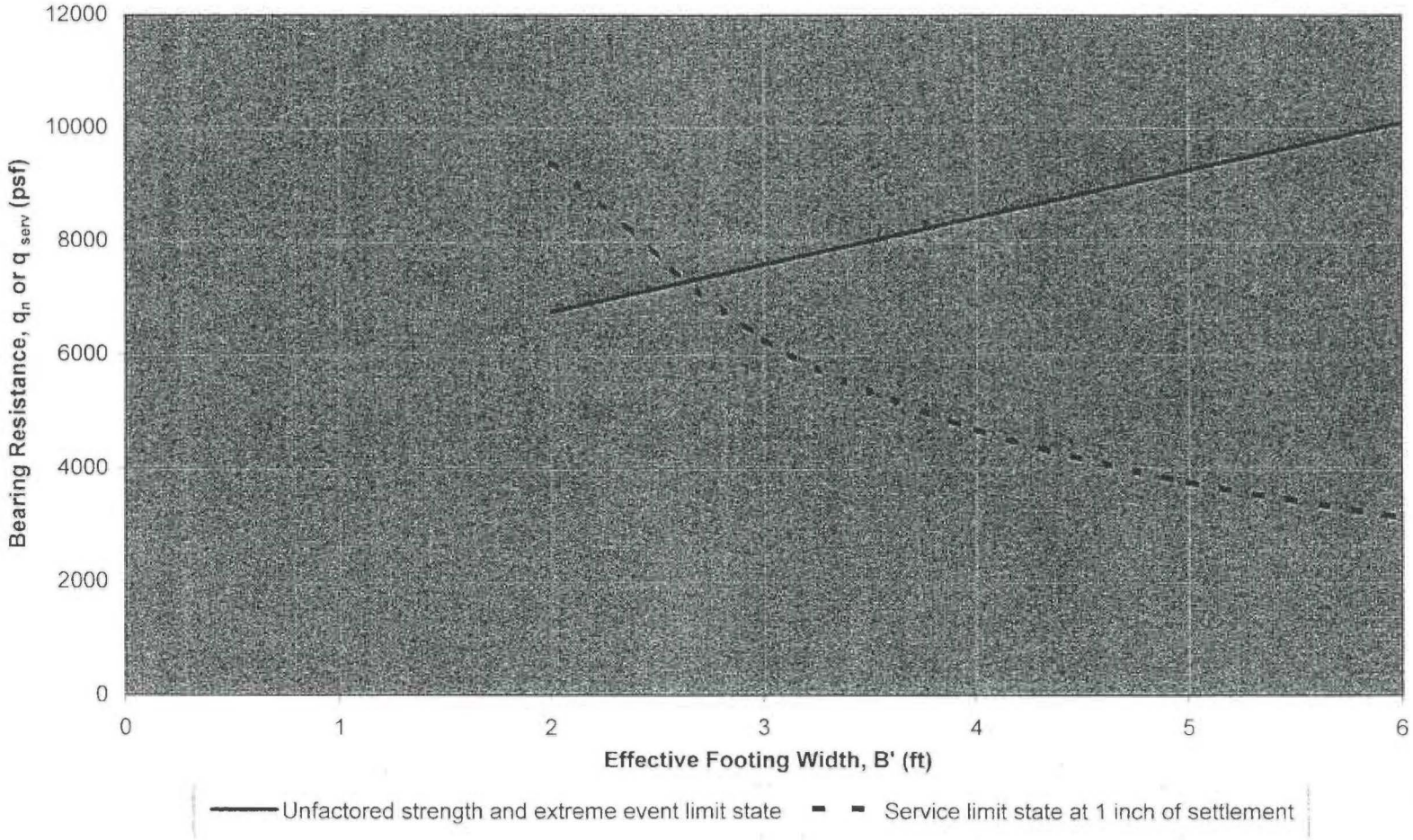


TABLE 1
LATERAL SOIL-PILE INTERACTION
P-Y CURVE SOIL PARAMETERS

Sheet 1 of 2
South Abutment

Soil Layer	Elevation at Base of Layer (ft)	Analysis Type	Effective Unit Weight (pcf)	Friction Angle, φ (degrees)	Modulus of Subgrade Reaction, *k* (pci)
1	63.5	Static Dynamic	105 105	30 30	40 40
2	58.5	Static Dynamic	53 53	32 32	70 70
3	Tip	Static Dynamic	68 68	36 36	130 130

TABLE 1
LATERAL SOIL-PILE INTERACTION
P-Y CURVE SOIL PARAMETERS

Sheet 2 of 2
North Abutment

Soil Layer	Elevation at Base of Layer (ft)	Analysis Type	Effective Unit Weight (pcf)	Friction Angle, φ (degrees)	Modulus of Subgrade Reaction, k (pci)
1	63	Static	110	30	50
		Dynamic	110	30	50
2	58	Static	115	32	70
		Dynamic	115	32	70
3	Tip	Static	68	36	130
		Dynamic	68	36	130

APPENDIX A
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FIELD EXPLORATION PROCEDURES AND LOGS

Our field exploration for the project included two geotechnical borings completed on December 11, 2007. The approximate exploration locations are shown on Figure 1, Site and Exploration Plan. Exploration locations were determined by measuring distances from existing site features by pacing or taping. As such, the exploration locations should be considered accurate only to the degree implied by the measurement method. The following sections describe our procedures associated with the explorations. Descriptive logs of the explorations are enclosed in this appendix.

The exploratory borings were advanced with a hollow stem auger using a trailer-mounted drill rig operated by an independent drilling company working under subcontract to ZZA. An engineering geologist from our firm continuously observed the borings, logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further visual classification and testing. Samples were obtained by means of the Standard Penetration Test throughout the drilling operation.

The Standard Penetration Test (ASTM: D-1586) procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows is struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring, as well as laboratory tests performed on these soil samples. If groundwater was encountered in a borehole, the approximate groundwater depth, and date of observation, are depicted on the log.

LOG OF BORING NO. B-1

CLIENT: Sargent Engineers, Inc.

SITE: 3000 Chico Way NW, Chico, WA

PROJECT: Chico Way Bridge Replacement

Boring Location: South Side

Approx. Surface Elev.: 76 ft

DEPTH, ft.	DESCRIPTION	USCS SYMBOL	SAMPLES NUMBER	TYPE	RECOVERY, in.	SPT - N * ** BLOWS / ft.	TESTS WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED STRENGTH, psf
0.5	Asphalt (75.5)			HS					
1	**SILTY SANDY GRAVEL,** Brown, Dense, Damp (Base Course) (76)								
	LEAN SILT, Mottled Tan/Rust, Stiff, Moist (Probable Fill)		1	SS	12	14			
5 (71)	**SANDY SILT, WITH GRAVEL,** Rust, Stiff, Damp (Probable Fill) Driller noted Gravels			HS					
			2	SS	12	11	10		
12.5 (63.5)	**GRAVELLY SAND, TRACE SILT,** Tan, Medium Dense, Damp			HS					
			3	SS	6	12			
	Cobbles in cuttings			HS					
17.5 (58.5)	**SANDY GRAVEL, WITH SILT** Brown, Very Dense, Moist		4	SS	10	88	6		
				HS					
	Grades Wet		5	SS	8	50/5"			
				HS					

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

*SPT-N Blows might be overstated due to presence of gravel and cobbles
**CME 140H SPT hammer

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 26	WD	▼
WL	▽		▽
WL			

ZZA-Terracon

BORING STARTED	12-11-07		
BORING COMPLETED	12-11-07		
RIG Deep Rock XL		CO.	Geologic Drill
LOGGED	JGE	JOB #	81075880

LOG OF BORING NO. B-1

CLIENT: Sargent Engineers, Inc.

SITE: 3000 Chico Way NW, Chico, WA

PROJECT: Chico Way Bridge Replacement

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	TYPE	RECOVERY, in.	SPT - N * ** BLOWS / ft.	TESTS WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED STRENGTH, psf
27.5 / 48.5	**GRAVEL, WITH SAND AND SILT,** Brown, Very Dense, Wet			6	SS	2	50/4"			
					HS					
		30								
32.5 / 43.5	**GRAVELLY SAND, TRACE SILT,** Brown, Very Dense, Wet			7	SS	12	69			
					HS					
		35								
				8	SS	12	89	8		
					HS					
		40								
42.5 / 33.5	**SANDY GRAVEL, WITH SILT,** Brown, Very Dense, Wet (Smaller Gravels)			9	SS	6	86			
					HS					
		45								
				10	SS	8	66			
					HS					
		50								
52.5 / 23.5				11	SS	0	50/1"			
	BOTTOM OF BORING									

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

*SPT-N Blows might be overstated due to presence of gravel and cobbles
**CME 140H SPT hammer

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 26	WD	▼
WL	▼		▽
WL			

ZZA-Terracon

BORING STARTED	12-11-07		
BORING COMPLETED	12-11-07		
RIG Deep Rock XL		CO.	Geologic Drill
LOGGED	JGE	JOB #	81075880

LOG OF BORING NO. B-2

CLIENT
Sargent Engineers, Inc.

SITE **3000 Chico Way NW**
Chico, WA

PROJECT
Chico Way Bridge Replacement

Boring Location: North Side

Approx. Surface Elev.: 68 ft

DESCRIPTION	Elev.	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	TYPE	RECOVERY, in.	SPT - N * ** BLOWS / ft.	TESTS WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED STRENGTH, psf
4" Asphalt	0.5 / 67.5				HS					
SILTY SANDY GRAVEL, Brown, Dense, Damp (Base Course)	1.5 / 66.5									
SILT, WITH GRAVEL, Brown, Very Stiff, Moist (Probable Fill)				1	SS	6	23			
					HS					
SILTY GRAVEL, Brown, Very Dense, Moist (Probable Fill)	5 / 63	5								
				2	SS	0	50/0"			
					HS					
Driller Noted Cobbles	10 / 58	10								
SILTY GRAVEL, Brown, Very Dense, Moist				3	SS	3	50/3"			
					HS					
		15								
				4	SS	0	50/5"			
					HS					
SANDY GRAVEL, TRACE SILT Brown, Very Dense, Wet	20 / 48	20		5	SS	8	58			
					HS					
				6	SS	0	50/4"			
					HS					
		25								

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

*SPT-N Blows might be overstated due to presence of gravel and cobbles
**CME 140H SPT hammer

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 16	WD	▼
WL	▼		▼
WL			

ZZA-Terracon

BORING STARTED	12-11-07
BORING COMPLETED	12-11-07
RIG Deep Rock XL	CO. Geologic Drill
LOGGED JGE	JOB # 81075880

BOREHOLE ZZA 81075880 DRAFT LOGS.GPJ TERRACON.GDT 2/19/08

LOG OF BORING NO. B-2

CLIENT: Sargent Engineers, Inc.

SITE: 3000 Chico Way NW, Chico, WA

PROJECT: Chico Way Bridge Replacement

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	TYPE	RECOVERY, in.	SPT - N * ** BLOWS / ft.	TESTS WATER CONTENT, %	DRY UNIT WT pcf	UNCONFINED STRENGTH, psf
	SANDY GRAVEL, TRACE SILT Brown, Very Dense, Wet			7	SS	18	47	9		
					HS					
		30								
				8	SS	12	37			
					HS					
		35								
				9	SS	12	59			
					HS					
		40								
				10	SS	4	50/5"			
					HS					
		45								
				11	SS	18	59			
	49	19								
	BOTTOM OF BORING									

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

*SPT-N Blows might be overstated due to presence of gravel and cobbles
**CME 140H SPT hammer

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 16	WD	▼
WL	▼		▼
WL			

ZZA-Terracon

BORING STARTED	12-11-07		
BORING COMPLETED	12-11-07		
RIG Deep Rock XL	CO. Geologic Drill		
LOGGED JGE	JOB # 81075880		

APPENDIX B
81075880

LABORATORY TESTING PROCEDURES AND RESULTS

A series of laboratory tests were performed during the course of the study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of tests performed are given below.

Visual Classification

Samples recovered from the explorations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with the Unified Soil Classification System (Appendix C). Visual soil classification includes evaluation of color, relative moisture content, soil type based upon grain size, and accessory soil types included in the sample. Soil classifications are presented on the exploration logs in Appendix A.

Moisture Content Determinations

Moisture content determinations were performed on representative samples obtained from the explorations in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM D 2216.

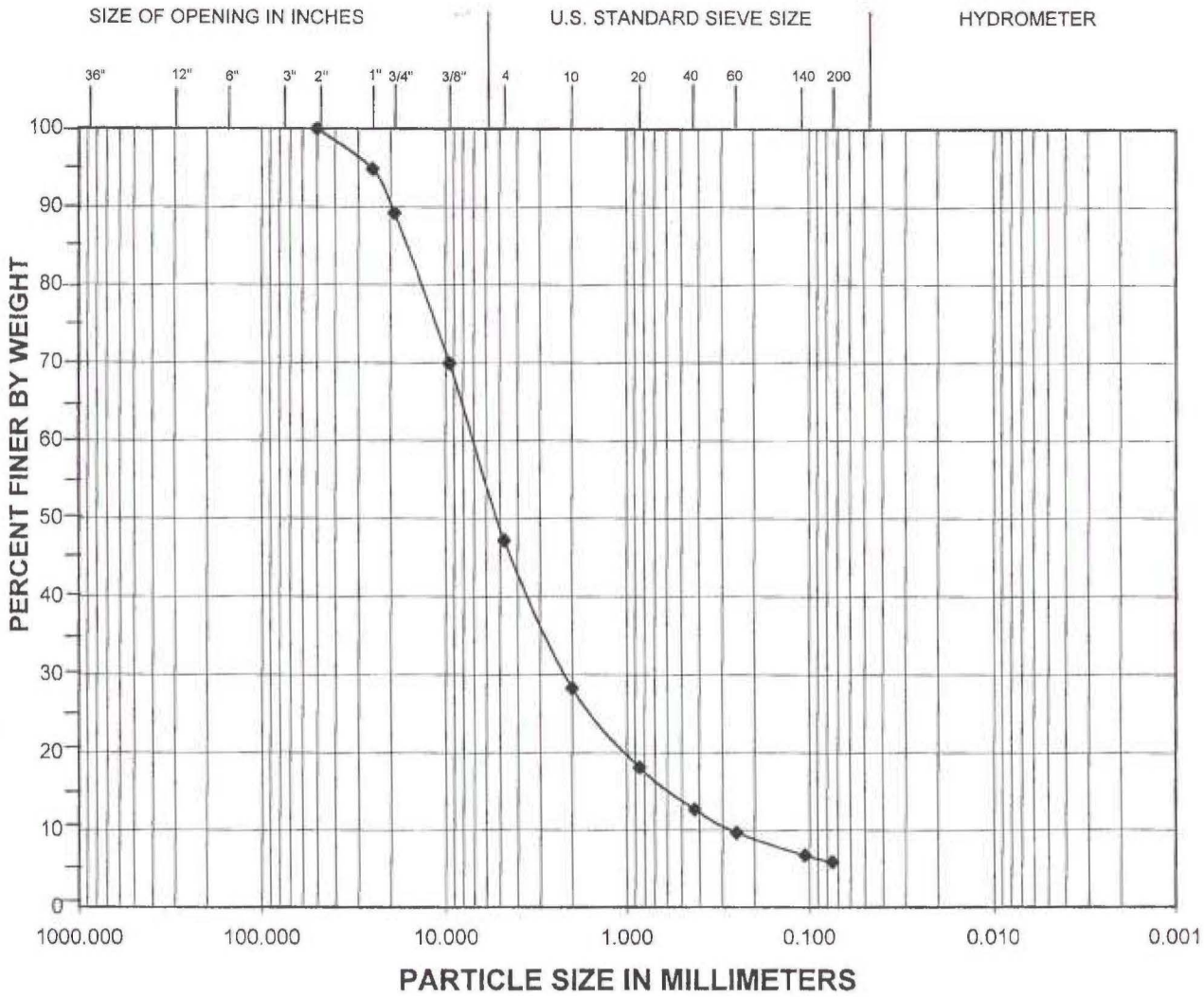
Grain Size Analysis

A grain size analysis indicates the range in diameter of soil particles included in a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM D 422. The results of the grain size determinations were used in classification of the soils, and are presented in this appendix.

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D 422



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-4	17.5	6	5.9	sandy GRAVEL with silt

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Geotechnical and Environmental Consulting
2115 S.56th Street, #405 Tacoma, WA 98409
Phone: (253) 573-9939 Fax: (253) 573-9959

JOB NO: 81075880

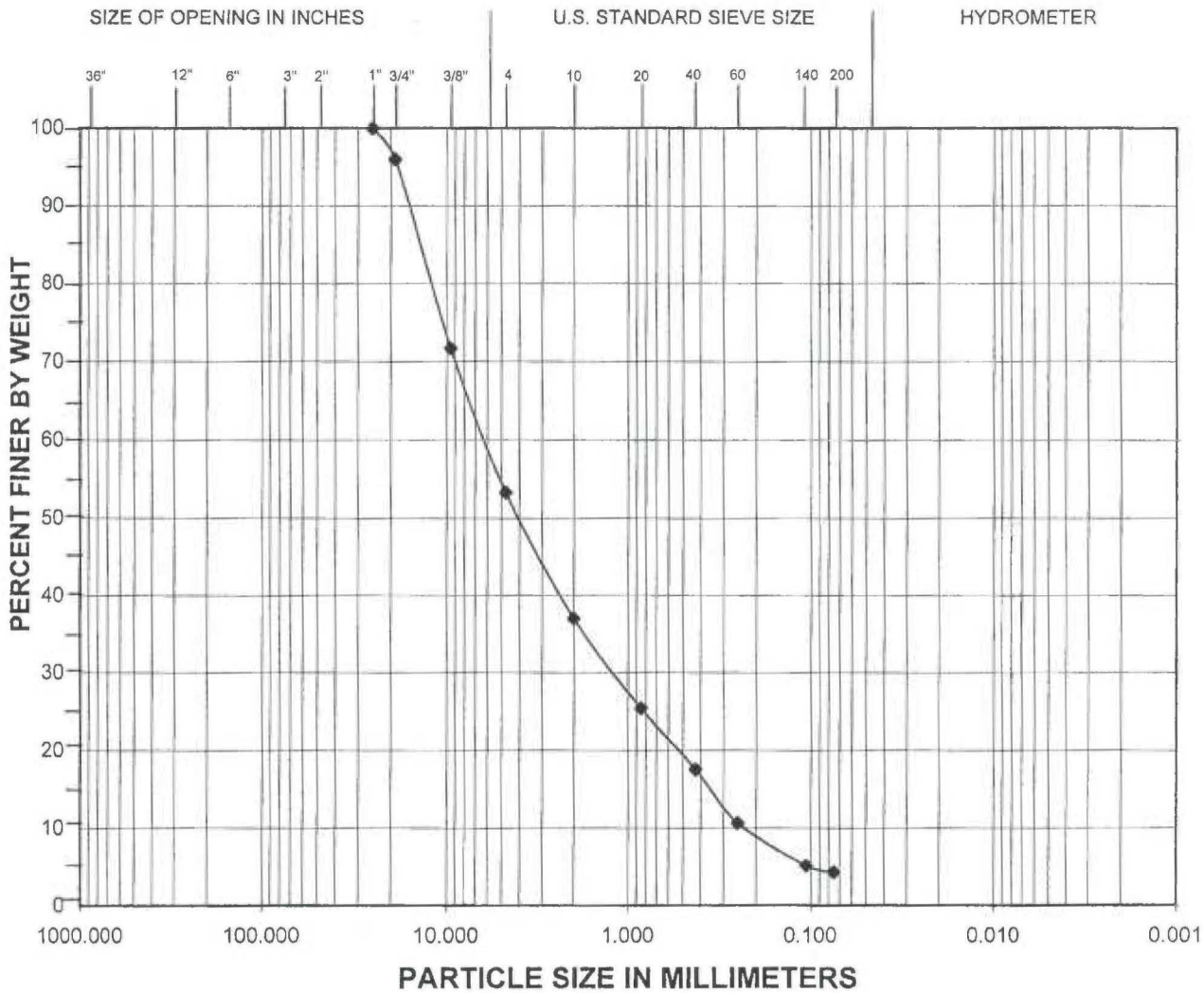
DATE OF TESTING: 12/12/2007

PROJECT NAME: Chico Way Bridge Repair

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D 422



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-8	37.5	8	4.3	gravelly SAND, trace silt

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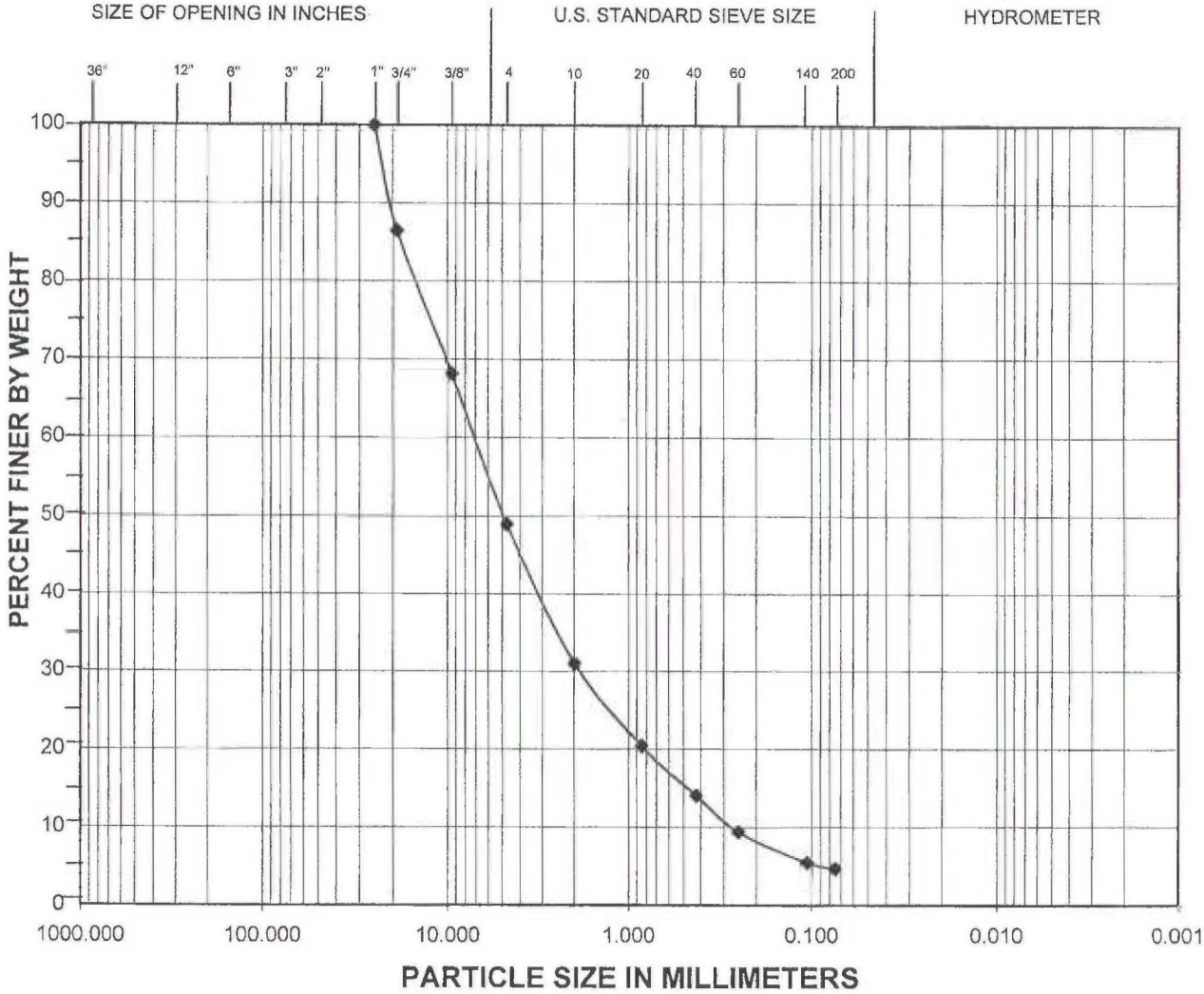

		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-7	27.5	9	4.7	sandy GRAVEL, trace silt

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DATE OF TESTING: 12/12/2007

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APPENDIX C
81075880

GENERAL NOTES AND UNIFIED SOIL CLASSIFICATION SYSTEM

GENERAL NOTES

DRILLING & SAMPLING SYMBOLS:

SS:	Split Spoon - 1-3/8" I.D., 2" O.D., unless otherwise noted	HS:	Hollow Stem Auger
ST:	Thin-Walled Tube - 2" O.D., unless otherwise noted	PA:	Power Auger
RS:	Ring Sampler - 2.42" I.D., 3" O.D., unless otherwise noted	HA:	Hand Auger
DB:	Diamond Bit Coring - 4", N, B	RB:	Rock Bit
BS:	Bulk Sample or Auger Sample	WB:	Wash Boring or Mud Rotary

The number of blows required to advance a standard 2-inch O.D. split-spoon sampler (SS) the last 12 inches of the total 18-inch penetration with a 140-pound hammer falling 30 inches is considered the "Standard Penetration" or "N-value".

WATER LEVEL MEASUREMENT SYMBOLS:

WL:	Water Level	WS:	While Sampling	N/E:	Not Encountered
WCI:	Wet Cave in	WD:	While Drilling		
DCI:	Dry Cave in	BCR:	Before Casing Removal		
AB:	After Boring	ACR:	After Casing Removal		

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. Groundwater levels at other times and other locations across the site could vary. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels may not be possible with only short-term observations.

DESCRIPTIVE SOIL CLASSIFICATION: Soil classification is based on the Unified Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

CONSISTENCY OF FINE-GRAINED SOILS

Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-value (SS) Blows/Ft.	Consistency
< 500	<2	Very Soft
500 – 1,000	2-3	Soft
1,001 – 2,000	4-6	Medium Stiff
2,001 – 4,000	7-12	Stiff
4,001 – 8,000	13-26	Very Stiff
8,000+	26+	Hard

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Standard Penetration or N-value (SS) Blows/Ft.	Relative Density
0 – 3	Very Loose
4 – 9	Loose
10 – 29	Medium Dense
30 – 49	Dense
50+	Very Dense

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 – 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75 mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 Sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 – 12
Modifiers	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1-10
Medium	11-30
High	30+

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UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	Cu ≥ 4 and 1 ≤ Cc ≤ 3[E]	GW	Well-graded gravel[F]
			Cu < 4 and/or 1 > Cc > 3[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	Cu ≥ 6 and 1 ≤ Cc ≤ 3[E]	SW	Well-graded sand[I]
			Cu < 6 and/or 1 > Cc > 3[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines Classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay[K,L,M,N]
					Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic Silt[K,L,M]
		organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay[K,L,M,P]
					Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

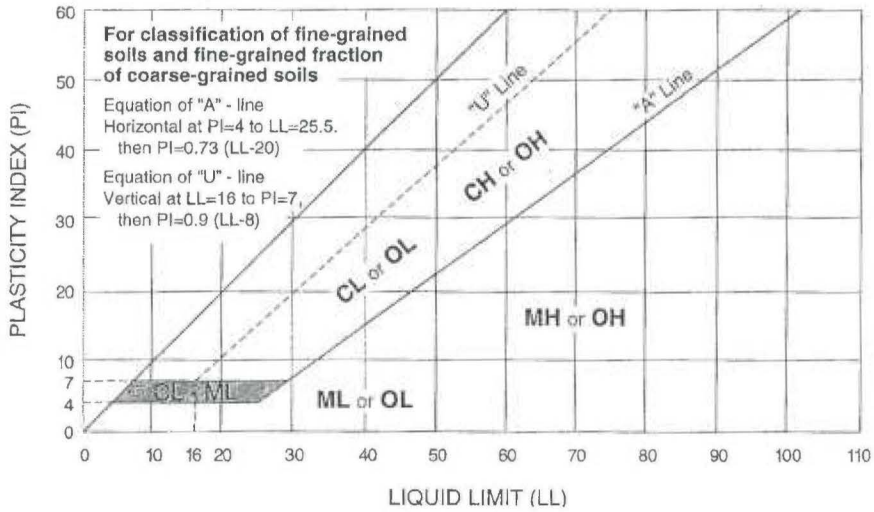
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



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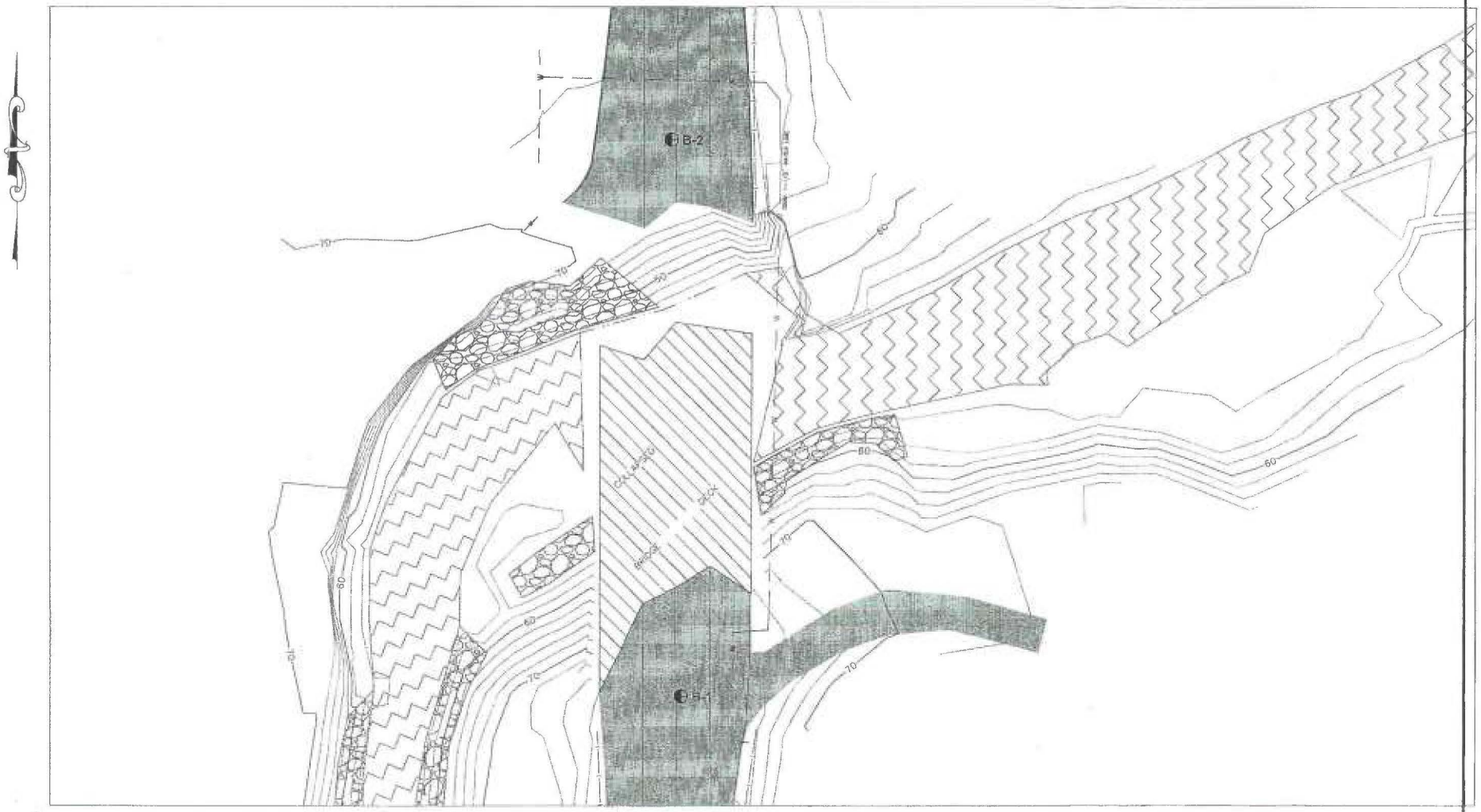

LEGEND:

B-1 BORING NUMBER AND APPROXIMATE LOCATION

0 30 60
APPROXIMATE SCALE IN FEET

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2115 S. 56th Street, Suite 405
Tacoma, Washington 98409
Tele: (253) 573-9939 Fax: (253) 573-9959

Project No: 81075880
Drawn by: J. Eastman
Date: Feb. 19, 2007
Scale: As Noted

Chico Way Bridge Replacement
Chico, Washington

FIGURE 1 - SITE AND EXPLORATION PLAN

Basemap DWG File Provided by Sargent Engineers, Inc. and modified by ZZA-Terracon