

APPENDIX C
FOUNDATION REPORT

FOUNDATION REPORT

Replacement of County Road 305 Bridge over Watson Creek Caltrans Bridge No. 11C-0091 Glenn County, California

Dist	Co	Rte	PM	EA
XX	Glenn	305	N/A	N/A

Report Prepared By:

WILLDAN GEOTECHNICAL



1515 South Sunkist Street, Suite E
Anaheim, CA 92806
Tel: (714) 634-3318
Fax: (714) 634-3372

Project No. 102330-4000
November 30, 2016

November 30, 2016 (Revised)
August 2, 2016 (Original)

Mr. Gary Gordon
Willdan Engineering
2400 Washington Avenue, Suite 101
Redding, CA 96001

Subject: Foundation Report
Replacement of County Road 305 Bridge over Watson Creek
Caltrans Bridge No. 11C-0091, Glenn County, California
Willdan Geotechnical Project No. 102330-4000

Dear Mr. Gordon,

Willdan Geotechnical is pleased to submit this Foundation Report (FR) for the proposed replacement of County Road 305 Bridge over Watson Creek (Caltrans Bridge No. 11C-0091) in Glenn County, California. This report has been written to meet the requirements for a FR per "Caltrans Foundation Report Preparation for Bridges", December 2009 Edition.

Should you have any questions regarding the contents of this report, or should you require additional information, please contact us.

Respectfully Submitted,
WILLDAN GEOTECHNICAL



Mohsen Rahimian

Mohsen Rahimian, PE, GE
Sr. Project Engineer
C 73396, GE 3059



Ross Khiabani

Ross Khiabani, PE, GE
Principal Engineer
C 37156, GE 2202

Distribution: Addressee

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- Appendix B: Laboratory Test Results
- Appendix C: Pile Capacity Graphs
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1.0 SCOPE OF WORK

This Foundation Report (FR) is presented to assist in the structure type selection for the proposed replacement of the County Road 305 Bridge over Watson Creek (Caltrans Bridge No. 11C-0091) in Glenn County, California.

This FR documents existing foundation conditions, provides preliminary structure-specific seismic recommendations, and makes preliminary foundation recommendations. The site geology and subsurface conditions discussed in this FR are based on review of available published data and the findings from the field exploration.

We have performed the following tasks as the scope of work for this FR:

- Provide site geology and subsurface conditions based on review of published data and the findings from the field exploration;
- Provide preliminary seismic recommendations, including addressing seismic hazards such as liquefaction potential, surface fault rupture potential, seismically induced settlement, and seismic slope instability, as applicable;
- Provide preliminary design recommendations for foundation, retaining walls, earthwork, and construction considerations.

No scour evaluation is included in this FR. Scour evaluation will be done as part of the hydraulics report prepared by others.

2.0 PROJECT DESCRIPTION AND SITE LOCATION

The existing bridge crosses Watson Creek and is located on County Road 305, approximately 400 feet east of the intersection of County Roads 305 and 306. The latitude and longitude at the approximate center of the proposed new bridge are 39.6801° N and 122.5301° W, respectively. The location of the project site is shown on Figure 1, Site Location Map.

The existing bridge is a four-span bridge comprised of reinforced concrete deck, girders, piers and abutments. According to the Caltrans Bridge Inspection Report dated 8/7/2013, the bridge has been built in 1920. The project entails replacement of the existing bridge with a new bridge on a new alignment.



3.0 FIELD INVESTIGATION AND TESTING PROGRAM

Willdan Geotechnical drilled and sampled four (4) soil borings. Borings A-16-001 and A-16-004 were drilled along the approach roadways and advanced to the maximum depth of 21.5 feet below ground surface (bgs). Borings A-16-002 and A-16-003 were drilled at the approximate locations of the abutments of the proposed new bridge to the maximum depth of 31.5 feet bgs. Underground Service Alert of Northern California and Nevada (USA North) was notified for clearance of underground utilities in the vicinity of the borings. Approximate borings locations are shown on Figure 2, Boring Location Plan.

Borings were advanced using a truck-mounted rig equipped with 7-inch diameter hollow-stem augers. Disturbed and relatively undisturbed drive samples were collected at select depth intervals from each soil boring. Bulk samples were collected from auger cuttings obtained from within the near-surface soils. Relatively undisturbed samples were collected by driving a three-inch outside diameter Modified California Sampler lined with brass rings and disturbed samples were collected by driving a 1 $\frac{3}{8}$ -inch inside diameter Standard Penetration split-spoon sampler. The samplers were driven into the underlying soil for intervals varying between 6 and 18 inches, with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6-inch penetration interval. The blow count for the final 12 inches, or for a lesser distance if the sampler could not be driven 12 inches, is shown on the Log of Test Borings in Appendix A. The number of blows required to drive the sampler the last 12 inches was used to estimate the in-situ relative density of granular soils, and to a lesser degree of accuracy, the consistency of cohesive soils. All soil samples were retained for laboratory testing. Upon completion of the borings, the boreholes were backfilled with soil cuttings.

Classification of the soils encountered in the exploratory borings was made in general accordance with the Unified Soil Classification System (USCS), using visual-manual procedure (ASTM D2488) and/or based on laboratory testing (ASTM D2487). A Log of Test Borings (LOTB) is included as Appendix A. The soil and rock descriptions in the LOTB are per Appendix A of Caltrans "Soil and Rock Logging, Classification, and Presentation Manual, 2010 Edition".

4.0 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. Laboratory testing included determination of in-situ moisture and density, passing #200 sieve, sieve analysis, direct shear, expansion index, R-value and corrosion potential for soil samples. Laboratory tests were conducted in general accordance with American Society for Testing of Materials (ASTM) Standards or California



Test Methods. The in-situ dry density and moisture content test results are shown on the LOTB. The remaining laboratory test results are presented in Appendix B, Laboratory Test Results.

Groundwater observations were made in the borings during drilling operations. Upon completion of the borings, the boreholes were backfilled with soil cuttings and pavement was patched with cold asphalt. Soil samples were delivered to Willdan's laboratory for testing.





Imagery ©2016 Google, Map data ©2016 Google 500 ft 

FIGURE 1. SITE LOCATION MAP

**REPLACEMENT OF COUNTY ROAD 305 BRIDGE OVER WATSON CREEK
GLENN COUNTY, CALIFORNIA**



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*extending
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reach*

Drawn By: MR

Date: 08-02-2016

Approved By: MR

Project No.: 102330-4000



Google earth

feet
meters



FIGURE 2. BORING LOCATION PLAN

REPLACEMENT OF COUNTY ROAD 305 BRIDGE OVER WATSON CREEK
GLENN COUNTY, CALIFORNIA



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5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 REGIONAL GEOLOGY

The project site lies on the western sloping flank of Sierra Nevada Mountains, east of the Great Valley geomorphic province in the east-central California. The Sierra Nevada Mountains were initially formed by uplifts that occurred during the Jurassic and Cretaceous periods of geologic time (greater than 65 million years ago). Renewed uplift began in the Sierra Nevada during late Tertiary time, and is continuing today. Sediments derived from the local mountainous areas are deposited in the San Joaquin Valley to the west. The Great Valley is a large northwestward trending, asymmetric structural trough that has been filled with as much as six vertical miles of sediment. The trough is situated between the Sierra Mountains on the east and the Coast Range Mountains on the west (Kleinfelder, 2009). Most of the localized drainage in the area is generally trending south toward downhill.

The subsurface soils consist of Quaternary Pleistocene and recent deposits such as siltstone, claystone and shale that overlie near surface Cretaceous granitic rock with varying thicknesses of overlying, fine to coarse soils. The Cretaceous granite is a fine grained, light colored granitic rock with Biotite and Muscovite. Locally sparse Plagioclitic K-feldspar phenocrysts as much as one centimeter in diameter can be observed (Krauskopf, 1985). Tectonic process, granite intrusion, hydrothermal alteration, and seepage are the main factors resulting in weakening of rocks and deeper penetration of mechanical and chemical weathering within the subject project site. Borings placed on the site during our investigation on January 1, 2016 encountered alluvium consisting of clayey gravel with sand, clayey sand with gravel, poorly graded gravel with clay and sand, and well graded gravel with clay and sand.

5.2 GROUNDWATER

The subject site is approximately 645 feet Above Mean Sea Level (AMSL). There is currently no map or data published by the Department of Conservation or the United States Geological Survey (USGS) to provide historical groundwater information at the site vicinity. Groundwater was encountered at about 11 feet bgs in all of our exploratory borings which corresponds to approximate elevation of 634 feet AMSL. The elevation of the encountered groundwater is higher than the elevation of the channel bed at approximately 630 feet AMSL and therefore, it is our opinion that the highest groundwater level is expected to correspond to the encountered groundwater level at 634 feet AMSL. Due to the type of the proposed development and expected depth of grading/excavation, it is likely that groundwater would be encountered during the the course of construction for the proposed development.



5.3 SUBSURFACE INFORMATION

The subsurface soils encountered in the borings to a depth ranging from 10 to 15 feet bgs, are consisted of layers of clayey gravel with sand, clayey sand with gravel, poorly graded gravel with clay and sand, and well graded gravel with clay and sand, which were found in medium dense to dense condition. Following these layers and to the maximum depth drilled a layer of very dense clayey sand with gravel was encountered. Table 1 provides a generalized subsurface profile interpreted based on our current geotechnical investigation.

Table 1. Generalized Subsurface Profile

Layer No.	Material	Top of Layer Elevation (feet AMSL)
1	Clayey Sand/Gravel	645
2	Clayey Sand with Gravel	630

Table 2 summarizes the estimated soil strength properties for the generalized subsurface strata profile shown in Table 1.

Table 2. Idealized Soils Properties

Layer No.	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degree)	N _{SPT}
1	130	N/A	N/A	N/A
2	135	1300	15	60

6.0 SCOUR EVALUATION

An evaluation of the scour potential is not included as part of this FR. Scour evaluation will be done as part of the hydraulics report prepared by others.

7.0 CORROSION EVALUATION

The available test results for pH, minimum resistivity, soluble chloride content and soluble sulfate content on samples for the bridge site vicinity shows pH value of 7.5, minimum resistivity of 880 ohm-cm, soluble chloride content of 28 parts per million (ppm), and soluble sulfate content of 35 ppm.

The Caltrans Corrosion Guidelines (Caltrans, 2012) classifies soil as corrosive if the soluble



chloride content is 500 ppm or greater, if the soluble sulfate content is 2,000 ppm or greater, or if the pH is 5.5 or less. Based on the above test results and the Caltrans criteria, the on-site soils are not considered to be corrosive to bare metals and concrete.

8.0 SEISMIC CONSIDERATIONS

8.1 SEISMIC GROUND MOTION INFORMATION

According to current data from Caltrans, the controlling fault for a deterministic scenario is the Great Valley 01 fault, located approximately 13.8 km east of the site. Table 3 summarizes the fault parameters.

Table 3. Controlling Fault for Deterministic Seismic Scenario

Name	Type	Dip	PGA	Maximum Moment Magnitude
Great Valley 01 Fault	Reverse	15°	0.311 g	6.7

There is currently no map or data published by the USGS to provide information with respect to the special studies zones at the site vicinity, however the site lies in seismically active zone and will be subject to strong ground shaking.

8.2 DESIGN RESPONSE SPECTRUM

Figure 3, Design Acceleration Response Spectrum, shows a plot of the acceleration response spectrum (ARS) curve considering near-fault effects. The corrections for near-fault effects were done as per recommendations contained in Appendix B of the Caltrans Seismic Design Criteria.

The design spectral acceleration values are the envelope of the probabilistic and deterministic spectra, and are controlled by probabilistic criteria. The deterministic spectrum is determined using version 2.3.07 of the Caltrans ARS Online tool, and the probabilistic spectrum using the probabilistic seismic hazard assessment program developed by the United States Geological Survey (USGS) 2008 Interactive Deaggregation website. We estimated a deaggregated moment magnitude of 7.07 for a return period of 975 years (5% probability of exceedance in 50 years). The shear wave velocity at a depth of 30 meters used for the deaggregation analyses is 560 m/s, estimated based on the NEHRP classification (FEMA, 1994 & 1997) and the data collected during current subsurface investigations by Willdan within the project site.



8.3 LIQUEFACTION

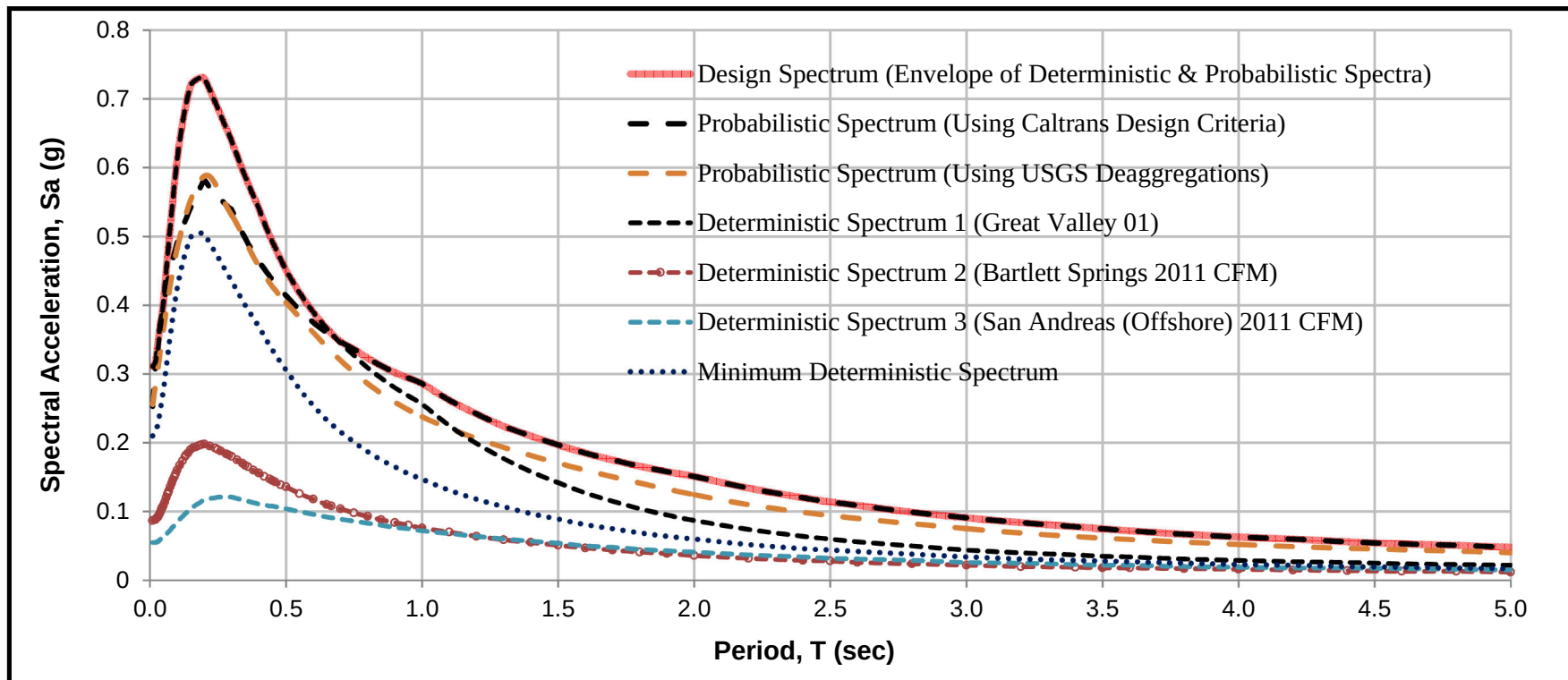
Liquefaction is the loss of strength that can occur in saturated coarse-grained soils during earthquake seismic shaking. The susceptibility of a granular soil to liquefaction is a function of the gradation, relative density, and fines content of the soil. Susceptibility to liquefaction generally decreases with increasing mean grain size, relative density, fines content and clay-size fraction of the fines, and the age of the deposit.

The subsurface soils at the bridge site are classified as clayey sand and gravel. Although bedrock was not encountered in the borings drilled at the site for the current investigation, but the SPT N-values (above 100) indicate that the recommended foundation bearing soils located below the elevation of 630 feet AMSL are in very dense condition. As such, it is our professional opinion that the bridge site is not susceptible to liquefaction under the design seismic scenario.

8.4 SURFACE FAULT RUPTURE POTENTIAL

No known faults project through the site. As such, it is our professional opinion that surface fault rupture is not likely to occur at the project site during the design seismic scenario.





Design Curve	Design Criteria	Probability of Exceedence	Return Period	Moment Magnitude	PGA (g)
	Envelope of Deterministic & Probabilistic Spectra with Near-Fault Effect Included ^{1, 2}	5 percent in 50 years	975 years	7.07	0.256

Notes:
 1 - Based on Caltrans Seismic Design Criteria, Version 2.3.06, November 23, 2013
 2 - Design spectrum is controlled by probabilistic criteria. Probabilistic spectral accelerations for small periods are obtained using the United State Geological Survey (USGS) 2008 Interactive Deaggregations website at geohazards.usgs.gov/deaggint/2008/

FIGURE 3. DESIGN ACCELERATION RESPONSE SPECTRUM Replacement of County Road 305 Bridge over Watson Creek Glenn County, California		
	Drawn By: MR	Date: 9-Mar-16
	Approved By: RK	Project No. 102330-4000

9.0 SLOPE STABILITY

The embankment slopes at a slope ratio of 2:1 (H:V) or flatter are expected to be stable under both static and design seismic loads.

10.0 FOUNDATION RECOMMENDATIONS

It is our opinion that the proposed new bridge may be supported on conventional spread/strip footings or cast-in-drilled-hole (CIDH) concrete piles. The following sections of this report contain our geotechnical recommendations for design and construction of two different types of foundation system. For the purposes of this report we have assumed that the column loads and continuous load will be less than 90 kilo pounds (kips) and 10 kips per foot, respectively.

10.1 ALTERNATIVE A: SPREAD/STRIP FOOTINGS

Bearing Capacity: The footings shall have a minimum width of 24 inches and be embedded in competent bearing soils at least 6 feet below the bottom of creek at approximate elevation of 624 feet. The bottom of footing excavation shall be scarified to a minimum depth of 8 inches and compacted to at least 90% relative compaction of the maximum density as determined by the ASTM D1557 test procedure. The footings may be designed using a maximum allowable bearing value of 2,500 pounds per square foot (psf). A one-third increase in the bearing value may be used when considering wind or seismic loads.

Lateral Resistance: Lateral soil resistance will be provided by a combination of frictional resistance between the bottom of the footings and the underlying soils, and by passive soil resistance acting against side of the footing. For frictional resistance between concrete and soil, a frictional coefficient of 0.35 may be used. For passive resistance, an allowable pressure developed by a fluid with density of 150 pound per cubic foot (pcf), to a maximum pressure of 3500 psf, may be used for a level ground surface condition in front of the footing. To consider the scour effect, the soil within the ground surface and 6 feet below the bottom of creek at approximate elevation of 624 feet should be neglected in passive resistance calculation. When combining both frictional and passive resistance, the passive resistance should be reduced by one-third.

Settlement: Our preliminary computations indicate that the total settlement of the footings due to the anticipated loads, for footings designed as recommended here, will be less than ½ inch, and the differential settlements are expected to be less than ¼ inch over a 50-foot span.

10.2 ALTERNATIVE B: CDIH PILES

Axial Capacity: Allowable downward and uplift capacities for piles with different diameters were evaluated using SHAFT 2012 program and are presented in Appendix C. The presented



graphs are provided for 24, 30 and 36 inches diameter piles. Similar graphs for different diameters other than above will be provided upon request. The capacities are based on frictional resistance of the piles. For frictional pile design using the attached graphs, the weight of the shaft can be assumed to be taken by end-bearing resistance of the pile and it is not necessary to add the weight of the shaft to the structural loads. Uplift capacity of the pile may be assumed as half of the downward capacity of the pile. It is recommended that the piles have a minimum diameter of 24 inches and minimum embedment length of 10 feet. The actual length of the drilled piles shall be calculated by the structural engineer for the project, considering recommendations provided herein. The provided capacities are based on the strength of the soils, not the pile section, which should be designed and checked by the project structural engineer.

Lateral Capacity: Lateral loads can be resisted by passive pressure developed against the vertical shafts. The lateral capacity of the pile depends on the permissible deflection and the degree of fixity at the top of the pile. For this project, lateral resistance of a free-head and a fixed-head single pile were evaluated using LPILE 2015 program.

A lateral deflection of ½ inch has been applied to the top of the pile, and the lateral capacity graphs of lateral deflection, shear force and bending moment vs. depth, for a 25-foot long and 36-inch diameter pile with 90 kips axial load are presented within Appendix C. The provided capacities are based on the strength of the soils, not the pile section, which should be designed and checked by the project structural engineer.

Settlements: Our preliminary computations indicate that the total settlement at the top of the piles due to the anticipated loads, for piles designed as recommended here, will be less than ½ inch.

11.0 ABUTMENT, WING AND RETAINING WALLS

11.1 ABUTMENT WALLS

The lateral earth pressure behind the abutment walls, which are restrained at the top, may be estimated using the recommendations of Section 5.5.5.11 of the Caltrans Bridge Design Specifications (2004). The walls may be designed using the pressure that is developed by an equivalent fluid with density of 65 pounds per cubic foot (pcf) and 45 pcf for at-rest and active pressure, respectively.

The abutment walls shall also be designed in accordance to the recommendations of Section 7.8 of the Caltrans Seismic Design Criteria (Caltrans SDC 1.7, 2013). The walls may be designed for a passive resistance force calculated using Equation 7.8.1-3 from the SDC to resist movement at the abutment walls.



11.2 WING WALLS AND RETAINING WALLS

Lateral earth pressures for the design of wing walls and retaining walls may be assumed to be equal to the pressure developed by an equivalent fluid with density presented in Table 4.

Table 4. Summary of Lateral Earth Pressures

Lateral Earth Pressure Condition/Factor	Equivalent Fluid Density / Frictional Coefficient
Active Pressure	45 pcf
At-Rest Pressure	65 pcf
Passive Pressure	400 pcf
Frictional Coefficient (between bottom of concrete footing and soil)	0.35

In addition to the above active earth pressure, the walls more than 12 feet high, should be designed to support a seismic active pressure. The seismic active lateral earth pressure may be assumed to be an inverted triangular pressure distribution equal to 30H psf at the top of the retaining wall, and decreasing linearly to zero at the bottom of retaining wall, where H is the height of retaining wall in feet.

11.3 WALLS FOUNDATIONS

The walls may be supported on shallow or deep foundations designed in accordance with the recommendations provided in Section 10.0 of this report.

11.4 SUBDRAIN INSTALLATION

Subdrain systems shall be installed to prevent hydrostatic pressure build-up acting as an additional lateral load. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. Retaining wall backfill and typical subdrain details for conditions of native soil, imported sand, or crushed rock are provided in Appendix D.

12.0 PAVEMENT DESIGN

Laboratory testing of a bulk sample from the shallow subsurface soil of the approach roadway of the subject bridge indicates a minimum R-value of 8. A flexible section consisting of asphalt concrete (AC) over aggregate base (AB), or a full-depth AC section may be used. The pavement sections listed in Table 5 have been developed in accordance with the procedure



presented in the Caltrans Highway Design Manual for a range of traffic index (TI) values.

Table 5. Flexible Pavement Design

TI	AC/AB (in/in)	Full Depth AC (in)
6	3.5/12.0	9.0
8	5.0/17.0	12.0
10	6.5/21.5	15.5

The pavement section shall be supported on the subgrade prepared per recommendations of Section 13.0 of this report. The base material shall consist of AB-Class 2 as specified in the Caltrans Standard Specifications (2010), and compacted to a minimum of 95% of maximum dry density.

13.0 EARTHWORK RECOMMENDATIONS

All earthwork and grading should be performed in accordance with the recommendations of this report and requirements of Section 19 of the Caltrans Standard Specifications (2010). Within the approach roads, any existing fills or soils disturbed during construction and associated site clearing operations should be removed down to a minimum of 24 inches and replaced with engineered fill.

The exposed subgrade to receive fill or pavement section should be scarified to a minimum of 8 inches and compacted to minimum of 90% relative compaction. The fill materials under the roadways and behind the retaining walls shall be placed in loose lifts not exceeding 8 inches in thickness, moisture-conditioned and compacted to minimum 90% of relative compaction. The onsite soil free of debris and deleterious material, or import granular material may be used as backfill material.

14.0 CONSTRUCTION CONSIDERATIONS

14.1 TEMPORARY EXCAVATION

Temporary excavations shall be properly sloped or shored. Based on the earth materials encountered in our borings, excavation of 5 feet or less in depth may be performed with



vertical sidewalls. Deeper excavation up to a depth of 15 feet can be accomplished in accordance with the Occupational Safety and Health Administration (OSHA) requirements for Type B soils. The contractor is responsible for maintaining the stability of the cuts and personnel safety in the field during construction. All excavations shall be performed in accordance with applicable requirements established by the State, County, or local government. The regulatory requirement may supersede the recommendations presented in this section. A representative of the geotechnical engineer of record should be present during all excavations.

14.2 CIDH PIERS INSTALLATION

Although during the course of field investigation, no caving was noticed in the borings, caving should be anticipated when the layers are sandy, gravelly or less cohesive, and when drilling below the groundwater table. Precautions should be taken during the drilling operation to minimize caving of the drilled holes. To minimize caving potential, it is recommended to keep pile diameter as small as possible. Other means and methods such as using casing or drilling mud may be employed by contractor when necessary. Experienced contractors shall be retained to install drilled pile foundations. It is necessary to perform continuous observation during pile construction by a project geotechnical engineer's representative.

Piles closer than three pile diameter on center to each other shall be drilled and filled with concrete alternately and concrete shall be permitted to set at least 8 hours before drilling an adjacent pile. The drilled hole shall be inspected and filled with concrete as soon as possible. The holes should not be left open overnight. The concrete shall be poured using tremie method.

15.0 LIMITATIONS

This report is based on the available information for the project and obtained from the current subsurface investigations. The materials data available from the current investigation are believed to be representative of the subject project site, and the conclusions and recommendations contained in this report are presented on that basis. However, soil materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. Any changes noted during construction should be brought to the attention of the Geotechnical Engineer so that any changes to these recommendations can be made as appropriate.

This Foundation Report (FR) has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and time period.



The contents of this report are professional opinions and as such, are not to be considered a guarantee or warranty.

The information contained herein has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.



16.0 REFERENCES

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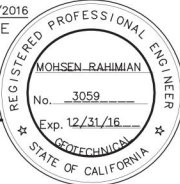


APPENDIX A: LOGS OF TEST BORINGS



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	Gle	CR 305	NA	1	4

<u> </u> <u> </u> MOHSEN RAHIMIAN <u> </u> GEOTECHNICAL ENGINEER	<u>2/16/2016</u> <u> </u> DATE
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<u> </u> <u> </u> PLANS APPROVAL DATE	
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The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

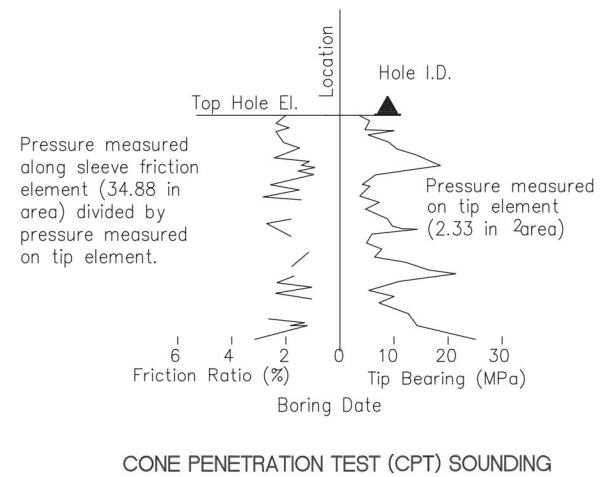
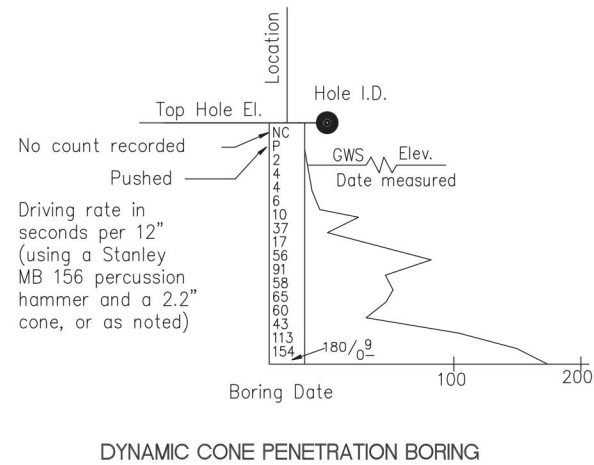
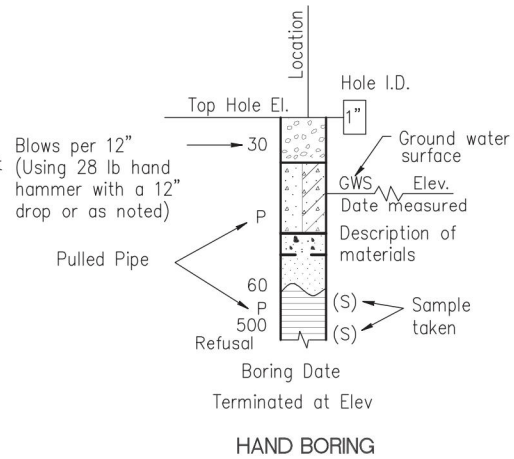
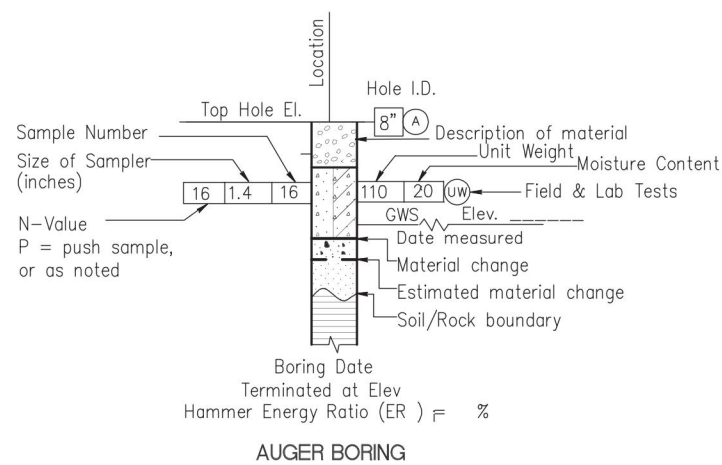
CEMENTATION	
Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS				
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

BOREHOLE IDENTIFICATION		
Symbol	Hole Type	Description
	A	Auger Boring
	R	Rotary drilled boring
	P	Rotary percussion boring (air)
	R	Rotary drilled diamond core
	HD	Hand driven (1-inch soil tube)
	HA	Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS	
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



DESIGN OVERSIGHT SIGN OFF DATE:	DRAWN BY: AM CHECKED BY: MR	FIELD INVESTIGATION BY: DATES:		PREPARED FOR THE GLENN COUNTY DEPARTMENT OF PUBLIC WORKS	G. GORDON PROJECT ENGINEER	BRIDGE NO. 11C0035 POST MILES .NA	WATSON CREEK BRIDGE (REPLACE) LOG OF TEST BORINGS												
GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)			ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3	UNITS: PROJECT NUMBER & PHASE	Contract No.:	DISREGARD PRINTS BEARING EARLIER REVISION DATES <table border="1" style="float: right; margin-top: 10px;"> <thead> <tr> <th colspan="4">REVISION DATES</th> <th>SHEET</th> <th>OF</th> </tr> </thead> <tbody> <tr> <td>1</td> <td></td> <td></td> <td></td> <td>1</td> <td>4</td> </tr> </tbody> </table>		REVISION DATES				SHEET	OF	1				1	4
REVISION DATES				SHEET	OF														
1				1	4														

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	Gle	CR 305	NA	2	4

MOHSEN RAHIMIAN
GEOTECHNICAL ENGINEER

2/16/2016
DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER

MOHSEN RAHIMIAN

No. 3059

Exp. 12/31/16

GEOTECHNICAL

STATE OF CALIFORNIA

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GROUP SYMBOLS AND NAMES					
Graphic/Symbol		Group Names	Graphic/Symbol		Group Names
	GW	Well-graded GRAVEL Well-graded GRAVEL with SAND		CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
	GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
	GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND		ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
	GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
	GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND		OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
	GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
	GM	SILTY GRAVEL SILTY GRAVEL with SAND		MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
	GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
	GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND		OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
	SW	Well-graded SAND Well-graded SAND with GRAVEL		OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	SP	Poorly graded SAND Poorly graded SAND with GRAVEL			
	SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL			
	SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)			
	SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL			
	SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)			
	SM	SILTY SAND SILTY SAND with GRAVEL			
	SC	CLAYEY SAND CLAYEY SAND with GRAVEL			
	SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL			
	PT	PEAT			
		COBBLES COBBLES and BOULDERS BOULDERS			

FIELD AND LABORATORY TESTING	
	Consolidation (ASTM D 2435)
	Collapse Potential (ASTM D 5333)
	Compaction Curve (CTM 216)
	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
	Consolidated Undrained Triaxial (ASTM D 4767)
	Direct Shear (ASTM D 3080)
	Expansion Index (ASTM D 4829)
	Moisture Content (ASTM D 2216)
	Organic Content-% (ASTM D 2974)
	Permeability (CTM 220)
	Particle Size Analysis (ASTM D 422)
	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
	Point Load Index (ASTM D 5731)
	Pressure Meter
	Passing # 200 (ASTM D 1140-00)
	R-Value (CTM 301)
	Sand Equivalent (CTM 217)
	Specific Gravity (AASHTO T 100)
	Shrinkage Limit (ASTM D 427)
	Swell Potential (ASTM D 4546)
	Pocket Torvane
	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
	Unconsolidated Undrained Triaxial (ASTM D 2850)
	Unit Weight (ASTM D 4767)
	Vane Shear (AASHTO T 223)

APPARENT DENSITY OF COHESIONLESS SOILS	
Description	SPT N ₆₀ (Blows / 12 inches)
Very loose	0 – 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

MOISTURE	
Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

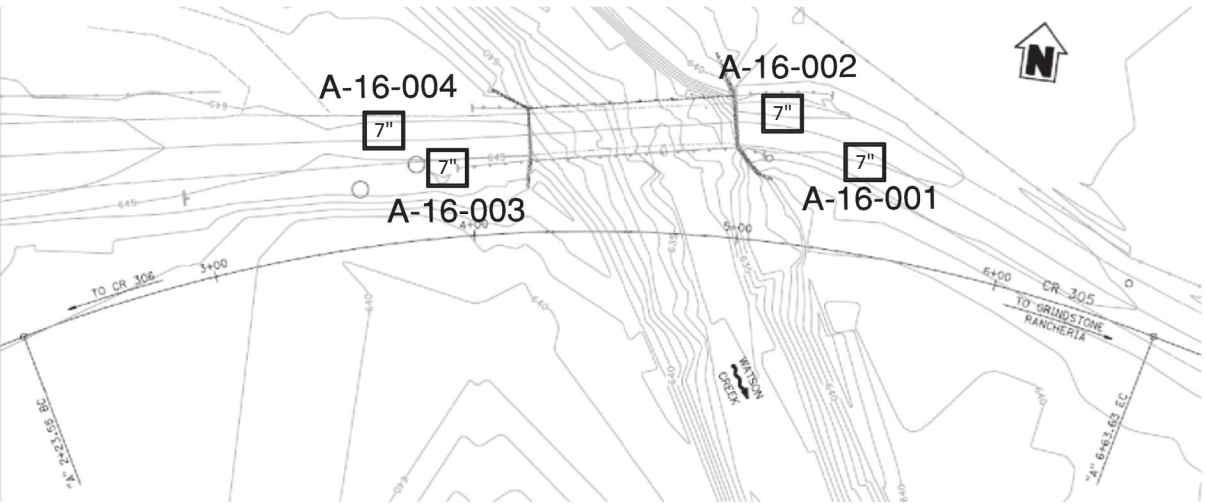
PERCENT OR PROPORTION OF SOILS	
Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE		
Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT	DRAWN BY: AM		FIELD INVESTIGATION BY:		PREPARED FOR THE GLENN COUNTY DEPARTMENT OF PUBLIC WORKS	G.GORDON PROJECT ENGINEER	BRIDGE NO. 11C0035	SOIL LEGEND			
SIGN OFF DATE	CHECKED BY: MR		DATES:				POST MILES NA	LOG OF TEST BORINGS			
GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)						UNITS: PROJECT NUMBER & PHASE	Contract No.:	DISREGARD PRINTS BEARING EARLIER REVISION DATES			
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS						0 1 2 3		REVISION DATES	1		
								SHEET	2	OF	4

BENCH MARK

Elevations are based on 1988 National Geodetic Vertical Datum (NGVD)
Bench Mark elevations are based on a set PK Nail and Shiner (Point 84) located at 15.71' left of "A" 6+48.07 having an elevation equal to 642.91.



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	Gle	CR 305	NA	3	4

MOHSEN RAHIMIAN
GEOTECHNICAL ENGINEER

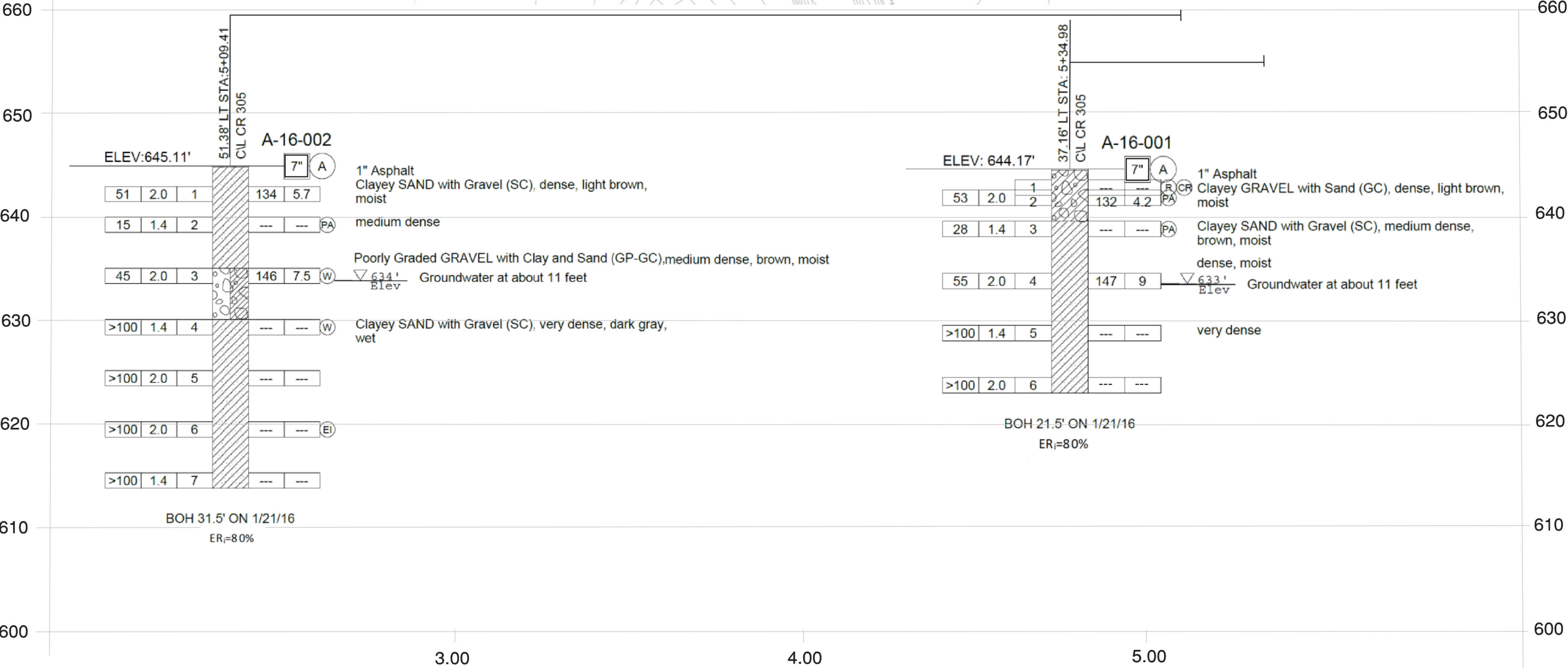
2/16/2016
DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
MOHSEN RAHIMIAN
No. 3059
Exp. 12/31/16
STATE OF CALIFORNIA

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HOR SCALE: 1"=30'
VER SCALE: 1"=10'



DESIGN OVERSIGHT	DRAWN BY: AM	FIELD INVESTIGATION BY:	PREPARED FOR THE GLENN COUNTY DEPARTMENT OF PUBLIC WORKS	G. GORDON PROJECT ENGINEER	BRIDGE NO. 11C0035	WATSON CREEK BRIDGE (REPLACE)
SIGN OFF DATE	CHECKED BY: MR	DATES:			POST MILES NA	LOG OF TEST BORINGS
GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)				UNITS: PROJECT NUMBER & PHASE	Contract No.:	REVISION DATES 1 3 4

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS

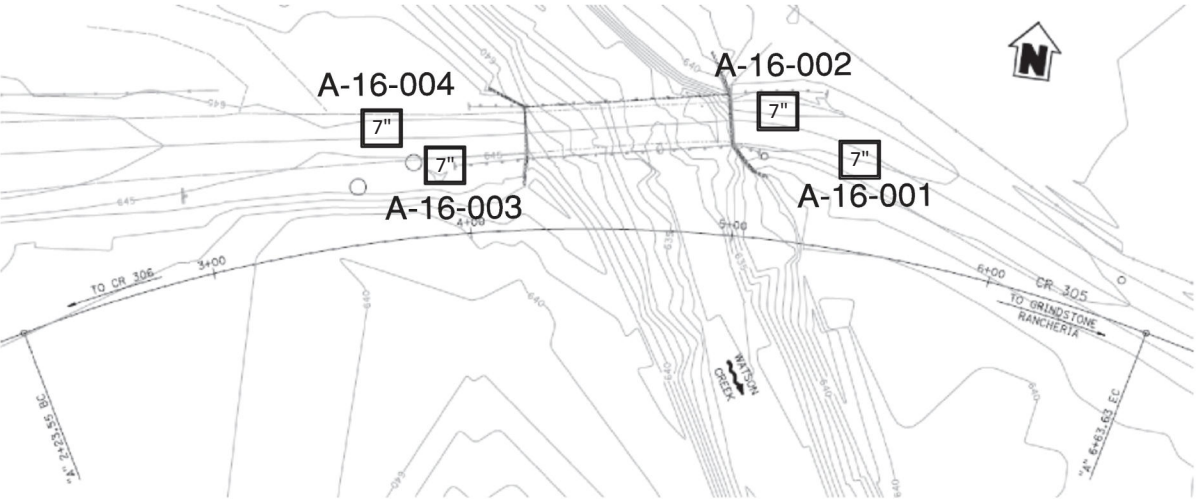
0 1 2 3

DATE PLOTTED => \$TIME

USER NAME => \$USER

BENCH MARK

Elevations are based on 1988 National Geodetic Vertical Datum (NGVD)
Bench Mark elevations are based on a set PK Nail and Shiner (Point 84) located at 15.71' left of "A" 6+48.07 having an elevation equal to 642.91.



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	Gle	CR 305	NA	4	4

MOHSEN RAHIMIAN
GEOTECHNICAL ENGINEER

2/16/2016
DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER

MOHSEN RAHIMIAN

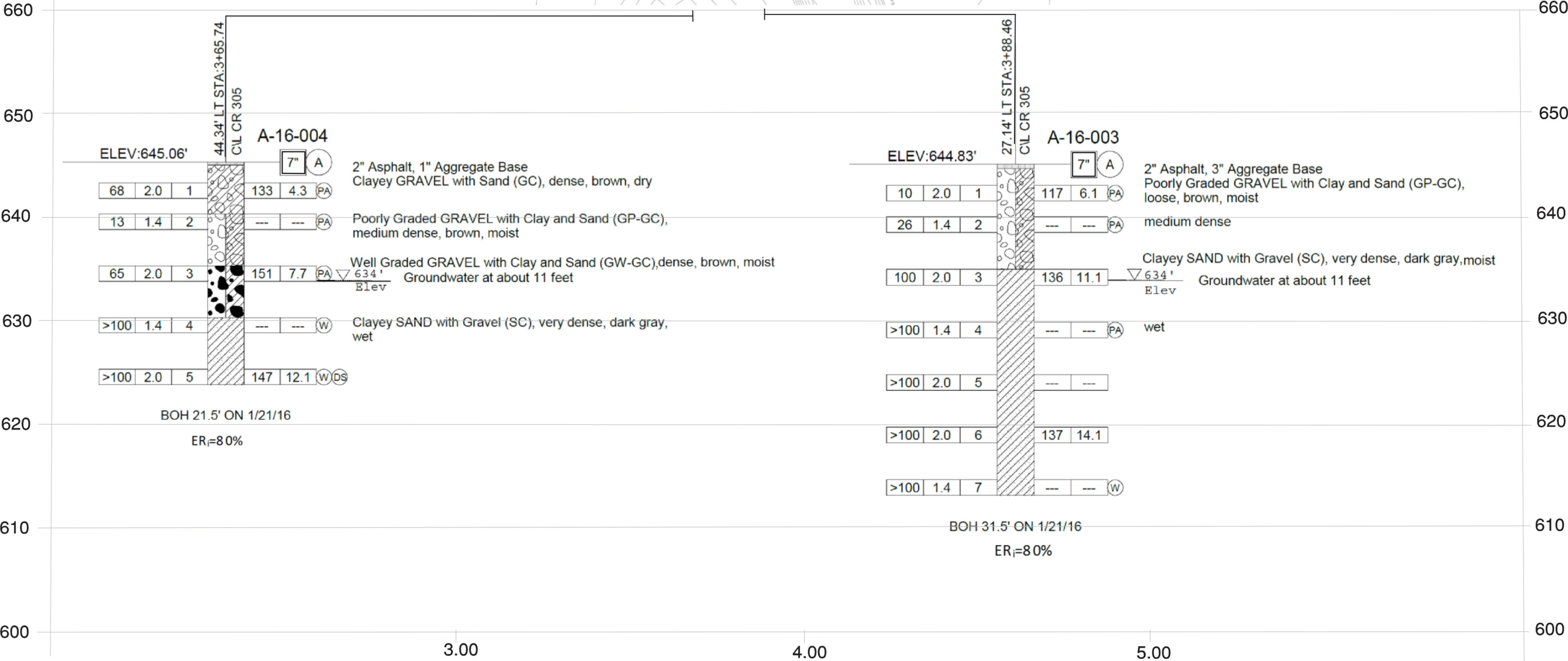
No. 3059

Exp. 12/31/16

STATE OF CALIFORNIA

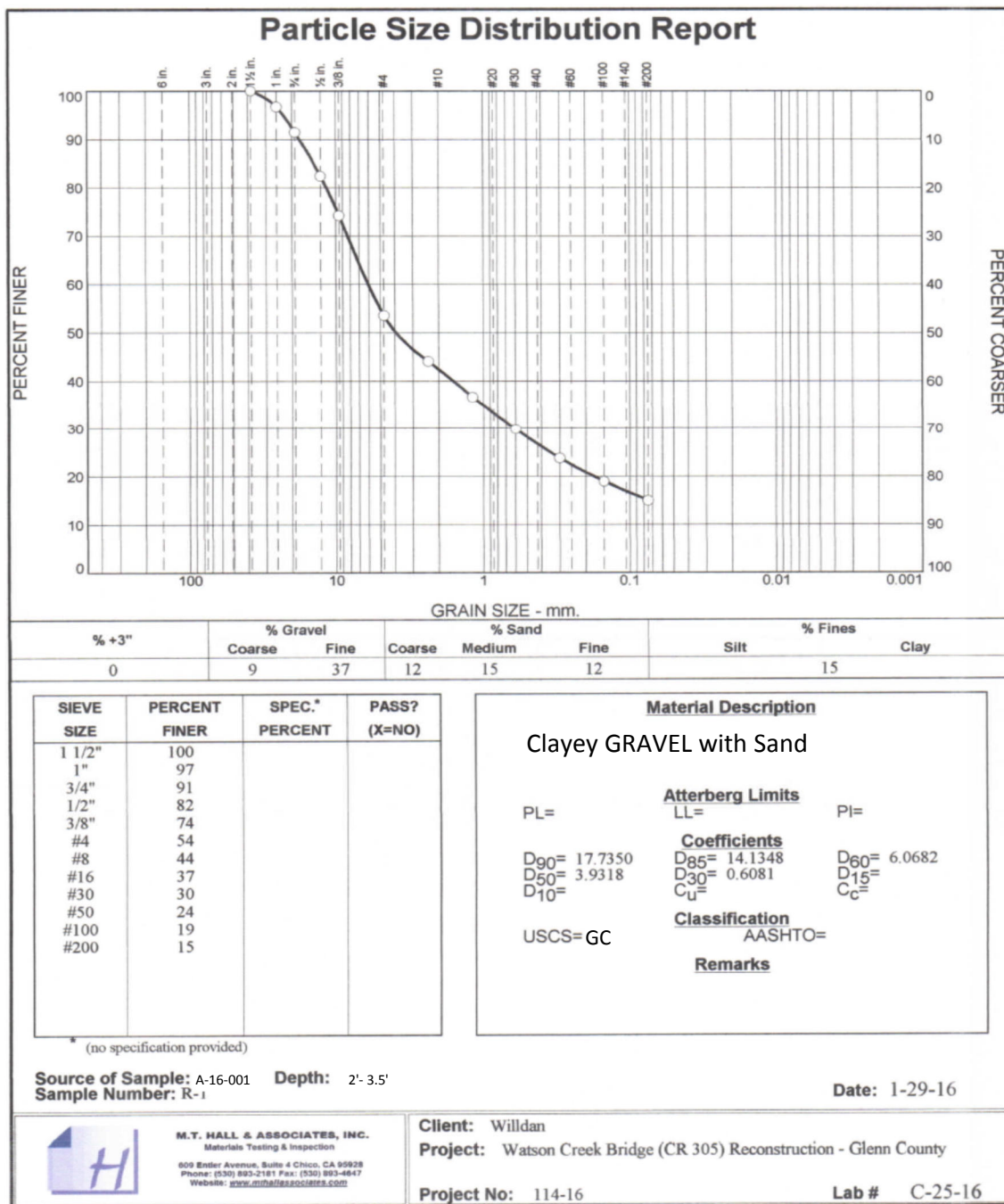
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HOR SCALE: 1"=30'
VER SCALE: 1"=10'



APPENDIX B: LABORATORY TEST RESULTS





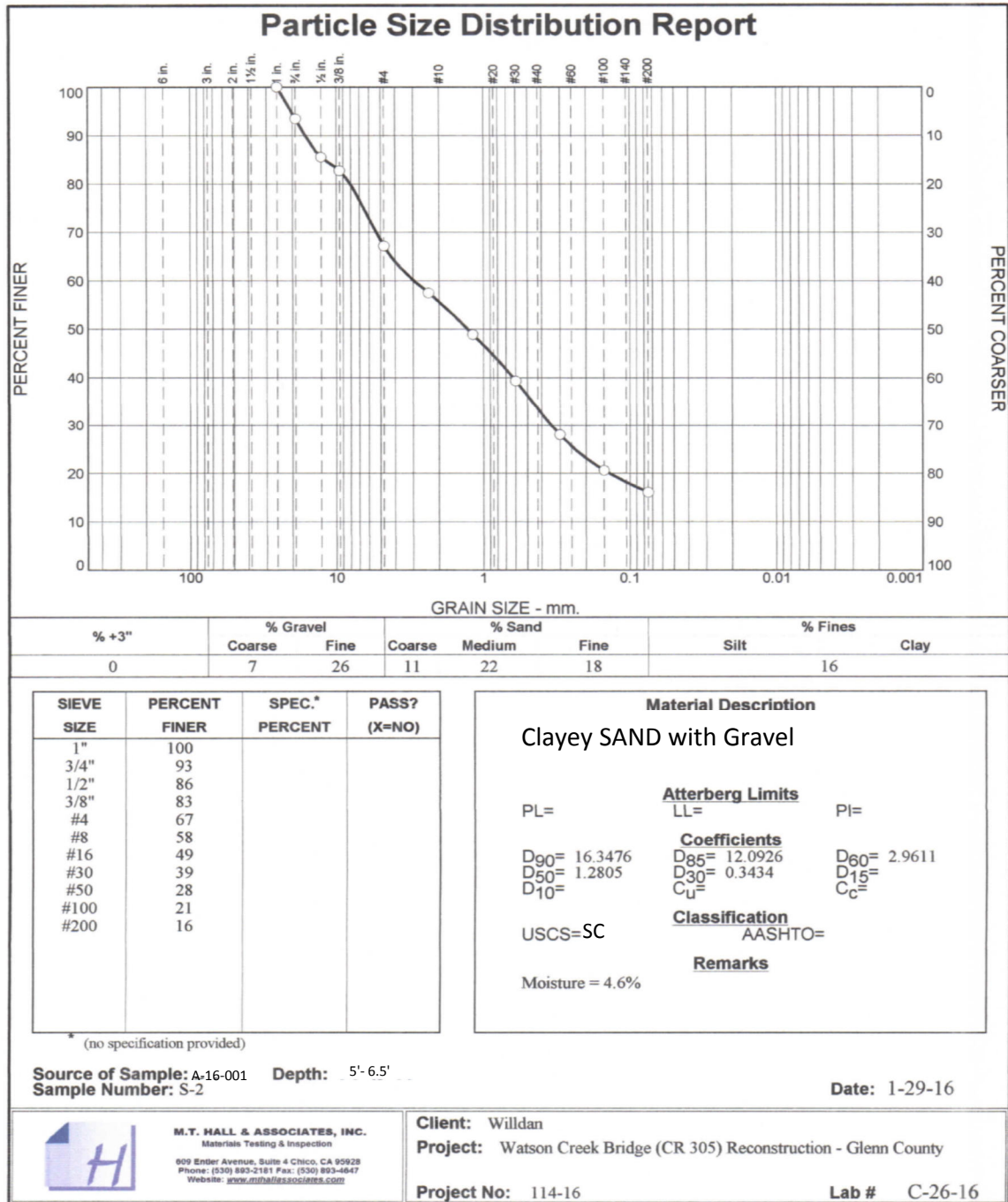
Tested By: Rick Kuntz Checked By: Tom Hall P.E.

PARTICLE SIZE DISTRIBUTION TEST

**WATSON CREEK BRIDGE (REPLACE)
GLENN COUNTY, CALIFORNIA**



Drawn By: MTH	Date: 17-Feb-2016
Approved By: MR	Project No. 102330-4000



Tested By: Rick Kuntz Checked By: Tom Hall P.E.

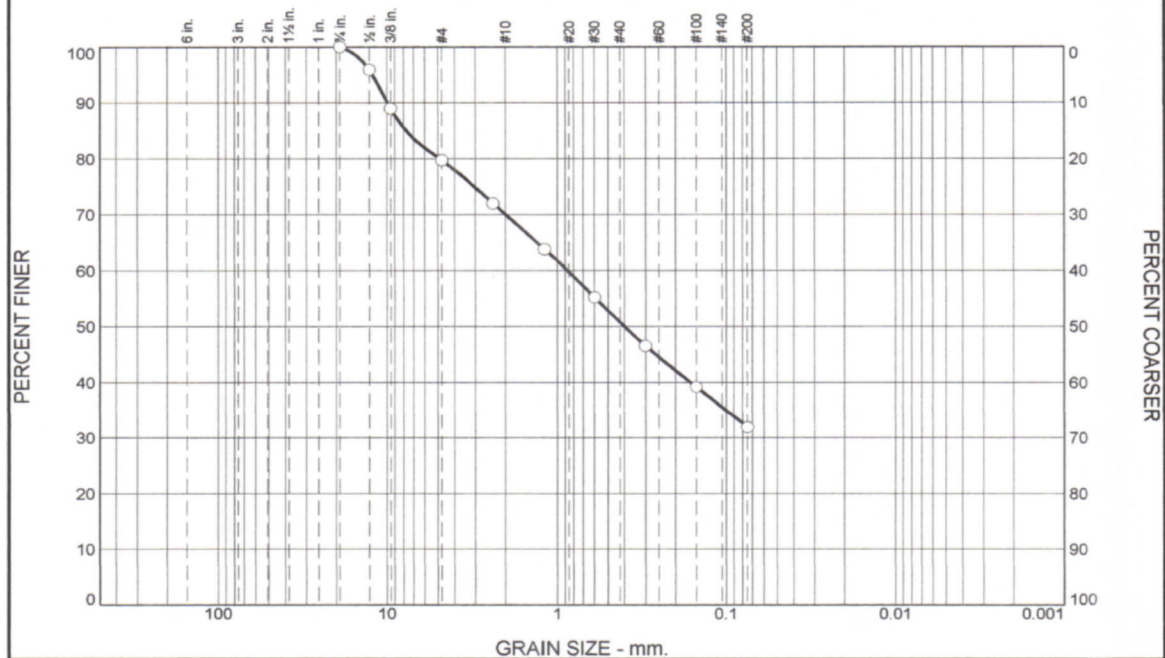
PARTICLE SIZE DISTRIBUTION TEST

**WATSON CREEK BRIDGE (REPLACE)
GLENN COUNTY, CALIFORNIA**



Drawn By: MTH	Date: 17-Feb-2016
Approved By: MR	Project No. 102330-4000

Particle Size Distribution Report



% +3"		% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0		0	20	10	19	19	32	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	96		
3/8"	89		
#4	80		
#8	72		
#16	64		
#30	55		
#50	47		
#100	39		
#200	32		

* (no specification provided)

Material Description

Clayey SAND with Gravel

Atterberg Limits
 PL= LL= PI=
Coefficients
 D₉₀= 9.9508 D₈₅= 7.7303 D₆₀= 0.8699
 D₅₀= 0.3999 D₃₀= D₁₅=
 D₁₀= C_u= C_c=
Classification
 USCS= SC AASHTO=
Remarks
 Moisture = 7.3%

Source of Sample: A-16-002 Depth: 5'-6.5'
 Sample Number: S-2

Date: 2-1-16



M.T. HALL & ASSOCIATES, INC.
 Materials Testing & Inspection
 609 Enter Avenue, Suite 4 Chico, CA 95928
 Phone: (530) 893-2181 Fax: (530) 893-4647
 Website: www.mthallassociates.com

Client: Willdan
 Project: Watson Creek Bridge (CR 305) Reconstruction - Glenn County

Project No: 114-16

Lab # C-11-16

Tested By: Rick Kuntz Checked By: Tom Hall P.E.

PARTICLE SIZE DISTRIBUTION TEST

**WATSON CREEK BRIDGE (REPLACE)
 GLENN COUNTY, CALIFORNIA**



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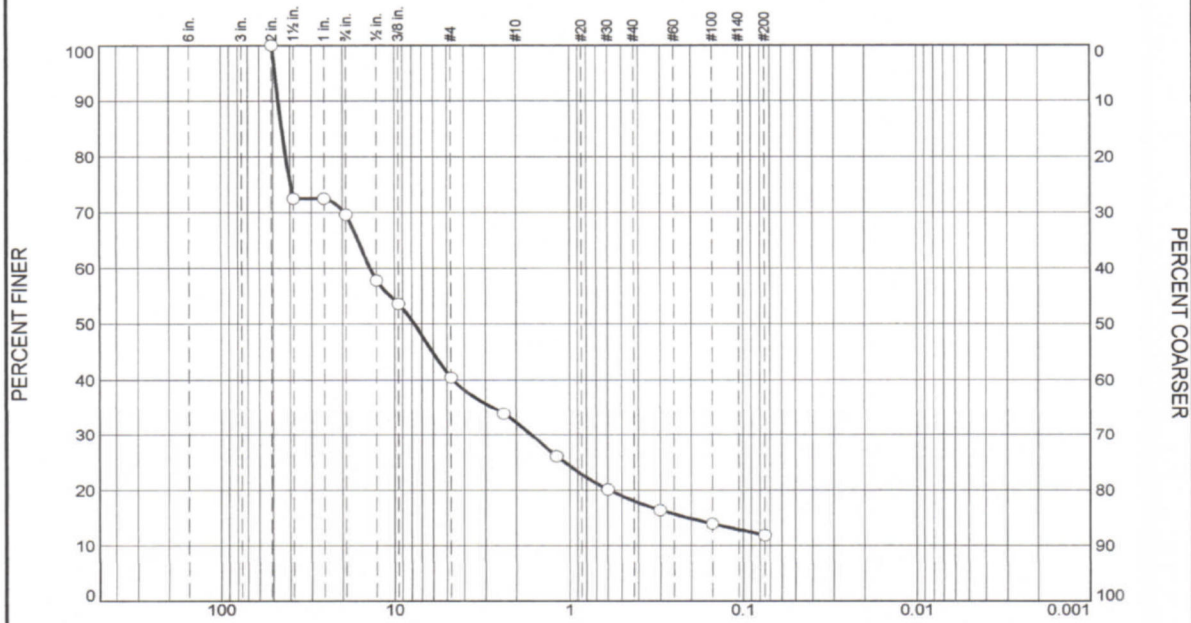
Drawn By: MTH

Date: 17-Feb-2016

Approved By: MR

Project No. 102330-4000

Particle Size Distribution Report



GRAIN SIZE - mm.							
% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	30	30	8	14	6	12	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1 1/2"	72		
1"	72		
3/4"	70		
1/2"	58		
3/8"	54		
#4	40		
#8	34		
#16	26		
#30	20		
#50	16		
#100	14		
#200	12		

Material Description
Poorly Graded GRAVEL with Clay and Sand

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= 46.5950 D₈₅= 44.4863
 D₅₀= 7.7290 D₃₀= 1.6488
 D₁₀= C_u= C_c=

Classification
 USCS= GP-GC AASHTO=

Remarks
 Moisture = 6.1%

* (no specification provided)

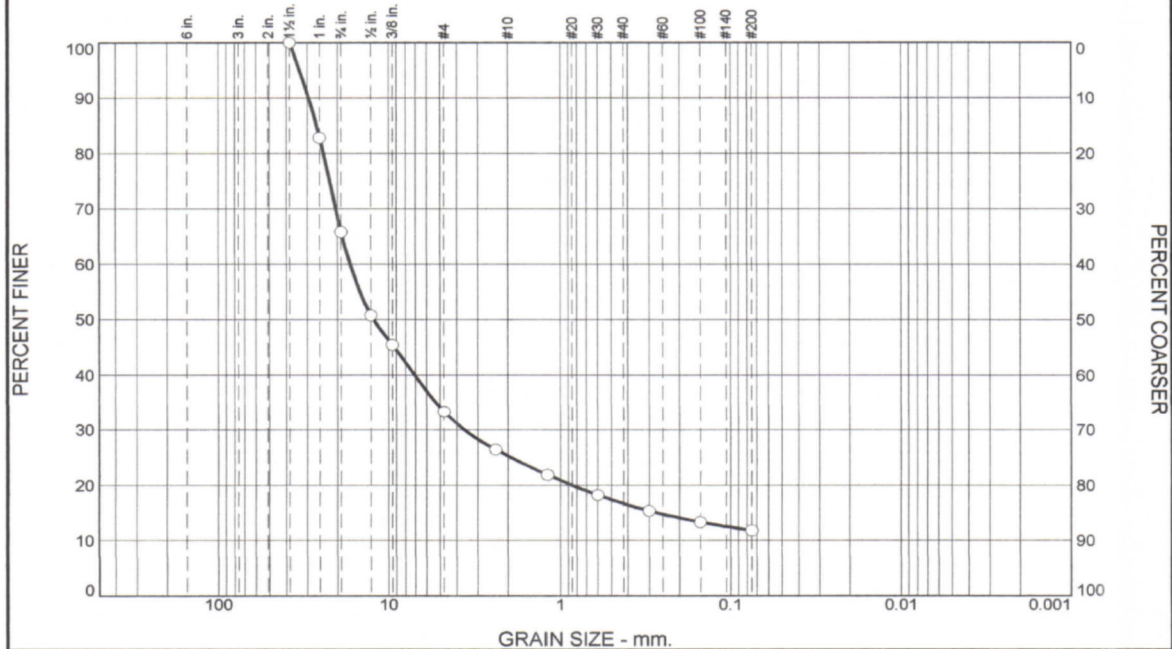
Source of Sample: A-16-003 Depth: 2' - 3.5' Date: 1-29-16
 Sample Number: R-1

 M.T. HALL & ASSOCIATES, INC. Materials Testing & Inspection 609 Enter Avenue, Suite 4 Chico, CA 95928 Phone: (530) 893-2181 Fax: (530) 893-4047 Website: www.mthallassociates.com	Client: Willdan Project: Watson Creek Bridge (CR 305) Reconstruction - Glenn County Project No: 114-16 Lab # C-14-16
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Tested By: Rick Kuntz Checked By: Tom Hall P.E.

PARTICLE SIZE DISTRIBUTION TEST WATSON CREEK BRIDGE (REPLACE) GLENN COUNTY, CALIFORNIA	 <table border="1" style="width: 100%;"> <tr> <td>Drawn By: MTH</td> <td>Date: 17-Feb-2016</td> </tr> <tr> <td>Approved By: MR</td> <td>Project No. 102330-4000</td> </tr> </table>	Drawn By: MTH	Date: 17-Feb-2016	Approved By: MR	Project No. 102330-4000
Drawn By: MTH	Date: 17-Feb-2016				
Approved By: MR	Project No. 102330-4000				

Particle Size Distribution Report



% +3"		% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0		34	33	8	8	5	12	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100		
1"	83		
3/4"	66		
1/2"	51		
3/8"	45		
#4	33		
#8	26		
#16	22		
#30	18		
#50	15		
#100	13		
#200	12		

* (no specification provided)

Material Description

Poorly Graded GRAVEL with Clay and Sand

PL= **Atterberg Limits** LL= PI=

D₉₀= 29.4129 **Coefficients** D₈₅= 26.4651

D₅₀= 12.2585 D₃₀= 3.6157 D₁₅= 0.2722

D₁₀= C_u= C_c=

USCS= GP-GC **Classification** AASHTO=

Moisture = 4.6% **Remarks**

Source of Sample: A-16-003 Depth: 5'-6.5'

Sample Number: S-2

Date: 2-1-16



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Material Testing & Inspection
609 Enter Avenue, Suite 4 Chico, CA 95928
Phone: (530) 893-2181 Fax: (530) 893-4847
Website: www.mthallassoc.com

Client: Willdan
Project: Watson Creek Bridge (CR 305) Reconstruction - Glenn County
Project No: 114-16 Lab # C-15-16

Tested By: Rick Kuntz Checked By: Tom Hall P.E.

PARTICLE SIZE DISTRIBUTION TEST

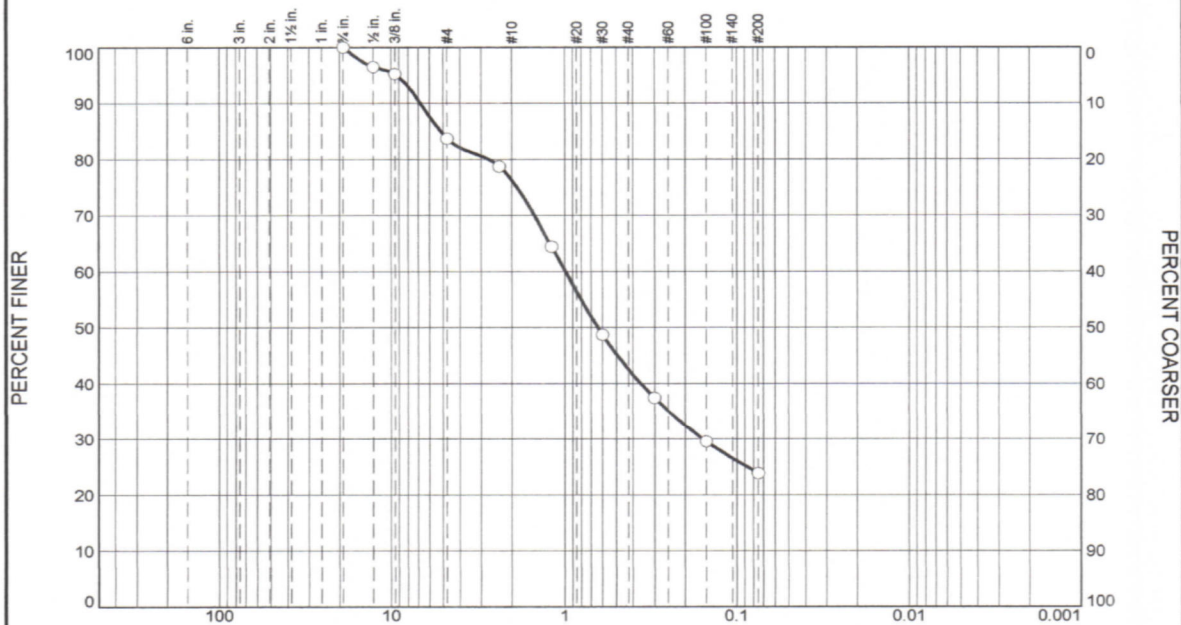
**WATSON CREEK BRIDGE (REPLACE)
GLENN COUNTY, CALIFORNIA**



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Drawn By: MTH	Date: 17-Feb-2016
Approved By: MR	Project No. 102330-4000

Particle Size Distribution Report



% +3"		% Gravel		% Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay		
0		0	16	8	33	19	24	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	97		
3/8"	95		
#4	84		
#8	79		
#16	64		
#30	49		
#50	37		
#100	30		
#200	24		

Material Description

Clayey SAND with Gravel

PL= **Atterberg Limits** PI=

LL=

Coefficients

D₉₀= 6.7850 D₈₅= 5.1974 D₆₀= 0.9901

D₅₀= 0.6401 D₃₀= 0.1576 D₁₅=

C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Moisture = 12.4%

* (no specification provided)

Source of Sample: A-16-003 Depth: 15' -16.5'

Sample Number: S-4

Date: 2-1-16



M.T. HALL & ASSOCIATES, INC.
Materials Testing & Inspection
609 Ender Avenue, Suite 4 Chico, CA 95928
Phone: (530) 893-2181 Fax: (530) 893-4647
Website: www.mthallassociates.com

Client: Willdan

Project: Watson Creek Bridge (CR 305) Reconstruction - Glenn County

Project No: 114-16

Lab # C-17-16

Tested By: Rick Kuntz Checked By: Tom Hall P.E.

PARTICLE SIZE DISTRIBUTION TEST

**WATSON CREEK BRIDGE (REPLACE)
GLENN COUNTY, CALIFORNIA**



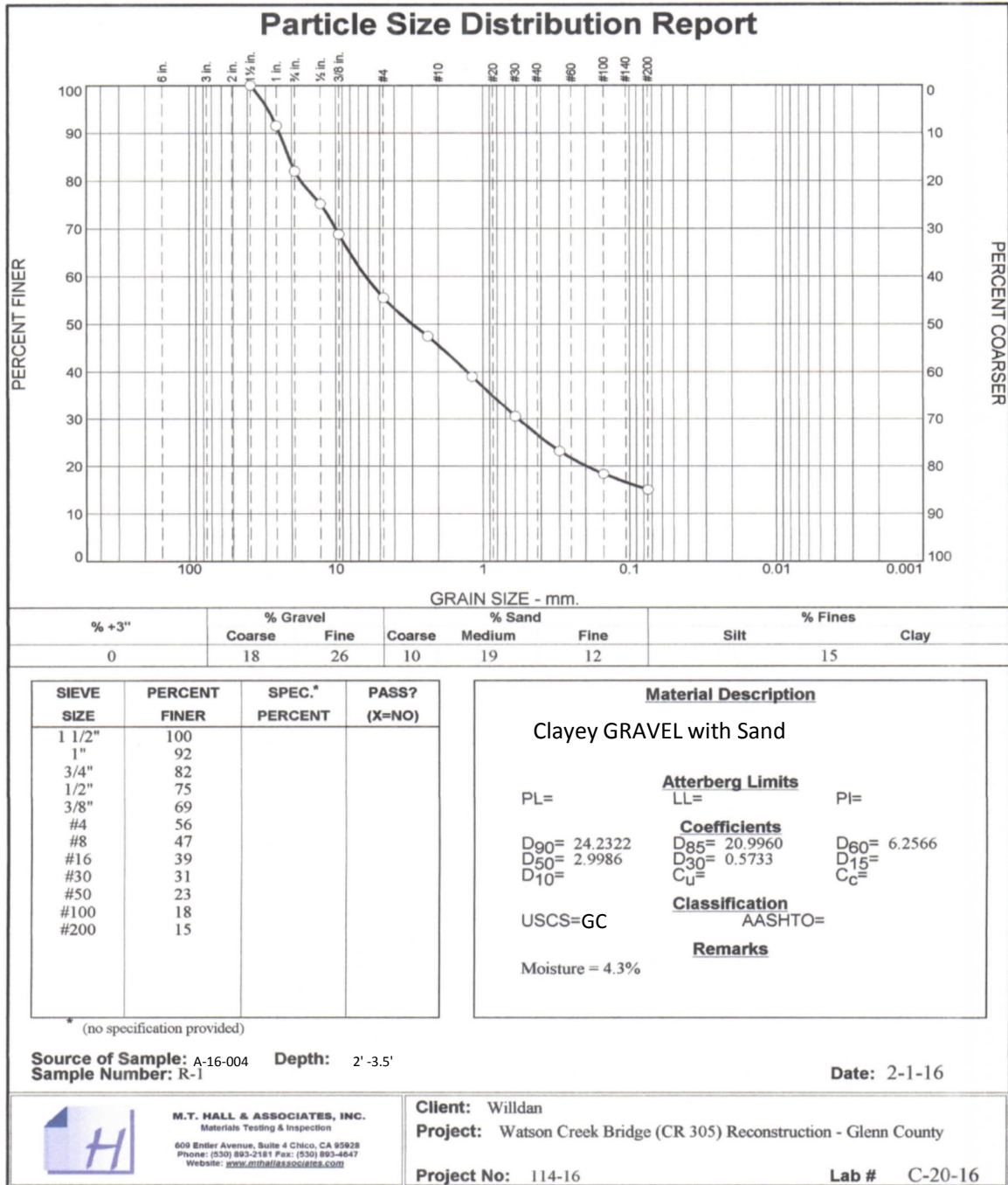
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Drawn By: MTH

Date: 17-Feb-2016

Approved By: MR

Project No. 102330-4000



Tested By: Rick Kuntz **Checked By:** Tom Hall P.E.

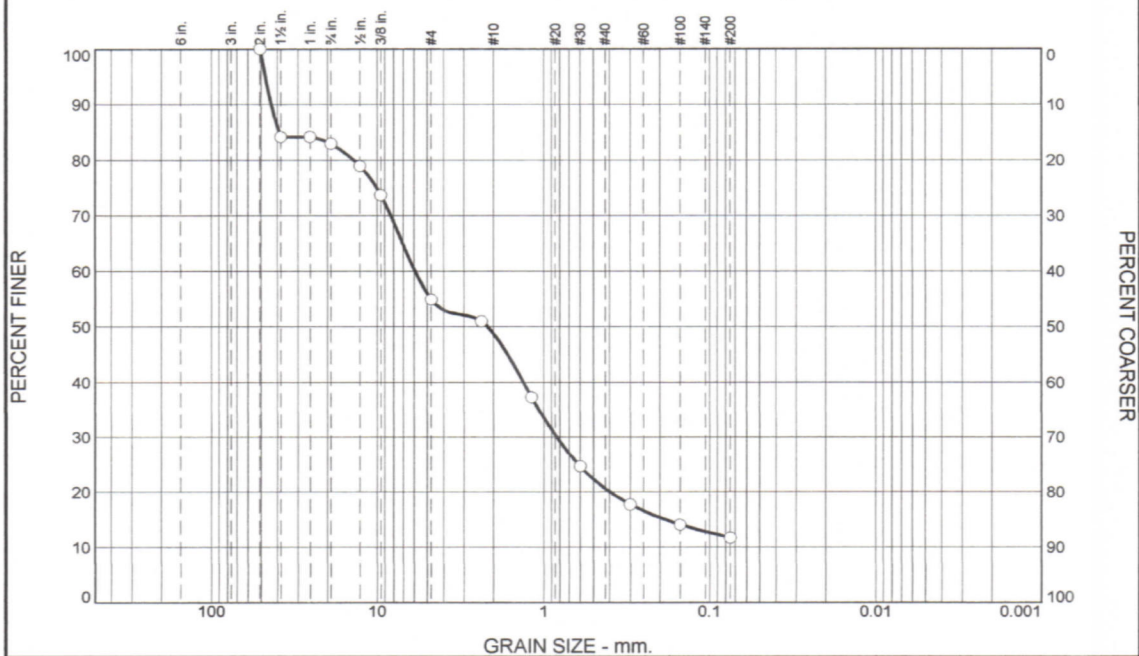
PARTICLE SIZE DISTRIBUTION TEST

**WATSON CREEK BRIDGE (REPLACE)
GLENN COUNTY, CALIFORNIA**



Drawn By: MTH	Date: 17-Feb-2016
Approved By: MR	Project No. 102330-4000

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	17	28	6	28	9	12	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1 1/2"	84		
1"	84		
3/4"	83		
1/2"	79		
3/8"	74		
#4	55		
#8	51		
#16	37		
#30	25		
#50	18		
#100	14		
#200	12		

* (no specification provided)

Material Description

Well Graded GRAVEL with Clay and Sand

Atterberg Limits
 PL= LL= PI=
Coefficients
 D₉₀= 43.4628 D₈₅= 39.0931 D₆₀= 5.9474
 D₅₀= 2.1675 D₃₀= 0.8330 D₁₅= 0.1850
 D₁₀= C_u= C_c=
Classification
 USCS=GW-GC AASHTO=
Remarks
 Moisture = 7.7%

Source of Sample: A-16-004 Depth: 10' - 11.5' Sample Number: R-3

Date: 1-29-16



M.T. HALL & ASSOCIATES, INC.
 Materials Testing & Inspection
 609 Enter Avenue, Suite 4 Chico, CA 95928
 Phone: (530) 893-2181 Fax: (530) 893-4647
 Website: www.mthallassociates.com

Client: Willdan
 Project: Watson Creek Bridge (CR 305) Reconstruction - Glenn County

Project No: 114-16

Lab # C-22-16

Tested By: Rick Kuntz Checked By: Tom Hall P.E.

PARTICLE SIZE DISTRIBUTION TEST

WATSON CREEK BRIDGE (REPLACE)
GLENN COUNTY, CALIFORNIA



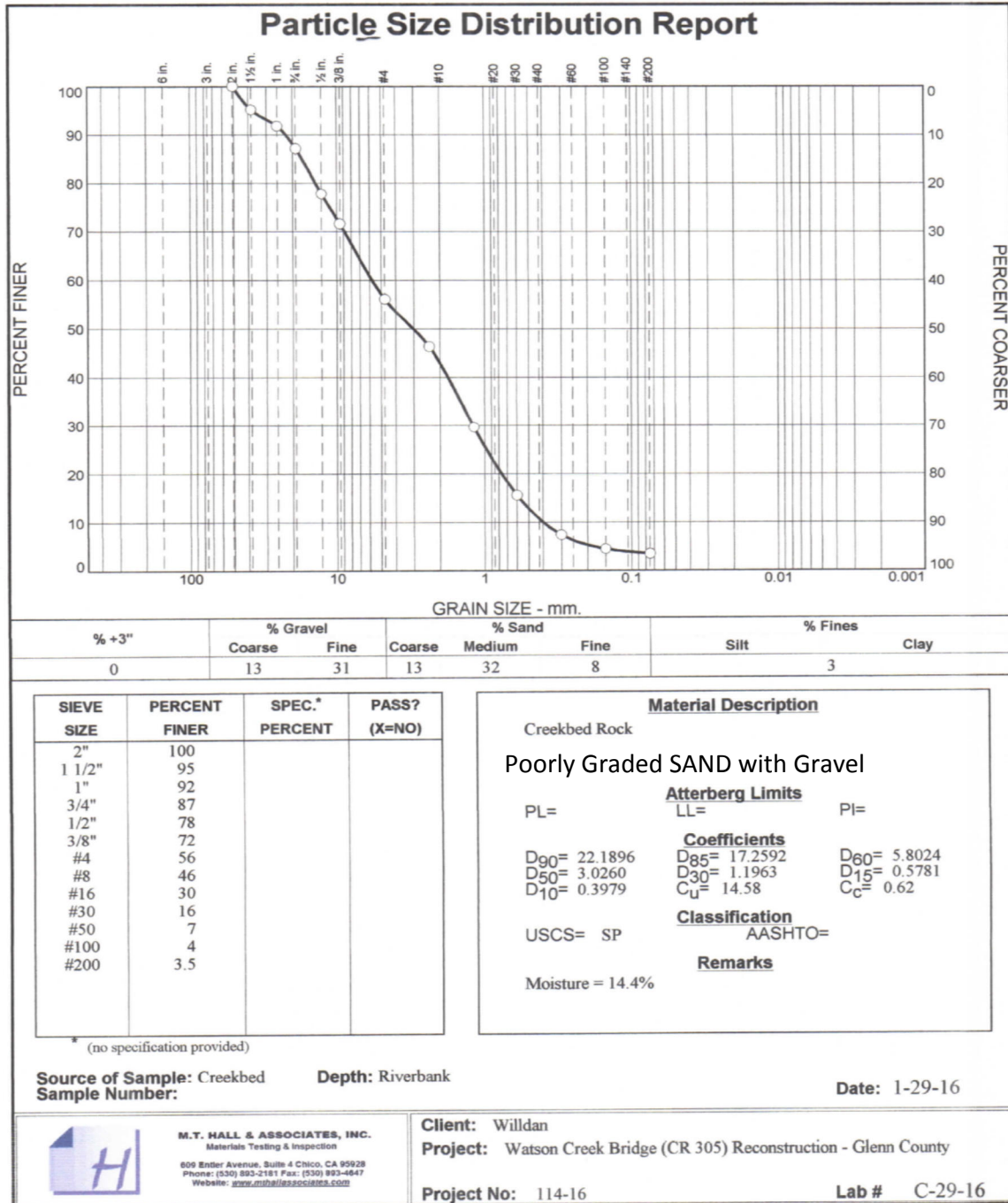
WILLDAN | extending your reach
 Geotechnical

Drawn By: MTH

Date: 17-Feb-2016

Approved By: MR

Project No. 102330-4000



Tested By: Rick Kuntz Checked By: Tom Hall P.E.

PARTICLE SIZE DISTRIBUTION TEST

**WATSON CREEK BRIDGE (REPLACE)
GLENN COUNTY, CALIFORNIA**



WILLDAN
Geotechnical

extending
your
reach

Drawn By: MTH

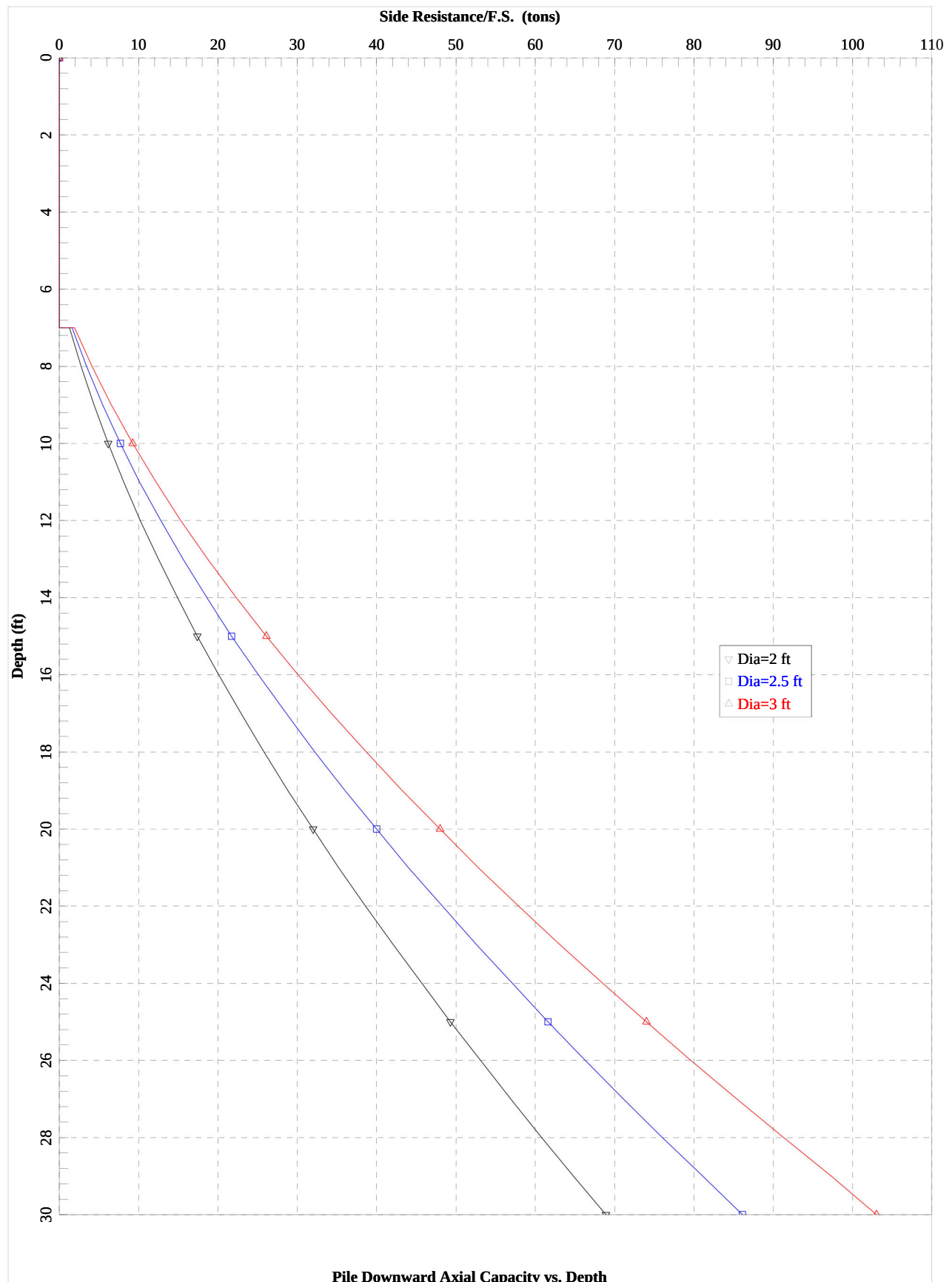
Date: 17-Feb-2016

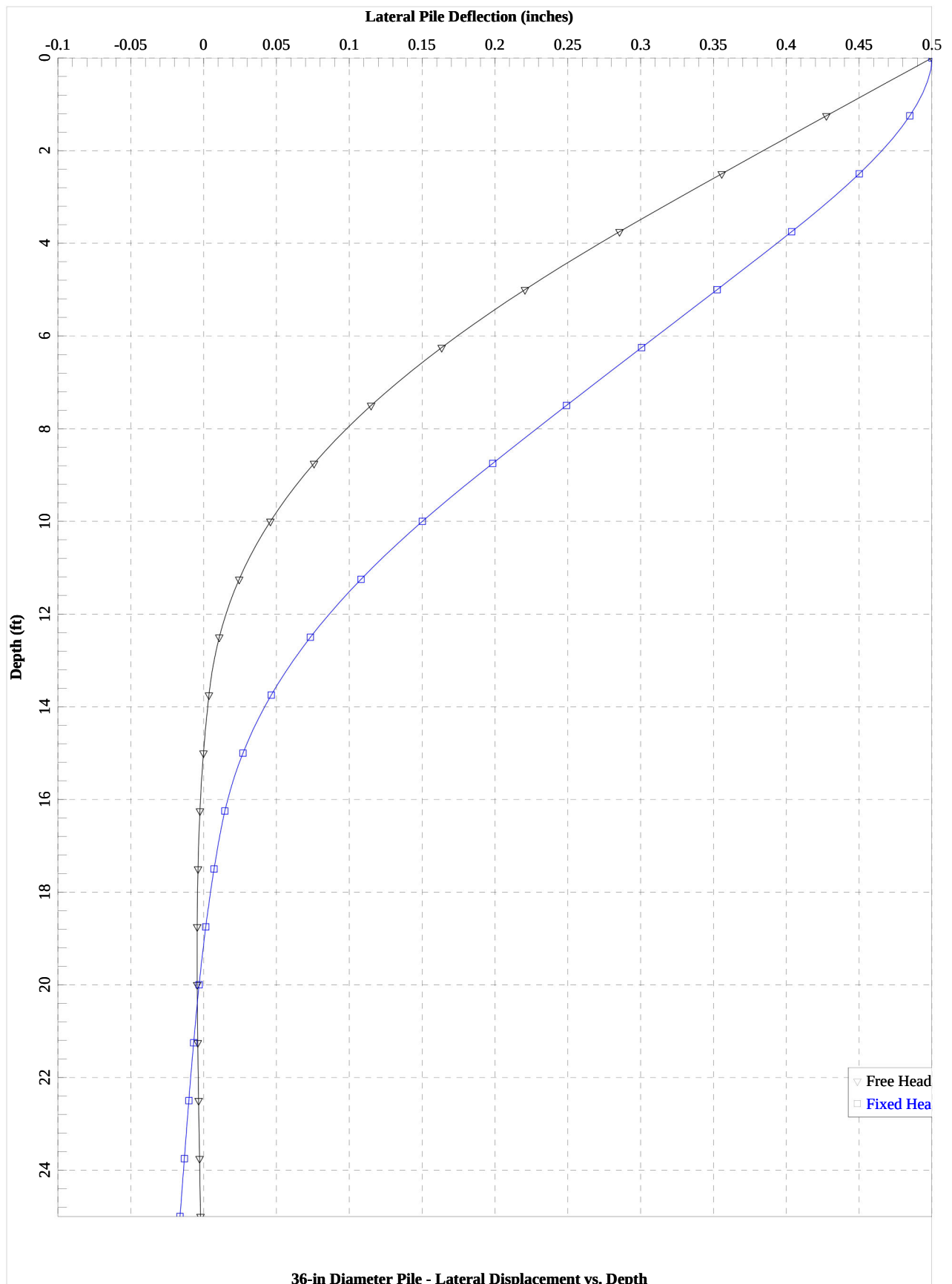
Approved By: MR

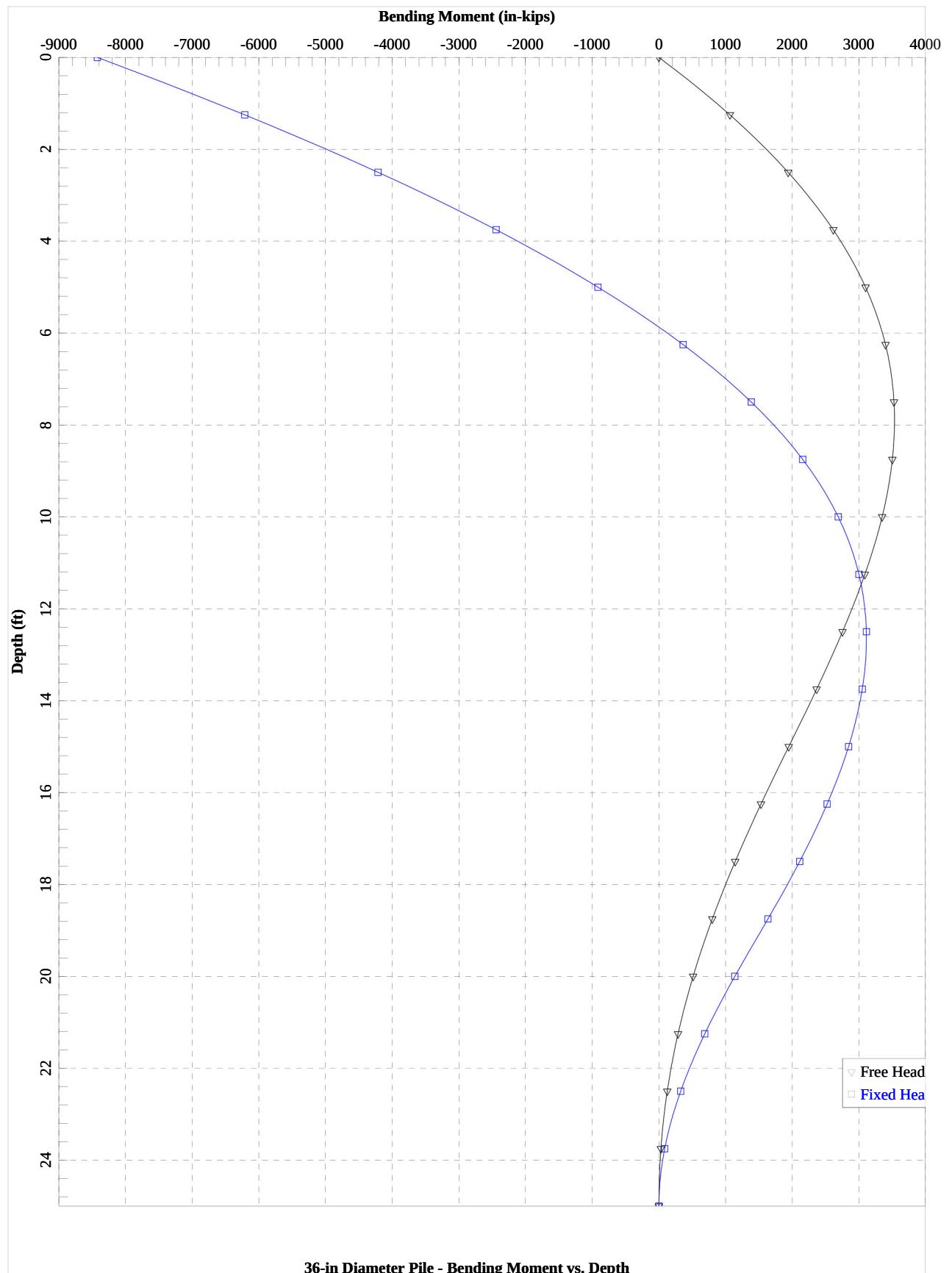
Project No. 102330-4000

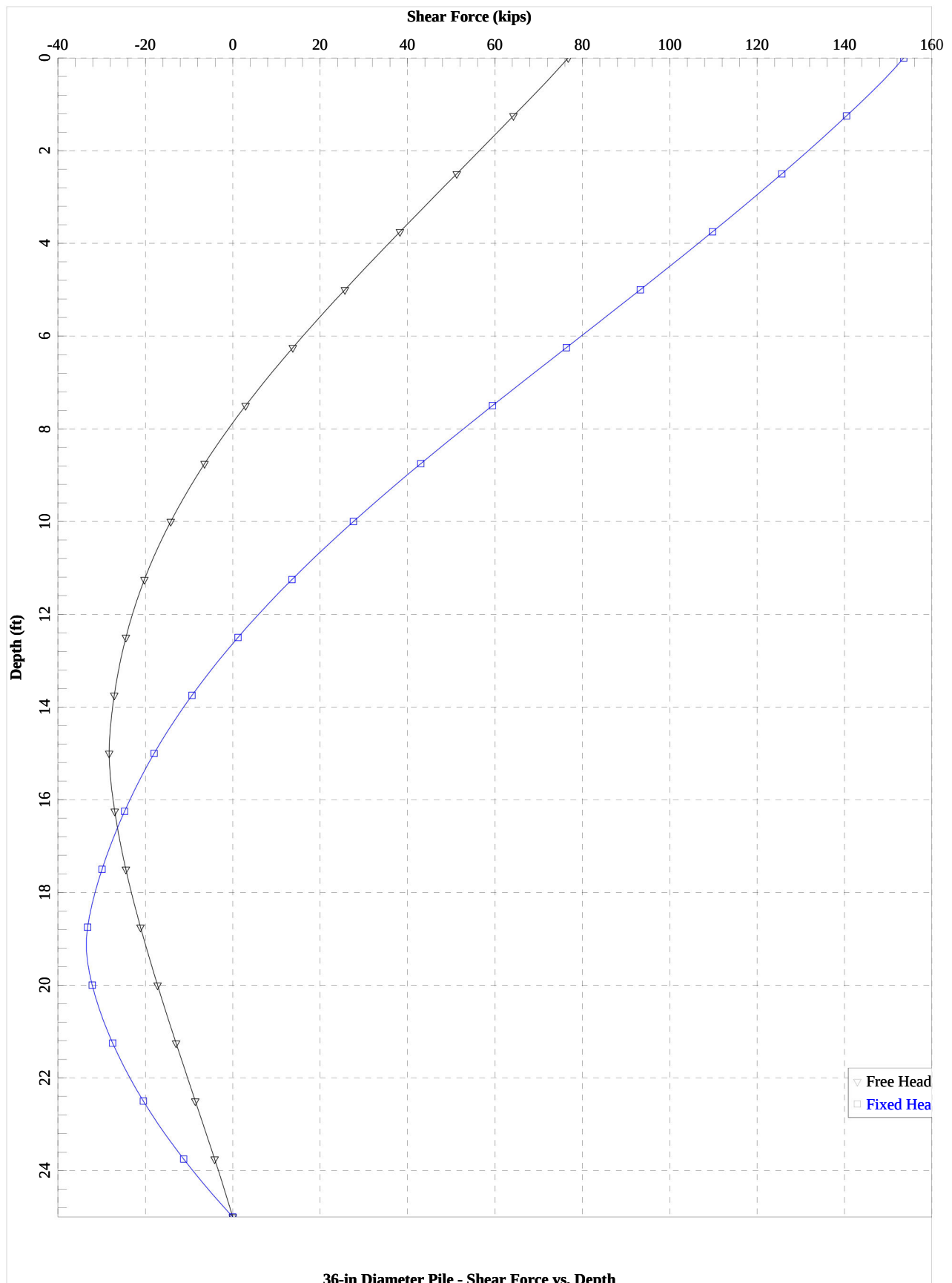
APPENDIX C: PILE CAPACITY GRAPHS







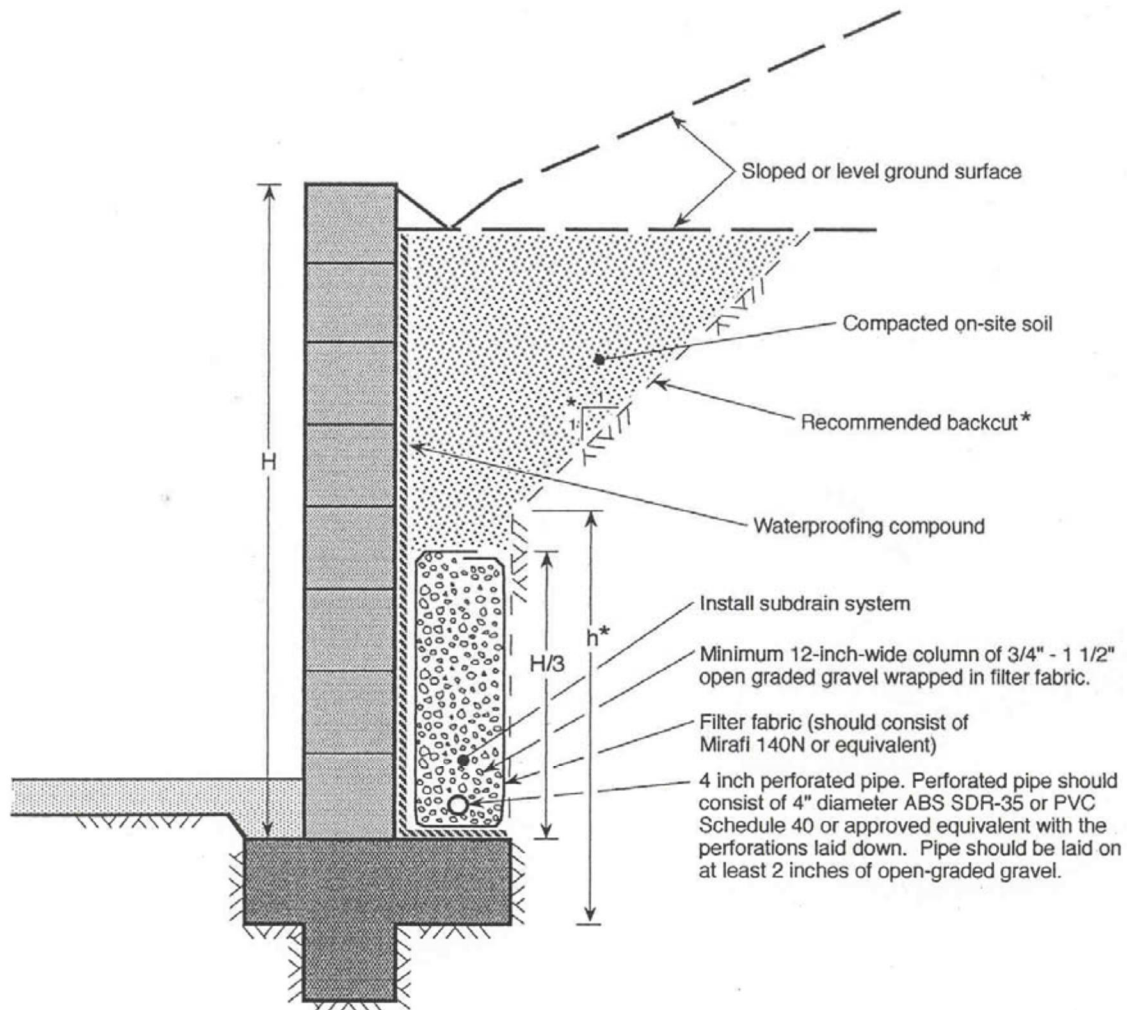




APPENDIX D: TYPICAL RETAINING WALL BACKFILL DETAILS

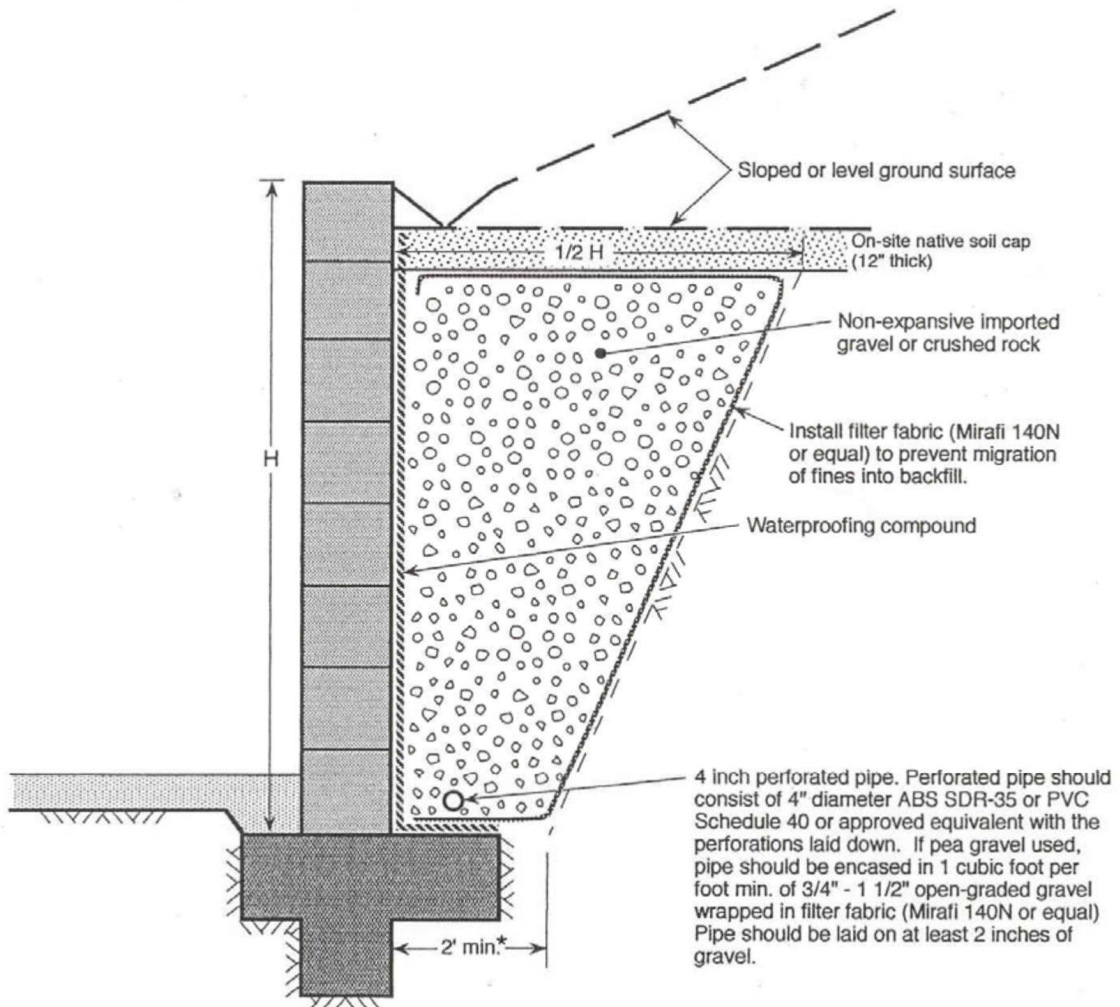


NATIVE SOIL BACKFILL



* Vertical height (h) and slope angle of backcut per soils report. Based on geologic conditions, configuration of backcut may require revisions (i.e. reduced vertical height, revised slope angle, etc.)

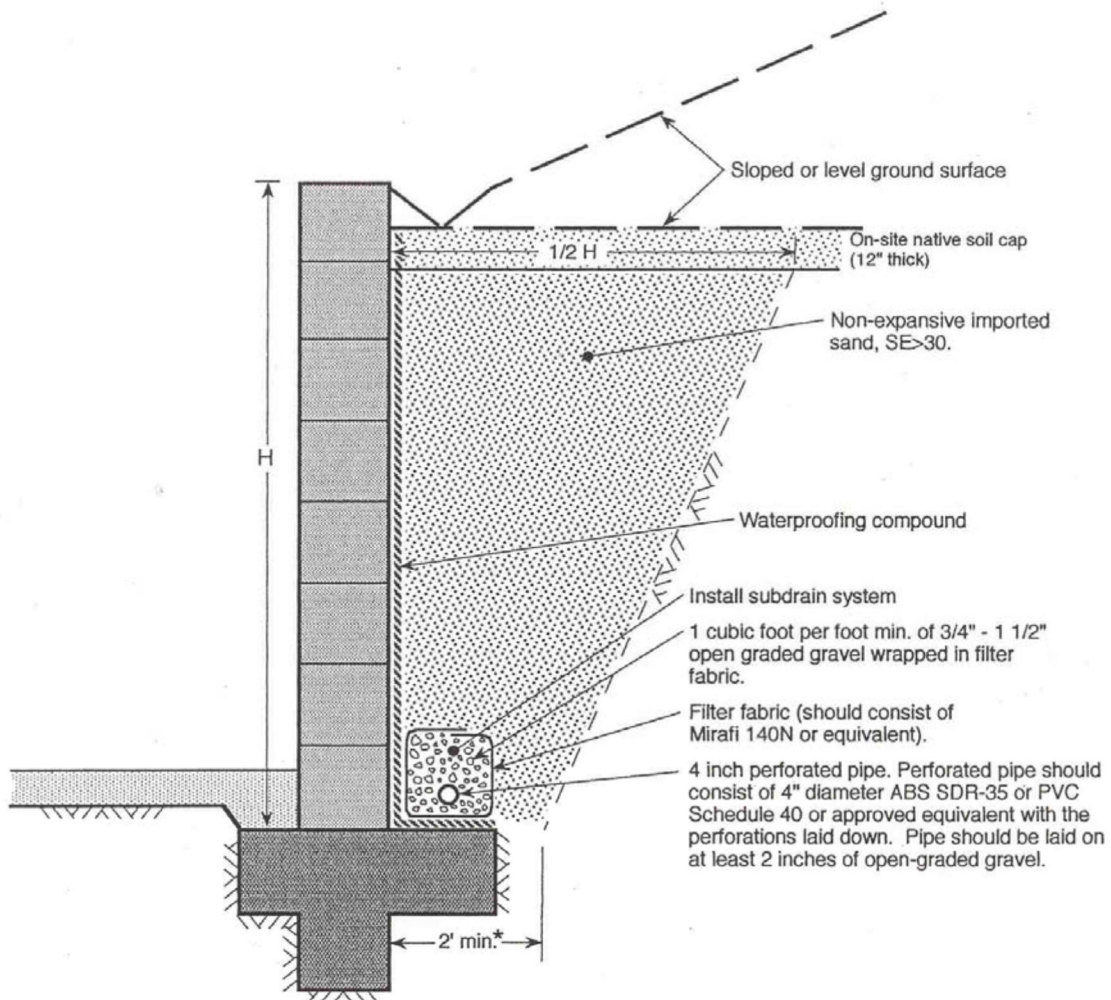
IMPORTED GRAVEL OR CRUSHED ROCK BACKFILL



* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.



IMPORTED SAND BACKFILL



* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.